Sinkhole and subsidence size distribution across dolomitic land in Gauteng

Performance of thermoplastic road-marking material

Modelling manpower and equipment productivity in tall residential building projects in developing countries

Can detrimental carbonation of cement or lime-stabilised road base layers, and the occurrence of biscuit layers as a result of carbonation, be controlled by proper construction techniques only?

Impact of low viscosity grade bitumen on foaming characteristics

Determination of base and shaft resistance factors for reliability-based design of piles
## CONTENTS

<table>
<thead>
<tr>
<th>Page</th>
<th>Title</th>
<th>Authors</th>
<th>DOI</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>Sinkhole and subsidence size distribution across dolomitic land in Gauteng</td>
<td>S Constantinou, J L van Rooy</td>
<td>10.17159/2309-8775/2018/v60n2a1</td>
</tr>
<tr>
<td>9</td>
<td>Performance of thermoplastic road-marking material</td>
<td>S Naidoo, W J vd M Steyn</td>
<td>10.17159/2309-8775/2018/v60n2a2</td>
</tr>
<tr>
<td>23</td>
<td>Modelling manpower and equipment productivity in tall residential building projects in developing countries</td>
<td>M K Parthasarathy, R Murugasan, Ramya Vasan</td>
<td>10.17159/2309-8775/2018/v60n2a3</td>
</tr>
<tr>
<td>34</td>
<td>Can detrimental carbonation of cement or lime-stabilised road base layers, and the occurrence of biscuit layers as a result of carbonation, be controlled by proper construction techniques only?</td>
<td>C J Semmelink, N J Jooste, J E Raubenheimer</td>
<td>10.17159/2309-8775/2018/v60n2a4</td>
</tr>
<tr>
<td>40</td>
<td>Impact of low viscosity grade bitumen on foaming characteristics</td>
<td>S S Kar, A K Swamy, D Tiwari, P K Jain</td>
<td>10.17159/2309-8775/2018/v60n2a5</td>
</tr>
</tbody>
</table>
Sinkhole and subsidence size distribution across dolomitic land in Gauteng

S Constantinou, J L van Rooy

Karst-related sinkholes and subsidence occur on areas underlain by Chuniespoort Group dolomite bedrock in the Gauteng Province, South Africa. Although dolomite land is found across several South African provinces, sinkhole and subsidence formation in Gauteng, the economic hub of South Africa, has been considerably more severe than in other provinces. Thousands of sinkhole and subsidence events have occurred in the past 50 years. Previously, data on sinkhole and subsidence occurrence was amassed separately by various consultants, companies, research institutions and state authorities. There is no legal requirement for sinkhole and subsidence events to be reported to a central authority, yet the data is crucial for future assessment of sinkhole hazards and decision-making. Information regarding the sinkhole record for Gauteng was collated to develop an inventory of the study area. Compiling a comprehensive database presented many challenges; most importantly the large volume of incomplete data that could not be retrieved. Sinkholes, and subsidence and crack events prior to the end of 2011 were considered for Gauteng. Data was organised into multi-wave frequency tables, and various aspects were then analysed. This paper deals with available dimension data (i.e. diameter and depth) across the Gauteng Province.

INTRODUCTION

According to Ford and Williams (1992), sinkholes are the most diagnostic surface expression of karst landscapes and can be found extensively throughout the world (approximately 7–10% of the earth land surface has been classified as karst terrain). Karst-related sinkholes and subsidence events occur on areas underlain by dolomite ground in South Africa. Dolomite land occurs across several South African provinces, including Gauteng, Mpumalanga, Limpopo, North West and the Northern Cape. However, sinkhole formation in Gauteng Province poses a greater risk to infrastructure than in any of the other provinces to date.

Thousands of sinkhole, subsidence and crack events have occurred in the past 50 years within the Gauteng Province. According to Buttrick et al (2011), four to five million people currently work or reside on dolomite land, and these instability events have resulted in loss of life and/or damage to property when they coincide with human development. Damage to buildings and other infrastructure has been more severe on dolomite than on any other rock type in South Africa (Brink 1979; Wagener 1985), and thus far 39 people have lost their lives (Buttrick & Roux 1993).

In the past, data on sinkhole and subsidence occurrences was amassed separately in papers, research theses and databases held by various consultants, companies, research institutions and state authorities. There is currently no legal requirement for sinkhole and subsidence events to be reported to a central authority (Heath & Oosthuizen 2008). Sinkhole statistics have not been available since the work by Wolmarans (1984) and Schöning (1990), although Heath and Oosthuizen (2008) indicated in excess of 2 400 instability events in a preliminary overview of the sinkhole record for South Africa. More recent research (Richardson 2013) shows numbers are in excess of 3 000 (sinkhole/subsidence/ground crack events). Sinkhole and subsidence data is crucial for future assessment of sinkhole hazards and decision-making.

DOLOMITE AND SINHOLE FORMATION

In Gauteng Province the Malmani Subgroup (Chuniespoort Group, Transvaal Supergroup) dolomites occupy a surface area of approximately 2 576 km² (14% of Gauteng’s surface area) and form two broad arches (northern and southern) around the Halfway House Granite. However, the area...
considered as “dolomitic land” (Bosch 2003), including areas covered by younger non-dolomitic formations but still underlain by dolomite at depth (within 60–100 m), covers an area of approximately 4 005 km², i.e. 24% of Gauteng’s surface area (see Figure 1).

The dolomite rock which occurs in the Transvaal Supergroup comprises a series of alternating bands of insoluble chert and soluble dolomite. Small amounts of iron and manganese carbonates are also commonly present (Brink & Partridge 1965). Dolomite rock possesses a system of discontinuities (fractures, joints and faults) which act as preferential solution passages for water ingress. Although dolomite rock is relatively impervious (porosity less than 0.3%) and insoluble in pure water, rainwater which has become charged with carbon dioxide in its passage through the atmosphere and the soil, flows along these discontinuities, slowly acting to dissolve the rock (Brink 1979). Eventually steep valleys are corroded within the shear zones (of faults), with dolomite rock standing as pinnacles between the corroded grykes/valleys. The hard rock dolomite is usually covered by an upward succession of residual products (weathered dolomite, wad1, chert and residual chert) that are often overlain by younger formations or are intruded by dykes or sills (Brink 1979).

The residual mantle can be extremely irregular (Martini in Johnson et al 2006). A residual product such as wad has low strength in most cases, and is highly compressible and may be tens of metres thick. The vertical succession of these residual products normally reflects a decrease in strength and compressibility with depth. Voids are also sometimes present in the wad (De Bruyn & Bell 2001).

The mechanism of sinkhole and subsidence formation is described in detail by Jennings et al (1965) and Buttrick (1992). They describe ground subsidence in dolomitic formations as taking place in one of two ways: as a gradual or caving subsidence or a rapid and catastrophic sinkhole. These events are most often caused by ingress water or lowering of the groundwater table.

**SINKHOLES AND SUBSIDENCES IN THE GAUTENG PROVINCE**

Numerous papers and reports have been published, and an abundance of research exists on the subject of sinkholes and investigation techniques on dolomitic land in South Africa, particularly on Gauteng in which the bulk of the incidents have been reported. Historically the Far West Rand (the area from Westonaria to Carletonville) was the focus of many studies (i.e. Brink & Partridge 1965; Brink 1979; Kleywegt & Pike 1982, Wolmarans 1984) due to the frequent occurrence of sinkholes and subsidences in the 1960s and 1970s resulting from dewatering of several of the groundwater compartments by mining companies. The area south of Pretoria has also seen scores of sinkholes in recent history (Roux 1984; Schöning1990; De Bruyn & Trollip 2000; Heath & Oosthuizen 2008; Buttrick et al 2011), while relatively few events have been reported in the municipalities of Ekurhuleni (De Bruyn & Trollip 2000; Heath & Oosthuizen 2008), Sedibeng and the City of Johannesburg, all within the Gauteng Province.

**DEVELOPMENT OF SIZE CATEGORIES**

While Wolmarans (1984) indicates numerous sinkhole events prior to 1984 on the Far West Rand, dimension data was not given and size categories were not yet developed. Research by Schöning (1990) analysed surface diameters and depths of over 200 sinkhole events in the area south of Pretoria (Tables 1 and 2 refer). Schöning's results show that most sinkholes (>50%)

**Table 1 Sinkhole diameters in the area south of Pretoria (after: Schöning 1990)**

<table>
<thead>
<tr>
<th>Sinkhole diameter</th>
<th>0–2 m</th>
<th>2–4 m</th>
<th>4–6 m</th>
<th>6–8 m</th>
<th>&gt;10 m</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total (%)</td>
<td>93 (38.9)</td>
<td>54 (22.6)</td>
<td>29 (12.1)</td>
<td>16 (6.7)</td>
<td>47 (19.7)</td>
<td>239</td>
</tr>
</tbody>
</table>

**Figure 1** Distribution of instability events and dolomitic land across Gauteng in the different District and Metropolitan Municipalities
Table 2 Sinkhole depths in the area south of Pretoria (after: Schöning 1990)

<table>
<thead>
<tr>
<th>Sinkhole diameter</th>
<th>Total (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0–1 m</td>
<td>73 (32.9)</td>
</tr>
<tr>
<td>1–2 m</td>
<td>34 (15.3)</td>
</tr>
<tr>
<td>2–4 m</td>
<td>58 (26.1)</td>
</tr>
<tr>
<td>4–6 m</td>
<td>29 (13.1)</td>
</tr>
<tr>
<td>6–10 m</td>
<td>13 (5.9)</td>
</tr>
<tr>
<td>&gt;10 m</td>
<td>15 (6.8)</td>
</tr>
<tr>
<td>Total</td>
<td>222</td>
</tr>
</tbody>
</table>

Table 3 Diameter categories (after: Buttrick 1992; Buttrick & Van Schalkwyk 1995)

<table>
<thead>
<tr>
<th>Maximum potential development space</th>
<th>*Maximum diameter of surface manifestation (m)</th>
<th>Suggested terminology</th>
</tr>
</thead>
<tbody>
<tr>
<td>Small potential development space</td>
<td>≤2 m</td>
<td>Small sinkhole</td>
</tr>
<tr>
<td>Medium potential development space</td>
<td>2–5 m</td>
<td>Medium sinkhole</td>
</tr>
<tr>
<td>Large potential development space</td>
<td>5–10 m</td>
<td>Large sinkhole</td>
</tr>
<tr>
<td>Very large potential development space</td>
<td>&gt;10 m</td>
<td>Very large sinkhole</td>
</tr>
</tbody>
</table>

* Dimensions are based on study of existing sinkholes.

had a diameter smaller than 4 m and a depth of less than 4 m.

Buttrick (1992), for the ‘Method of Scenario Supposition’, used a historical frequency of sinkhole and subsidence events on a type area south of Pretoria, along with geophysical surveys and borehole results, to create a method to systematically characterise dolomite hazard.

Buttrick (1992), Buttrick and Van Schalkwyk (1995), and Buttrick et al (2001) describe a number of factors which can be used to evaluate the possible formation of sinkholes, including the nature and mobilisation potential of the blanketing layer, receptacles, mobilising agents and the maximum potential development space. The maximum size sinkhole can be assessed by estimating the maximum potential development space, which is associated with a receptacle and depends on the depth and ‘angle of draw’ of the overburden materials. The full realisation of the potential development space depends on whether the receptacle is large enough to accommodate all the material eroded from the overburden (Buttrick 1992).

Buttrick (1992) proposed broad categories of “potential development space” and the related scale of potential maximum size sinkholes (diameter) as input into hazard assessment (Table 3). Buttrick and Van Schalkwyk (1995) later amended the broad categories, as shown in Table 4.

Heath and Oosthuizen (2008) analysed sinkhole dimensions (based on limited records) for the area south of Pretoria, and concluded that the largest proportion (29%) of sinkholes in this area has a diameter range of 5–15 m, most (>50%) are shown to be less than 15 m in diameter.

Another recent assessment using sinkhole size data involved the design of the Gautrain Rapid Rail Link (Gautrain), which passes through Centurion (south of Pretoria) and across approximately 15 km of dolomitic ground. As the alignment could not avoid the dolomitic ground, the potential sinkhole size that could occur had to be designed for to accommodate a sudden loss of support. A study (Sartain et al 2011) of the frequency of sinkhole occurrence with a diameter >15 m was undertaken and a database of sinkholes in the Centurion region was compiled (some 287 sinkholes). The diameter distribution was established. It was concluded that the most appropriate sinkhole diameter to design for was 15 m; this gave a tolerable risk with 95% confidence.

**METHODOLOGY**

The study collated all available data on sinkholes and subsidences in Gauteng in an attempt to establish a size distribution. The data collection process included review of reports and historical maps held at the Council for Geoscience (CGS), databases compiled by CGS, and supplied to the CGS by various consultants, companies and state authorities, various research theses, topographic maps (1:50 000) covering dolomitic areas, published during 1984, and aerial photographs covering the study areas viewed on a Geographic Information System (GIS). A typical data point may be recorded as a sinkhole or subsidence, with various descriptive information on the event noted. A large number of the data points are incomplete, with one or more of the parameters related to an event missing. Therefore the sample number used in the analysis of diameter or depth data is limited to points with complete information.

Sinkhole diameter is a very important factor, as land use and design decisions are made according to the expected size of such an event, within the engineering geology industry in South Africa. The size distribution was analysed according to the size categories proposed by Buttrick et al (2001) in Table 4; however, it was necessary to be more specific in terms of defining the start and end of each category when assigning occurrences to a specific size category. The categories shown in Table 5 were used in terms of diameter to avoid overlap in categories. Depth categories were previously suggested by Schöning (1990); however, a modification of Schöning’s depth categories and Buttrick’s diameter categories was used (Table 6), in that the

**Table 4 Diameter categories (after: Buttrick et al 2001)**

<table>
<thead>
<tr>
<th>Maximum diameter of surface manifestation (m)</th>
<th>Terminology</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;2 m</td>
<td>Small sinkhole</td>
</tr>
<tr>
<td>2–5 m</td>
<td>Medium sinkhole</td>
</tr>
<tr>
<td>5–15 m</td>
<td>Large sinkhole</td>
</tr>
<tr>
<td>&gt;15 m</td>
<td>Very large sinkhole</td>
</tr>
</tbody>
</table>

**Table 5 Sinkhole and subsidence diameter categories**

<table>
<thead>
<tr>
<th>Small</th>
<th>Medium</th>
<th>Large</th>
<th>Very large</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 2 m</td>
<td>&gt;2 m – ≤ 5 m</td>
<td>&gt;5 m – ≤ 15 m</td>
<td>&gt;15 m</td>
</tr>
</tbody>
</table>

**Table 6 Sinkhole and subsidence depth categories**

<table>
<thead>
<tr>
<th>Small</th>
<th>Medium</th>
<th>Large</th>
<th>Very large</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 1 m</td>
<td>&gt;1 m – ≤ 5 m</td>
<td>&gt;5 m – ≤ 15 m</td>
<td>&gt;15 m</td>
</tr>
</tbody>
</table>
depth categories have not been described in terms of small, medium or large, etc, as the diameters have.

RESULTS AND INTERPRETATION

Over 2600 defined sinkhole or subsid- ence events were compiled as part of the inventory. However, very limited data sets had complete information with regard to
diameter and depth of events (i.e. approximately 40% and 30% respectively).

Sinkhole diameters

Considering the available sinkhole diameter data across the municipalities of the West Rand, City of Tshwane and Ekurhuleni, where almost all (99%) of the data originates, the dominant size range is between 5 m and 15 m diameter overall, and based on the available data (n = 996), 77% of the sinkholes have a diameter of 15 m or less.

With respect to available sinkhole diameters for each municipality, the follow- ing was shown (refer to Table 5, Graph 1 and Figures 2–4):

- >60% of sinkholes in the West Rand are large to very large
- >60% of sinkholes in Tshwane (Pretoria) are medium to large
- >70% of sinkholes in Ekurhuleni are small to medium.

The above may give some indication of the development space that can generally be expected in each area. For example, the West Rand is notorious for very large sinkholes and typically the depth to dolomite bedrock can reach very great depths. Areas in the West Rand are also covered by younger deposits. The depth to bedrock in the Tshwane region is typically intermediate to great, while areas in Ekurhuleni are known for shallow dolomite bedrock (except in places covered by thick Karoo sediments, where dewatering has led to very large events). The other factors indicated by Buttrick (1992), Buttrick and Van Schalkwyk (1995) and Buttrick et al (2001), i.e. nature of the blanketing layer, also play a role.
Figure 3: Sinkhole and subsidence size distribution across Tshwane (Pretoria)

Figure 4: Sinkhole and subsidence size distribution across Ekurhuleni
Sinkhole depths
Bearing in mind that the available sinkhole depth data across the municipalities of the West Rand, City of Tshwane and Ekurhuleni is very limited and is typically estimated by the recorder rather than measured, in general, and based on the available data (n = 821), 95% of the sinkholes have a depth of 15 m or less.

With respect to sinkhole depth for each municipality (Graph 2) most events (>50%) in all three regions are less than or equal to 5 m deep. However, some very deep sinkholes are known on the West Rand.

Subsidence diameters and depths
In view of the available subsidence dimension data across the municipalities of the West Rand, City of Tshwane and Ekurhuleni, subsidences are generally large to very large (refer to Table 5 and Figures 2–4). Based on the available data set (n = 210), >60% of the subsidences have a diameter of 15 m or less (Graph 3).

Only a quarter of the recorded subsidence occurrences have depth information recorded. Considering the available data (n = 122), 75% of the subsidence events are 1 m or less in depth and almost all (94%) are 5 m or less in depth (Graph 4).

CONCLUSION
The main objective was to compile historical and current dolomite instability event data for Gauteng Province, and to develop an inventory to be used in statistical analysis to investigate the size distribution of sinkholes and subsidences across three municipalities and for future research.

A large amount of the available data is incomplete and therefore useable sample numbers differ. Dimension data is sometimes estimated and in most cases is also recorded after a period of time following the event, and therefore may indicate a larger size event than that which initially occurred, due to sidewall collapse. Therefore some large events reported may be overestimated in size and some small events may not have been reported if they lacked severity.

Size categories modified from Schöning (1990) and Buttrick et al (2001) were used in considering sinkhole and subsidence size and depth distributions. Based on the available sinkhole diameter data, the dominant diameter size is between 5 m and 15 m diameter overall, with 77% of the sinkholes having a diameter of 15 m or less.

Dominant diameter ranges differ across the three municipalities considered (West Rand, City of Tshwane and Ekurhuleni).

Sinkhole depth information is very limited. Overall, 95% of the sinkholes recorded are less than or equal to 15 m in
depth. With respect to sinkhole depth for each municipality, most events (>50%) in all regions are 5 m or less deep. However, some very deep sinkholes are known to have occurred under special conditions. Subsidences are generally large to very large in diameter and 5 m or less in depth.

RECOMMENDATIONS
A sinkhole database is an ongoing, continuously updated system, and results of analyses may change significantly depending on the current available data set.

It is imperative to have as complete a database as possible. The database used during this research had missing data, which could not be retrieved or updated, and therefore the available data may not be truly representative. It is important that accurate and thorough inventory is undertaken in future, and it should also become mandatory to report such events to a centralised organisation. Sinkhole and subsidance data is crucial for the future assessment of sinkhole hazards and decision-making. The database is important for the future assessment of sinkhole hazards and decision-making. The database is important for the future assessment of sinkhole hazards and decision-making. The natural compressible nature of the rock. Wad forms a favourable horizon for cave or cavity formation. It has a high mobilisation potential, and groundwater seepage causes subsurface erosion. The highly compressible nature also supports the development of shallow but wide subsidences (Martini 2006).

REFERENCES

ACKNOWLEDGEMENTS
The support of the Council for Geoscience to undertake the work described in this paper is acknowledged with thanks. Also thanks to all the consultants, companies and state authorities that submit data to the CGS database.

END NOTE
1 Wad: an insoluble and highly compressible material that consists of a porous mixture of Mn and Fe oxides left behind after dissolution of dolomite and has a cellular structure inherited from the texture of the rock. Wad forms a favourable horizon for cave or cavity formation. It has a high mobilisation potential, and groundwater seepage causes subsurface erosion. The highly compressible nature also supports the development of shallow but wide subsidences (Martini 2006).
The initial skid resistance of white and yellow 1.2 mm thermoplastic road markings complied with the colour specification, while yellow road markings did not comply with the specification. There was no significant increase in R_L or colour compliance of the washed road markings, while there was generally an increase in Q_d after washing the test markings.

The objectives of this study were:

- To determine the R_L and Q_d service lives of various road-marking paints and road-marking materials on asphalt and chip seal road surfaces.
- To determine if there are significant differences in R_L, Q_d and the colour of the road markings when washed with liquid soap mixed with water and hard brooms.
- To check if the colour and skid resistance of the applied road markings comply with the specification.

Based on the data obtained from the study, the following conclusions were drawn:

- The R_L and Q_d service lives of various road-marking paints and road-marking materials on asphalt and chip seal road surfaces were determined as between 1 and 48 months, and 1 and 30 months respectively.
- There was no significant increase in R_L or colour compliance of the washed road markings, while there was generally an increase in Q_d after washing the test markings.
- White road markings generally complied with the colour specification, while yellow road markings did not comply with the specification.
- The initial skid resistance of white and yellow 1.2 mm thermoplastic road markings complied with the specification, while all other road markings did not comply with the specification.

**INTRODUCTION**

Public roads without road markings, especially roads carrying high traffic volumes, would lead to chaos and accidents resulting in injuries and loss of life. The road authorities need to ensure that all signage, i.e. horizontal (road markings) and vertical (traffic signs), are well maintained and conform to the **Southern African Development Community Road Traffic Signs Manual** (SADC RTSM) (SADC 1997) and the **South African Road Traffic Signs Manual** (SARTSM) (1999).

According to Letsoalo (2012), there are approximately 27 road accident fatalities per 100 000 in South Africa, while globally it is approximately 10.3 road accident fatalities per 100 000. Approximately 14 000 people are killed annually on the roads in South Africa. Middle and low-income countries with low vehicle ownership experienced high road fatality rates compared with high-income countries with high vehicle ownership (Letsoalo 2012). This could be attributed to the lack of road safety awareness campaigns, and the lack of maintenance of roads and roadside furniture. The contributing factors to the high number of road accident fatalities are human factors, vehicle factors, road factors and environmental factors (Letsoalo 2012). The highest percentage of road accidents occur at night, and this is the main reason for road markings to be retroreflective in order to guide the road user. It may not necessarily be that the road accidents have occurred as a result of defective road markings, but as road authorities have a certain degree of control over road markings, the authorities should ensure that the markings are continuously maintained to the required standards to eliminate road accidents.
markings as a possible cause of accidents. According to Martin et al (1996), an effective road-marking system facilitates driver guidance, improves traffic flow, contributes to driving comfort and enhances traffic safety.

BACKGROUND
The most important aspect of road markings is that it must be retroreflective (brightness at night under headlights). The minimum R₁ for white and yellow road markings must be 100 microlambert/m²/lux (cd/m²/lx) and 70 microlambert/m²/lux respectively (SADC RTSM) (SADC 1997). There are other important parameters that road markings should conform to, such as Qd, colour, and skid resistance. There are various types of road-marking paints and road-marking materials, namely solvent-borne, waterborne, cold plastic, thermoplastic and preformed tape applied universally. In South Africa, solvent-borne paints, waterborne paints and thermoplastic materials are widely used, with cold plastic materials being increasingly used of late. There are SABS standards on solvent-borne and waterborne road-marking paints covering a wide range of aspects. However, there are insufficient standards on plastic road-marking materials, and road-marking applicators could therefore be applying inferior quality plastic road-marking materials. The Committee of Land Transport Officials (COLTO) (1998) specifications refer to the use of hot melt thermoplastic as a possible road-marking material, subject to its specification in the project specifications. No further guidance or specifications for this material are given. Thermoplastic road-marking material is a 100% solid, environmentally and user-safe compound which consists of binder, pigment, filler and intermix glass beads. There are various suppliers of raw materials from countries such as the UK, US, China, India, Singapore and Saudi Arabia. Some of the local suppliers in South Africa are importing the constituents and formulating the mix, which is sold to road-marking applicators who then apply it on the road network. It is important to draw up a specification and establish testing systems to ensure that imported road-marking materials are suitable for the environment and climate, and that it should perform to at least a certain degree. There is little published information specific to South African road-marking practices and materials, and it remains necessary that on-going attention be given to the development of the local road-marking materials and their application. It is advised in the SARTSM (1999) that each road authority should develop its own estimated service life based on local conditions and experience, due to the variations in the parameters associated with the service life of road-marking paints and materials. Although thermoplastic road markings have been in use for a number of years, there is little agreement on their service life. The problem arises in attempting to establish an expected service life of a particular material on a given roadway. There are too many factors influencing performance to permit an average life to be predicted with any confidence without carrying out research. The South African National Roads Agency Limited (SANRAL) has adopted a performance-based specification in its road-marking contracts. The specification indicates only the required R₁, but experience has shown that some of the important performance standards may not be satisfactory. For example, yellow road markings tend to look white at night and in South Africa yellow road markings have a different meaning to white road markings, and as such could lead to the wrong action taken by the driver. In the City of Tshwane (CoT) the current road-marking maintenance programme is determined mostly by the visibility of the road marking. This is not objective, since each engineer may view the road markings differently, and as such there may be inconsistencies in the road-marking maintenance programme over the city roads.

TYPES OF ROAD-MARKING PAINTS AND MATERIALS
There are various types of road-marking materials available which vary in price and performance. Commonly used road-marking materials include solvent-borne and waterborne paints, thermoplastic (including preformed tape) and cold plastic. The most common road marking applied in South Africa for many years has been the solvent-borne paint, as it has been the cheapest in the market and readily available, being locally manufactured. Solvents contain volatile organic compounds (VOCs) which are carcinogenic. Concern over carcinogens prompted research to develop other pavement marking materials that contain no or lesser quantities of carcinogens, hence the introduction of thermoplastic road-marking materials (Martin et al 1996).

Although South Africa does not have a thermoplastic road-marking material standard, there are good EN standards that could be written into contract documents to ensure quality control of the road-marking product.

ROAD-MARKING STANDARDS
The standards commonly used for road-marking materials worldwide are:
- American Association of State Highway and Transportation Officials (AASHTO) Standards
- British Standards (BS)
- European Standards (EN).

Almost all national standards are based on one or the other of these standards. BS has merged with EN and thus the current BS for thermoplastic road markings is BS EN 2007. The original road-marking standard (BS 3262) was what is known as a “recipe” standard. It specified the percentage of each material to be included in the thermoplastic. Recipe standards have been discontinued for two decades in Europe, but are still widespread elsewhere in the world. Many countries are now switching to performance-based standards.

A recipe-based specification dictates to the manufacturer what has to be included in the material in terms of the ingredients. Any particular thermoplastic made to this specification may perform well or badly, depending on the quality of the raw materials used, the skill with which they are blended and the application, but poor performance is not the manufacturer’s problem, as the manufacturer has met the specification. A performance-based specification, on the other hand, allows the manufacturer to use whatever raw materials he chooses, but it lays down the performance standards that the thermoplastic road marking must meet in use.

THERMOPластIC ROAD-MARKING MATERIAL
Thermoplastic road-marking material is a mixture of glass beads, binder, pigment and filler material. Dry thermoplastic compound is generally heated in a statically controlled pre-heater or boiler to a temperature of 220°C and agitated continuously until a homogenised liquid
is achieved, before transferring it to an application vehicle (AASHTO 2016). The hot, melted liquid is applied onto the road surface with drop-on glass beads added on top to produce high initial R₁ as may be stipulated in the specifications of the road authority. The material usually dries within a short space of time.

**Glass beads**

Road markings must be applied with glass beads on roads where R₁ is required. The retroreflection of road markings in wet or rainy conditions is much lower than in dry conditions, and as such the road markings should be enhanced with special properties to improve the R₁ (BS EN 2007).

Drop-on glass beads are applied to the surface of both thermoplastic markings and liquid paint markings through a bead dispenser which should be an integral part of any paint sprayer. Glass beads are fed onto the paint by means of a mechanical process or by gravity. Thermoplastic road markings also contain intermix beads which become exposed as the surface of the thermoplastic wears off, thereby producing R₁ throughout most of its service life. The added advantage of glass beads is that it provides skid resistance. The important properties of glass beads to be analysed in order for the beads to perform optimally include gradation, refractive index, roundness and clarity, and coating (Gates et al 2003; Miglets et al 1994).

**Binder**

Binder (resin) is the constituent that provides adhesion to the road surface and the other constituents (the pigments, fillers and glass beads (SS 2002a)). An effective road marking system requires not only quality glass beads, but also a quality binder. If either part of the road marking system is not good, or they are not installed properly, then the road marking system will not perform as well as it could.

**Pigment**

Pigment is a fine powder added primarily to impart colour and opacity to the mixture. Titanium dioxide (TiO₂) is a common reflective pigment used in white road markings, and higher levels in R₂ can be achieved by adding more TiO₂ (Smadi et al 2013). Too much TiO₂ used in yellow road markings may result in the road markings appearing a lighter yellow, and this is usually observed on road markings where a high initial R₂ is specified.

**Filler**

Filler is a powder added to assist the dispersion of the pigment, thereby providing colour uniformity throughout the mix (SS 2002a). Fillers are made up of a mixture of calcium carbonate, sand and other inert materials to impart body to the mixture. The filler is also an important constituent to ensure that R₁ is achieved, as well as for the daylight appearance of the road markings.

**APPLICATION OF THERMOPLASTIC ROAD-MARKING MATERIALS**

Thermoplastic road-marking materials can be applied in either a liquid or a solid form. Liquid thermoplastic is applied in three ways, namely by spraying, extruding or by screed (TMRS 2012). Thermoplastic in a solid form is commonly referred to as preformed thermoplastic.

**Spray thermoplastic**

Spray thermoplastic is applied by mobile equipment at a rapid application rate and dries almost instantly. This results in the requirement for minimal traffic accommodation and shorter periods of road closures, which reduce travel time delays. Spray thermoplastic is usually applied at a thickness of 1.2 mm in South Africa, while in most European countries the minimum application thickness is 1.5 mm, and 2 mm in the US. In Australia spray thermoplastic is applied at 2 mm (TMRS 2012). One of the reasons for thermoplastic road marking being applied thicker in Europe and the US is due to the scraping away of some of the thermoplastic under the action of the snowplough where snow has settled on the road. The other reason is that the US, Australia and Europe consider road safety as a high priority and allocate sufficient budgets for road-marking maintenance.

**Extruded thermoplastic**

Extruded thermoplastic is also applied by mobile equipment, but the mix design of the material is different and the viscosity is also higher than spray thermoplastic. The applied product takes a longer time to dry than spray plastic, as it is usually applied at a thickness of 3 mm in South Africa. In Singapore extruded thermoplastic is applied between 2.5 mm and 3 mm, while in Europe and the US it is applied up to 5 mm and 4.8 mm respectively (SS 2002b; AASHTO 2016).

**Screed thermoplastic**

Screed thermoplastic is generally applied by manual hand-push equipment for small road markings, such as transverse road markings, arrows, words and symbols. Thermoplastic that is required to be applied thicker than 2 mm for small markings is usually applied by the screed method.

**Preformed thermoplastic**

Preformed thermoplastic which is manufactured in sheets (commonly 600 mm x 900 mm) will have to be cut up into the road marking size required before application (TMRS 2012). The road surface must be cleaned and then the cut-up preformed thermoplastic is placed in its final position and heated with the flame of a blowtorch to between 150°C and 180°C (TMRS 2012). It is usually manufactured to a thickness of 3 mm, and the dry film thickness is approximately 2 mm after application onto the road surface. It can be seen that, due to the manufactured sheet size, preformed thermoplastic will be more conducive as road marking arrows and symbols. It is also an ideal product to be used as Control of Speed by Illusion (COSBI) lines for traffic calming purposes.

**METHODOLOGY**

Road trials of permanent road markings should be conducted over the full climatic cycles of at least one year (BS EN 2011; SANS 2006a; SANS 2006b). The SABS conducts road trials only on transversely applied test markings (SANS 2006a; SANS 2006b). According to BS EN (2011), the test markings are applied transversely and longitudinally on a flat asphalt road which has been established for at least a year.

**RESEARCH DESIGN**

The following activities were undertaken as part of this study:

- Nine roads were selected, which included asphalt roads and chip seal roads, both new and long established. The roads were categorised into “clean asphalt surfaced roads” (roads that are self-cleansing, for example when it rains and sand from the edge of the road is washed onto the road, the sand will flow into the stormwater system due to the effective gradient and efficient stormwater system), “dirty asphalt surfaced roads” (sub-standard roads resulting in sand remaining on the roads due to...
the ineffective gradient and also due to a lack of stormwater systems), "clean sealed roads" and "dirty sealed roads".

- A link of the road, as can be seen in Figure 1, was selected, and at least two blocks of transverse markings were applied, depending on the length of the link. A set of transverse markings comprised various types of road markings, namely waterborne road-marking paints, cold plastic and thermoplastic road-marking materials.

- A block of transverse test road markings was made up of three sections of various white and yellow road-marking paints and materials, as can be seen in Figure 2. Each road marking was applied to a width of 150 mm, as recommended in SANS (2006a) with a gap of 200 mm over the width of a lane selected.

- The positions of the longitudinal test road markings were also broken up into blocks, similar to that of the transverse test road markings.

- The test road markings were applied on clean, dry surfaces and the temperature of the road surface was greater than 10°C, as recommended in COLTO (1998).

- The test road markings were applied on the road surface as follows: waterborne road-marking paints (0.42 l/m²), cold plastic (1 mm), and thermoplastic (1.2 mm and 3 mm). The road-marking paints and materials were applied on test plates at the same time when applying the test road markings on the road surface to ensure that the correct application rates were achieved.

- Initially five service providers agreed to provide road-marking paints and road-marking materials. At the time of applying the road-marking paints and materials, only four service providers kept to the arrangements. The acronym SP1 means service provider one, SP2 means service provider two, and so on.

- The traffic counting system was set up such that the traffic counts were obtained from the lane where the transverse test road markings were applied. The counts were obtained over 14 consecutive days during a period while schools were operational, and converted into Annual Average Daily Traffic (AADT).

The following road-marking paints and materials (both white and yellow) were applied on the test sites:

- Waterborne road-marking paint which conformed to SANS (2006b)
■ Imported waterborne road-marking paints
■ Cold plastic road-marking material blended in South Africa
■ Imported cold plastic road-marking material
■ Thermoplastic road-marking material blended in South Africa
■ Imported thermoplastic road-marking material.

DATA ANALYSIS AND FINDINGS

According to the SADC RTSM (SADC 1997), the minimum $R_L$ for used white and yellow road markings must be 100 mcd/m²/lx and 70 mcd/m²/lx respectively. These values were used to determine the $R_L$ service life of the test road markings in this study. There is no standard published by the SABS with regard to $Q_d$ on road markings, hence the BS EN (2007) specification was used to analyse this parameter. According to BS EN (2007), the minimum $Q_d$ for white and yellow road markings must be 100 mcd/m²/lx and 80 mcd/m²/lx respectively. These values were used to determine the $Q_d$ service life of the test road markings in this study.

There is a possible error with the colour coordinates indicated in Table 7.2 of the SADC RTSM (SADC 1997), as the coordinates indicated for the yellow road-marking colour specification do not form a logical envelope when plotted on the chromaticity diagram. Therefore the BS EN (2007) specification was used to analyse the colour parameter.

For the colour test, the calculated values of the actual measurements of the test road markings were plotted on the graph to determine if the white and yellow test road markings conformed to the colour range.

According to Table 7.2 of the SADC RTSM (SADC 1997), the minimum skid resistance of white and yellow road markings must be 50 Skid Resistance Test (SRT) units.

Although the markings were analysed in the unwashed and washed condition, the service lives of the test markings in $R_L$, $Q_d$ and colour are that of the markings in the unwashed condition, as road markings are generally viewed by road users in the dirty condition. Both unwashed and washed road marking graphs are presented to indicate the differences in the decline of $R_L$, $Q_d$ and colour.

The use of non-parametric methods was applied to compare group means of the various types of road-marking paints and materials due to the small sample size.

When comparing between two groups, Friedman’s Chi Square Test was used. Descriptive statistics were used to indicate the mean, Standard Deviation (SD) and Coefficient of Variation (CV).

EXAMINATION OF THE TEST ROAD MARKINGS

Four sets of $R_L$, $Q_d$ and colour measurements were conducted over at least one year on the applied test road markings. The first set of measurements on $R_L$ and $Q_d$ was conducted within a month of application of the test road markings, and the remaining three sets of measurements were conducted more or less evenly, depending on the availability of the testing equipment and weather conditions. The first set of colour measurements was conducted approximately four months after the application of the test markings, due to the unavailability of the machine. The measurements were conducted during the day under appropriate traffic accommodation according to the SARTSM (1999). Only one set of skid-resistance measurements was conducted on the test plates in a laboratory. White and yellow test markings were applied on nine different roads, but the detailed analysis of the long-established clean asphalt road (Middel Street in the City of Tshwane (Figure 3)) is presented in this paper.

Measuring retroreflectivity

The LTL-X retroreflectometer (Figure 4) was used to measure $R_L$. Nine measurements were conducted on each test road-marking line applied. Three measurements each
were conducted in the region of the left and right wheel part of each transverse test road-marking line painted, and three measurements were conducted towards the centre of each of the transverse road-marking lines painted. Two sets of measurements were conducted during the same day on transverse lines. The first set of measurements was conducted on the test road markings as found on that day, and the second set of measurements was conducted after washing the test road markings with liquid soap added into clean drinkable water with brooms and left to dry. Nine measurements were conducted over the length of the white and yellow longitudinal test road markings. The measurement points were distributed more or less evenly over the length of the longitudinally applied test road markings. The longitudinal lines were only measured in the dirty condition, as the washing of only the transverse markings was sufficient to identify if there were significant differences in the various parameters in the “dirty” and “washed” condition. The RL value was established as the average of a number of measurements conducted with shifts of the instrument in steps along the marking.

Measuring luminance
The LTL-XL (Figure 5) was used to measure Qd. The number of Qd measurements conducted was equal to the number of RL measurements conducted in the dirty and washed condition. Similar to RL, the Qd value was established as the average of a number of measurements conducted with shifts of the instrument in steps along the marking.

Measuring colour
An X-Rite Model SP60 colour machine (Figure 6) was used to conduct the measurements of the test road markings. When conducting measurements with this equipment, it was crucial to use consistent lighting conditions to ensure accurate and comparable results. The colour machine was calibrated according to the manufacturer’s instructions to ensure that the measurements were reliable.

Figure 5 LTL-XL retroreflectometer
Figure 6 X-Rite Model SP60 colour machine
Figure 7 Skid-resistance tester
Figure 8 Unwashed white transverse markings in retroreflectivity on long-established clean asphalt road

<table>
<thead>
<tr>
<th>Measurement dates</th>
<th>Waterborne (SP2)</th>
<th>1 mm cold plastic (SP1)</th>
<th>1.2 mm thermoplastic (SP2)</th>
<th>Waterborne (SP1)</th>
<th>1 mm cold plastic (SP4)</th>
<th>1.2 mm thermoplastic (SP1)</th>
<th>Waterborne (SP3)</th>
<th>3 mm screed (SP1)</th>
</tr>
</thead>
</table>
machine, the tristimuli values of X, Y and Z are obtained and converted into chromaticity coordinates x and y which are plotted on the chromaticity diagram. According to the X-Rite manual, the formulae to calculate x and y are as follows:

\[ x = \frac{X}{X + Y + Z} \]
\[ y = \frac{Y}{X + Y + Z} \]

### Measuring skid resistance

A British Portable Pendulum skid resistance tester (Figure 7) was used to measure the skid resistance of the test markings applied on the test plates. A pendulum fitted with a spring-loaded slider at its free end was released from a fixed position, and the frictional energy loss caused by the dragging motion of the rubber rear edge of the slider over the markings was measured and expressed in SRT units. The

![](image)

**Figure 9** Washed white transverse markings in retroreflectivity on long-established clean asphalt road

<table>
<thead>
<tr>
<th>Type of road-marking paint / material</th>
<th>23 July 2013</th>
<th>5 November 2013</th>
<th>20 March 2014</th>
<th>29 July 2014</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Unwashed</td>
<td>Washed</td>
<td>Unwashed</td>
<td>Washed</td>
</tr>
<tr>
<td>Waterborne (SP2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean (mcd/m²lx)</td>
<td>215.2</td>
<td>196.3</td>
<td>175.3</td>
<td>151.8</td>
</tr>
<tr>
<td>N</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
</tr>
<tr>
<td>SD</td>
<td>37.6</td>
<td>35.7</td>
<td>28.5</td>
<td>20.8</td>
</tr>
<tr>
<td>CV</td>
<td>0.18</td>
<td>0.18</td>
<td>0.16</td>
<td>0.12</td>
</tr>
<tr>
<td>Waterborne (SP1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean (mcd/m²lx)</td>
<td>254.2</td>
<td>222.6</td>
<td>203.9</td>
<td>168.1</td>
</tr>
<tr>
<td>N</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
</tr>
<tr>
<td>SD</td>
<td>46.6</td>
<td>28.7</td>
<td>31.0</td>
<td>27.4</td>
</tr>
<tr>
<td>CV</td>
<td>0.18</td>
<td>0.13</td>
<td>0.15</td>
<td>0.16</td>
</tr>
<tr>
<td>Waterborne (SP3)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean (mcd/m²lx)</td>
<td>220.5</td>
<td>202.3</td>
<td>170.9</td>
<td>147.2</td>
</tr>
<tr>
<td>N</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
</tr>
<tr>
<td>SD</td>
<td>31.0</td>
<td>21.6</td>
<td>19.2</td>
<td>22.7</td>
</tr>
<tr>
<td>CV</td>
<td>0.14</td>
<td>0.11</td>
<td>0.11</td>
<td>0.15</td>
</tr>
<tr>
<td>1.2 mm Thermoplastic (SP2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean (mcd/m²lx)</td>
<td>167.1</td>
<td>159.7</td>
<td>130.7</td>
<td>139.4</td>
</tr>
<tr>
<td>N</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
</tr>
<tr>
<td>SD</td>
<td>18.3</td>
<td>12.3</td>
<td>16.5</td>
<td>12.8</td>
</tr>
<tr>
<td>CV</td>
<td>0.11</td>
<td>0.08</td>
<td>0.13</td>
<td>0.09</td>
</tr>
<tr>
<td>1.2 mm Thermoplastic (SP1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean (mcd/m²lx)</td>
<td>352.2</td>
<td>294.5</td>
<td>285.7</td>
<td>285.9</td>
</tr>
<tr>
<td>N</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
</tr>
<tr>
<td>SD</td>
<td>57.9</td>
<td>79.7</td>
<td>53.7</td>
<td>69.4</td>
</tr>
<tr>
<td>CV</td>
<td>0.17</td>
<td>0.27</td>
<td>0.18</td>
<td>0.24</td>
</tr>
<tr>
<td>1 mm Cold plastic (SP1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean (mcd/m²lx)</td>
<td>264.2</td>
<td>266.0</td>
<td>247.6</td>
<td>256.8</td>
</tr>
<tr>
<td>N</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
</tr>
<tr>
<td>SD</td>
<td>26.4</td>
<td>30.4</td>
<td>49.4</td>
<td>19.4</td>
</tr>
<tr>
<td>CV</td>
<td>0.10</td>
<td>0.11</td>
<td>0.20</td>
<td>0.08</td>
</tr>
<tr>
<td>3 mm Screed (SP1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean (mcd/m²lx)</td>
<td>370.4</td>
<td>304.6</td>
<td>285.2</td>
<td>308.2</td>
</tr>
<tr>
<td>N</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
</tr>
<tr>
<td>SD</td>
<td>46.7</td>
<td>55.3</td>
<td>40.0</td>
<td>50.4</td>
</tr>
<tr>
<td>CV</td>
<td>0.12</td>
<td>0.18</td>
<td>0.17</td>
<td>0.06</td>
</tr>
<tr>
<td>1 mm Cold plastic (SP4)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean (mcd/m²lx)</td>
<td>194.6</td>
<td>200.0</td>
<td>122.2</td>
<td>125.1</td>
</tr>
<tr>
<td>N</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
</tr>
<tr>
<td>SD</td>
<td>49.0</td>
<td>36.4</td>
<td>37.3</td>
<td>23.6</td>
</tr>
<tr>
<td>CV</td>
<td>0.25</td>
<td>0.18</td>
<td>0.31</td>
<td>0.19</td>
</tr>
</tbody>
</table>

**Table 1** Descriptive statistics of white transverse markings in retroreflectivity on long-established clean asphalt road
test method used was in accordance with SANS 6260 (SANS 2007).

**DISCUSSION OF THE FOUR PARAMETERS**

**Retroreflectivity**

There was generally no significant increase in the $R_L$ service life of the washed test road markings, as can be seen in Table 1, and Figures 8 and 9. Some of the unwashed white test road markings had a longer $R_L$ service life than washed markings. There could be a few possible reasons for this, namely:

- The measurements of the washed markings could have been conducted before the road markings were completely dry, which possibly reduced the $R_L$ measurements after washing.
- The method of washing the markings was not very effective.
- The measurements might not have been conducted at the exact spot before washing and after washing the markings.

### Table 2 Summary of white transverse markings in luminance on long-established clean asphalt road

<table>
<thead>
<tr>
<th>Type of road-marking paint / material</th>
<th>23 July 2013</th>
<th>5 November 2013</th>
<th>20 March 2014</th>
<th>29 July 2014</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Unwashed</td>
<td>Washed</td>
<td>Unwashed</td>
<td>Washed</td>
</tr>
<tr>
<td>Waterborne (SP2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean (mcd/m²/lx)</td>
<td>141.7</td>
<td>145.4</td>
<td>128.7</td>
<td>136.5</td>
</tr>
<tr>
<td>N</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
</tr>
<tr>
<td>SD</td>
<td>11.7</td>
<td>12.5</td>
<td>9.8</td>
<td>9.1</td>
</tr>
<tr>
<td>CV</td>
<td>0.05</td>
<td>0.04</td>
<td>0.08</td>
<td>0.07</td>
</tr>
<tr>
<td>Waterborne (SP1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean (mcd/m²/lx)</td>
<td>136.6</td>
<td>141.8</td>
<td>119.5</td>
<td>125.5</td>
</tr>
<tr>
<td>N</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
</tr>
<tr>
<td>SD</td>
<td>6.2</td>
<td>6.1</td>
<td>7.2</td>
<td>6.3</td>
</tr>
<tr>
<td>CV</td>
<td>0.05</td>
<td>0.04</td>
<td>0.06</td>
<td>0.05</td>
</tr>
<tr>
<td>Waterborne (SP3)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean (mcd/m²/lx)</td>
<td>138.0</td>
<td>141.3</td>
<td>122.5</td>
<td>129.1</td>
</tr>
<tr>
<td>N</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
</tr>
<tr>
<td>SD</td>
<td>7.0</td>
<td>6.8</td>
<td>8.6</td>
<td>7.8</td>
</tr>
<tr>
<td>CV</td>
<td>0.05</td>
<td>0.05</td>
<td>0.07</td>
<td>0.06</td>
</tr>
<tr>
<td>1.2 mm Thermoplastic (SP2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean (mcd/m²/lx)</td>
<td>144.2</td>
<td>147.1</td>
<td>134.0</td>
<td>140.0</td>
</tr>
<tr>
<td>N</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
</tr>
<tr>
<td>SD</td>
<td>9.4</td>
<td>9.5</td>
<td>12.3</td>
<td>10.1</td>
</tr>
<tr>
<td>CV</td>
<td>0.07</td>
<td>0.07</td>
<td>0.09</td>
<td>0.07</td>
</tr>
<tr>
<td>1.2 mm Thermoplastic (SP1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean (mcd/m²/lx)</td>
<td>153.7</td>
<td>158.1</td>
<td>143.8</td>
<td>152.3</td>
</tr>
<tr>
<td>N</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
</tr>
<tr>
<td>SD</td>
<td>9.9</td>
<td>8.7</td>
<td>10.9</td>
<td>9.6</td>
</tr>
<tr>
<td>CV</td>
<td>0.07</td>
<td>0.07</td>
<td>0.09</td>
<td>0.07</td>
</tr>
<tr>
<td>1 mm Cold plastic (SP1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean (mcd/m²/lx)</td>
<td>139.4</td>
<td>143.3</td>
<td>134.3</td>
<td>139.8</td>
</tr>
<tr>
<td>N</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
</tr>
<tr>
<td>SD</td>
<td>5.0</td>
<td>3.2</td>
<td>7.6</td>
<td>7.3</td>
</tr>
<tr>
<td>CV</td>
<td>0.04</td>
<td>0.02</td>
<td>0.06</td>
<td>0.05</td>
</tr>
<tr>
<td>3 mm Screed (SP1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean (mcd/m²/lx)</td>
<td>155.5</td>
<td>160.0</td>
<td>144.8</td>
<td>151.2</td>
</tr>
<tr>
<td>N</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
</tr>
<tr>
<td>SD</td>
<td>12.2</td>
<td>9.6</td>
<td>15.1</td>
<td>15.2</td>
</tr>
<tr>
<td>CV</td>
<td>0.08</td>
<td>0.06</td>
<td>0.06</td>
<td>0.05</td>
</tr>
<tr>
<td>1 mm Cold plastic (SP4)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean (mcd/m²/lx)</td>
<td>139.7</td>
<td>144.7</td>
<td>125.9</td>
<td>131.3</td>
</tr>
<tr>
<td>N</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
</tr>
<tr>
<td>SD</td>
<td>11.0</td>
<td>10.5</td>
<td>6.9</td>
<td>6.5</td>
</tr>
<tr>
<td>CV</td>
<td>0.08</td>
<td>0.07</td>
<td>0.06</td>
<td>0.05</td>
</tr>
</tbody>
</table>

![Figure 10 Unwashed white transverse markings in luminance on long-established clean asphalt road](image-url)
A certain amount of dirt on road markings do not make a significant difference in the R_L.

Possible removal of glass beads could have happened while washing the markings with hard brooms.

### Luminance

There was generally an increase in the Qd measurement values of the washed test road markings, as can be seen in Table 2, and Figures 10 and 11. As a result of this, some of the washed markings produced a longer Qd service than the unwashed markings. The possible reasons for the increase in the Qd after washing are:

- The fines from the sand particles which had been stuck to the test road markings were removed to a certain extent, exposing more of the actual area of the road marking.
- Some of the oils and exhaust fumes were removed from the test road markings.

### Colour

Higher percentages of the later measurements in some instances fell into the compliance regions, as can be seen in Table 3, and Figures 12 and 13. The possible reasons are:

- The measurements might not have been conducted on the exact spot, as the machine measures only approximately 1 cm in diameter.
- The rains might have cleaned the markings.

The washing of road markings generally did not positively contribute to colour compliance. Although the colour of road markings complied in a few instances with the specification after being washed, road markings fell out of compliance in other cases.

### Table 3 Summary of the white markings colour analysis on long-established clean asphalt road

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Falling out of region (%)</td>
<td>Falling within region (%)</td>
<td>Falling out of region (%)</td>
<td>Falling within region (%)</td>
<td>Falling out of region (%)</td>
</tr>
<tr>
<td>Unwashed</td>
<td>Waterborne (SP1)</td>
<td>0.0</td>
<td>100.0</td>
<td>33.3</td>
<td>66.7</td>
</tr>
<tr>
<td></td>
<td>Waterborne (SP2)</td>
<td>66.7</td>
<td>33.3</td>
<td>0.0</td>
<td>100.0</td>
</tr>
<tr>
<td></td>
<td>Waterborne (SP3)</td>
<td>33.3</td>
<td>66.7</td>
<td>0.0</td>
<td>100.0</td>
</tr>
<tr>
<td></td>
<td>1 mm Cold plastic (SP1)</td>
<td>66.7</td>
<td>33.3</td>
<td>0.0</td>
<td>100.0</td>
</tr>
<tr>
<td></td>
<td>1 mm Cold plastic (SP4)</td>
<td>33.3</td>
<td>66.7</td>
<td>0.0</td>
<td>100.0</td>
</tr>
<tr>
<td></td>
<td>1.2 mm Thermoplastic (SP1)</td>
<td>66.7</td>
<td>33.3</td>
<td>66.7</td>
<td>33.3</td>
</tr>
<tr>
<td></td>
<td>1.2 mm Thermoplastic (SP2)</td>
<td>100.0</td>
<td>0.0</td>
<td>100.0</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>3 mm Screed (SP1)</td>
<td>33.3</td>
<td>66.7</td>
<td>66.7</td>
<td>33.3</td>
</tr>
<tr>
<td>Washed</td>
<td>Waterborne (SP1)</td>
<td>0.0</td>
<td>100.0</td>
<td>0.0</td>
<td>100.0</td>
</tr>
<tr>
<td></td>
<td>Waterborne (SP2)</td>
<td>0.0</td>
<td>100.0</td>
<td>0.0</td>
<td>100.0</td>
</tr>
<tr>
<td></td>
<td>Waterborne (SP3)</td>
<td>33.3</td>
<td>66.7</td>
<td>0.0</td>
<td>100.0</td>
</tr>
<tr>
<td></td>
<td>1 mm Cold plastic (SP1)</td>
<td>66.7</td>
<td>33.3</td>
<td>0.0</td>
<td>100.0</td>
</tr>
<tr>
<td></td>
<td>1 mm Cold plastic (SP4)</td>
<td>0.0</td>
<td>100.0</td>
<td>66.7</td>
<td>33.3</td>
</tr>
<tr>
<td></td>
<td>1.2 mm Thermoplastic (SP1)</td>
<td>33.3</td>
<td>66.7</td>
<td>33.3</td>
<td>66.7</td>
</tr>
<tr>
<td></td>
<td>1.2 mm Thermoplastic (SP2)</td>
<td>100.0</td>
<td>0.0</td>
<td>100.0</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>3 mm Screed (SP1)</td>
<td>66.7</td>
<td>33.3</td>
<td>100.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>
markings are generally not washed by the road authorities, and also the rainy season is usually of limited duration.

Skid resistance
The skid resistance of the test road markings was conducted on test plates in the laboratory, which might not be a true reflection of the road markings on the road surface. It is likely that the skid resistance of the road markings applied on the road surface will be higher due to the roughness of the road surface, especially road markings applied onto chip seal surfaces. The skid resistance of road markings is important, especially for motor cyclists and biker groups. It is often dangerous for motor cyclists if the skid resistance is not up to standard. The fact that there is no overlap between the 95% CIs of the first and last measurements, as can be seen in Figure 14, is an indication that the difference between them is statistically significant. Friedman’s Chi Square Test revealed that there is at least one pair of measurements over time that is significantly different within each of the different road-marking materials and paints, and referring to Figure 14 it seems as though these differences exist at least between the first and last repetitions.

LUMINANCE SERVICE LIFE OF WHITE TRANSVERSE MARKINGS
All the test road markings applied on the long-established clean asphalt road were higher than the minimum specification of 100 mcd/m²/lx over the four measurement periods, as can be seen in Table 2 and Figure 10. The average decline in Qd of all test markings over the year was approximately 40%, and if the decline continues at the same rate, then thermoplastic materials will produce a Qd service life of approximately two years. Although most of the test road markings applied on Middel Street complied with the specification for more than a year, the Rₙ mean of 1.2 mm thermoplastic (SP1) and 3 mm thermoplastic (SP1) were in the region of 200 mcd/m²/lx after a year. This is an indication that thermoplastic road marking materials are much more durable on clean asphalt roads, compared with waterborne road marking paints, which seem to start getting closer to the minimum Rₙ of 100 mcd/m²/lx after a year.

The cold plastic and thermoplastic road marking materials generally declined over the year. However, it can be seen in Table 1 and Figure 8 that the Rₙ mean of some of the measurements conducted later were higher than the Rₙ mean values of the measurements conducted earlier. For example, the Rₙ mean of unwashed 1.2 mm thermoplastic (SP2) was 130.7 mcd/m²/lx on 5 November 2013, and the same marking was 139.4 mcd/m²/lx on 20 March 2014, which is 4.5 months later. The intermix beads in the plastic road marking materials became exposed over time as a result of the wheels of the vehicles passing over the road markings, which contributed positively to the Rₙ.

Waterborne (SP1, SP2 and SP3), 1 mm cold plastic (SP1, SP2 and SP3) and 1 mm thermoplastic (SP1, SP1 and SP2) and 3 mm thermoplastic (SP1) produced Rₙ service lives of more than 12 months. The average decline in Rₙ of all test markings over the year was approximately 40%, and if the decline continues at the same rate, then thermoplastic materials will produce a Rₙ service life of approximately two years. Although most of the test road markings applied on Middel Street complied with the specification for more than a year, the Rₙ means of 1.2 mm thermoplastic (SP1) and 3 mm thermoplastic (SP1) were in the region of 200 mcd/m²/lx after a year. This is an indication that thermoplastic road marking materials are much more durable on clean asphalt roads, compared with waterborne road marking paints, which seem to start getting closer to the minimum Rₙ of 100 mcd/m²/lx after a year.

The fact that there is no overlap between the 95% CIs of the first and last measurements, as can be seen in Figure 14, is an indication that the difference between them is statistically significant. Friedman’s Chi Square Test revealed that there is at least one pair of measurements over time that is significantly different within each of the different road-marking paints and materials, and referring to Figure 14 it seems as though these differences exist at least between the first and last repetitions.
Figure 14 95% Confidence interval in retroreflectivity of unwashed white transverse markings on clean established asphalt road

Figure 15 95% Confidence interval in luminance of unwashed white transverse markings on clean established asphalt road
<table>
<thead>
<tr>
<th>Road marking colour</th>
<th>Type of road surface</th>
<th>Established period</th>
<th>Condition of road</th>
<th>Class of road</th>
<th>AUD</th>
<th>Transverse / Longitudinal</th>
<th>Transverse / Longitudinal</th>
<th>Rd service life (months)</th>
<th>Qd service life (months)</th>
<th>Colour compliance</th>
<th>Initial skid resistance compliance</th>
</tr>
</thead>
<tbody>
<tr>
<td>White</td>
<td>Chip seal</td>
<td>Newly</td>
<td>Dirty</td>
<td>3</td>
<td></td>
<td>4 to 9</td>
<td>4 to 9</td>
<td>1 to 4</td>
<td>1 to 4</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Longitudinal</td>
<td>Longitudinal</td>
<td>4 to 9</td>
<td>4 to 9</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Yellow</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Transverse</td>
<td>Transverse</td>
<td>1 to 4</td>
<td>1 to 4</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Longitudinal</td>
<td>Longitudinal</td>
<td>9 to 12</td>
<td>9 to 12</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>White</td>
<td>Chip seal</td>
<td>Long</td>
<td>Dirty</td>
<td>4</td>
<td></td>
<td>Light = 2454</td>
<td>Light = 2454</td>
<td>1 to 5</td>
<td>1 to 5</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Heavy = 109</td>
<td>Heavy = 255</td>
<td>4 to 9</td>
<td>4 to 9</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Yellow</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Transverse</td>
<td>Transverse</td>
<td>1 to 5</td>
<td>1 to 5</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Longitudinal</td>
<td>Longitudinal</td>
<td>9 to 12</td>
<td>9 to 12</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>White</td>
<td>Chip seal</td>
<td>Newly</td>
<td>Clean</td>
<td>2</td>
<td></td>
<td>Light = 7753</td>
<td>Light = 7753</td>
<td>&gt;12</td>
<td>&gt;12</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Heavy = 284</td>
<td>Heavy = 284</td>
<td>4 to 8</td>
<td>4 to 8</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>Yellow</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Transverse</td>
<td>Transverse</td>
<td>&gt;12</td>
<td>&gt;12</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Longitudinal</td>
<td>Longitudinal</td>
<td>8 to 12</td>
<td>8 to 12</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>White</td>
<td>Chip seal</td>
<td>Long</td>
<td>Clean</td>
<td>2</td>
<td></td>
<td>Light = 5508</td>
<td>Light = 5508</td>
<td>&gt;12</td>
<td>&gt;12</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Heavy = 739</td>
<td>Heavy = 739</td>
<td>4 to 8</td>
<td>4 to 8</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Yellow</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Transverse</td>
<td>Transverse</td>
<td>&gt;12</td>
<td>&gt;12</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Longitudinal</td>
<td>Longitudinal</td>
<td>8 to 12</td>
<td>8 to 12</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>White</td>
<td>Asphalt</td>
<td>Newly</td>
<td>Dirty</td>
<td>3</td>
<td></td>
<td>Light = 7822</td>
<td>Light = 7822</td>
<td>4 to 7</td>
<td>4 to 7</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Heavy = 434</td>
<td>Heavy = 434</td>
<td>1 to 4</td>
<td>1 to 4</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Yellow</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Transverse</td>
<td>Transverse</td>
<td>&gt;12</td>
<td>&gt;12</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Longitudinal</td>
<td>Longitudinal</td>
<td>7 to 10</td>
<td>7 to 10</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>White</td>
<td>Asphalt</td>
<td>Long</td>
<td>Dirty</td>
<td>4</td>
<td></td>
<td>Light = 1732</td>
<td>Light = 1732</td>
<td>1 to 6</td>
<td>1 to 6</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Heavy = 213</td>
<td>Heavy = 213</td>
<td>6 to 9</td>
<td>6 to 9</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Yellow</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Transverse</td>
<td>Transverse</td>
<td>1 to 6</td>
<td>1 to 6</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Longitudinal</td>
<td>Longitudinal</td>
<td>&gt;12</td>
<td>&gt;12</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>White</td>
<td>Asphalt</td>
<td>Newly</td>
<td>Clean</td>
<td>3</td>
<td></td>
<td>Light = 3981</td>
<td>Light = 3981</td>
<td>&gt;13</td>
<td>&gt;13</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Heavy = 629</td>
<td>Heavy = 629</td>
<td>9 to 13</td>
<td>9 to 13</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Yellow</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Transverse</td>
<td>Transverse</td>
<td>&gt;13</td>
<td>&gt;13</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Longitudinal</td>
<td>Longitudinal</td>
<td>&gt;13</td>
<td>&gt;13</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>White</td>
<td>Asphalt</td>
<td>Long</td>
<td>Clean</td>
<td>3</td>
<td></td>
<td>Light = 5547</td>
<td>Light = 5547</td>
<td>&gt;12</td>
<td>&gt;12</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Heavy = 273</td>
<td>Heavy = 273</td>
<td>9 to 12</td>
<td>9 to 12</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Yellow</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Transverse</td>
<td>Transverse</td>
<td>&gt;12</td>
<td>&gt;12</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Longitudinal</td>
<td>Longitudinal</td>
<td>&gt;12</td>
<td>&gt;12</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>White</td>
<td>Asphalt</td>
<td>Long</td>
<td>Clean (CBD)</td>
<td>3</td>
<td></td>
<td>Light = 1764</td>
<td>Light = 1764</td>
<td>7 to 11</td>
<td>7 to 11</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Heavy = 548</td>
<td>Heavy = 548</td>
<td>4 to 7</td>
<td>4 to 7</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Yellow</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Transverse</td>
<td>Transverse</td>
<td>4 to 7</td>
<td>4 to 7</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Longitudinal</td>
<td>Longitudinal</td>
<td>7 to 11</td>
<td>7 to 11</td>
<td>Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>

Class 2 – Metropolitan Distributor, Class 3 – District Distributor, Class 4 – Collector
is an indication that the difference between them is statistically significant. Friedman’s Chi Square Test revealed that there is at least one pair of measurements over time that is significantly different within each of the different road marking paints and materials, and referring to Figure 15 it seems as though these differences are at least between the first and last repetitions.

**SKID-RESISTANCE ANALYSIS OF MARKINGS ON THE TEST PLATES**

Only white waterborne (SP3), 1 mm cold plastic (SP1 and SP4), white 1.2 mm thermoplastic (SP1) and yellow 1.2 mm thermoplastic (SP1) complied with the minimum specification of 50 SRT units. All other test markings were between 35 and 45 SRT units. It is likely that the skid resistance of road markings will be higher on the road surface due to the combined effect of the road surface and the markings. Road-marking applicators need to consider increasing the amount of antiskid aggregates to improve the skid resistance of the markings.

**COLOUR ANALYSIS OF MARKINGS**

The white test road markings generally complied with the BS EN (2007) specification except for 1.2 mm thermoplastic (SP2) after one-and-a-half years of application. Figures 12 and 13 indicate the graphical positions of the white test road markings on the chromaticity diagrams.

**PERFORMANCE OF THERMOPLASTIC ROAD MARKING MATERIAL**

Based on the data obtained from the study, the performance of thermoplastic road-marking materials in R_L, Qd, colour and skid resistance is summarised in Table 4. It can be seen that even though the traffic volumes were higher on some of the clean roads than on the dirty roads, the service life of thermoplastic was higher on the clean roads than on the dirty roads. For example, on the long-established clean asphalt road, 1.2 mm thermoplastic produced an R_L service life of more than 12 months while it produced between one and six months on the long-established dirty asphalt road, as can be seen in Table 4. Although thermoplastic road-marking materials are generally known to produce high R_L over long periods of time, the outcome of the study indicates that R_L is not maintained for long periods on dirty type roads. It can be seen that road authorities are applying thermoplastic road marking materials that do not comply with the yellow colour specification. It can be further seen that the skid resistance of 3 mm thermoplastic road-marking material did not comply with the specification.

**CONCLUSIONS**

Based on the data obtained from the study, the following conclusions are drawn:

- The R_L and Qd service lives of various road-marking paints and road marking materials on asphalt and chip seal road surfaces were determined as between 1 and 48 months and 1 and 30 months respectively. The range in service lives were wide, due to the low service lives of waterborne road-marking paints on dirty roads and high service lives of plastic road-marking materials on clean roads.

- There was no significant increase in R_L or colour compliance of the washed road markings, while there was generally an increase in Qd after washing the test markings.

- White road markings generally complied with the colour specification, while yellow road markings did not comply with the specification.

- The initial skid resistance of white and yellow 1.2 mm thermoplastic complied with the specification, while all other road markings did not comply with the specification.

As R_L is the most critical parameter in road markings, the type of road-marking selection should be mainly based on the R_L service life, but due consideration should be given to the other three parameters, namely the luminance, colour and skid resistance. For example, specifying a very high R_L on yellow road markings may result in the service provider applying lighter shade yellow markings, which may cause confusion at night, because the markings will tend to look white in colour. There can be a certain degree of compromise, as all parameters cannot be easily achieved together, as indicated in BS EN (2007). Retroreflectivity measurements should not simply be rejected, especially if the values are close to the specification on thermoplastic road-marking material, since the values can increase when the road-marking material is worn away by the action of the tyres. More test points should be considered, or measurements should be conducted a few days later.

Although thermoplastic road-marking materials generally produced the longest service life, especially in respect of R_L, it might not be cost-effective to apply them on roads where they will have similar service lives to those of cheaper road-marking paints, such as waterborne. After classifying the roads into the dirty and clean categories, and using the tendered rates, a more realistic road-marking budget can be requested. The outcome of the results on the applied test road markings was not affected by extraordinary weather conditions, as the rainfall and temperature were similar to those of previous years.

The SABS should seriously consider continuing the work it started in developing a national thermoplastic road-marking standard, which will offer much guidance to all stakeholders involved.

**ACKNOWLEDGEMENT**

The authors would like to thank the City of Tshwane for the funding made available to execute this research project.

**DISCLAIMER**

This paper reflects the views of the authors, who are responsible for the facts and the accuracy of the data presented herein.

**REFERENCES**


COLTO (Committee of Land Transport Officials) 1998. Standard Specifications for Road and Bridge Works for State Road Authorities. Halfway House, South Africa: SAICE.

Modelling manpower and equipment productivity in tall residential building projects in developing countries

M K Parthasarathy, R Murugasan, Ramya Vasan

This study is aimed at developing productivity models based on the combined usage of manpower and equipment resources in construction for tall residential building projects in India. Data was collected from 52 tall residential building projects in different locations for a consecutive period of 18 months for the five basic activities of construction: concreting, reinforcement, formwork, blockwork and plastering. Multiple linear regression analysis was used to develop 15 models – three models each in different conditions for the five basic construction activities for manpower and equipment productivity, and were validated with independent field data. In addition, the factors affecting productivity of manpower and equipment were analysed through a qualitative study by collecting and examining responses to questionnaires from 96 respondents involved in the 52 projects. The study found that improper planning of work was the most important factor affecting the productivity in tall residential building projects. The models developed in the study, and the analysis of factors affecting productivity will be useful to cost engineers and project managers to estimate the productivity of resources in tall residential building projects in India and other developing countries where similar conditions prevail.

INTRODUCTION

The construction industry plays a vital role in the development of India, one of the steadiest and fastest growing countries in the world (Economic Report of the President 2016). Being the second largest contributor to Gross Domestic Product (GDP) after agriculture, construction employs a workforce of nearly 35 million in addition to a variety of equipment. In fact, its market size is worth about $126 billion, according to the Department of Industrial Policy and Promotion, Government of India in 2016. A Financial Express report predicted that more than 40% of the population would be residing in urban areas by 2030, and suggested that approximately 400 million people would be migrating to cities in a period of 15 years from 2016 to 2030 (Gulati 2016). Consequently, urbanisation and increased demand for space in developing countries like India have led to large-scale construction of tall residential buildings, and impacted the basic activities of construction like concreting, reinforcement, formwork, blockwork and plastering. In developing countries these basic activities are executed with a combination of manpower and equipment resources, due to the easy availability of manpower and the high cost of equipment. Such a combined approach to construction requires a scientific approach to the planning and execution of projects. This problem can be addressed primarily by tall building residential projects, a solution of great potential for the development of cities and urban spaces worldwide (Elbakheit 2012).

As in other developing countries, India is faced with the challenge of affordable and comfortable tall building construction projects to meet the high demand for space in urban localities. Studies on productivity in construction projects in the Indian context by Attar et al (2014), Patil (2015), Santosh and Apte (2014), and Shashank et al (2014) highlighted the productivity measurement, as well as the methods. They also discussed the large quantum of manpower and equipment resources involved, but did not suggest any models which could be used for tall building projects. On the other hand, the models developed by Antunes and Gonzalez (2015), Gundecha (2012), Jrade et al (2012), Wang (2005) and others in other countries for construction productivity mainly focused on work methods, introduction model.
The factors affecting the productivity, productive time of trade, etc. A careful study of these models indicated that they were area-specific and could not be used as prediction models for construction in developing countries, the major reason being that construction in developing countries involves a combination of equipment usage and manual labour, unlike in developed countries where the activities are predominantly mechanised.

This study is therefore aimed at developing scientific models to estimate the productivity of manpower and equipment in tall residential building projects with a height of 30 m or more in India, bearing in mind the seasonal variances in monsoon and non-monsoon periods, as well as factors affecting the productivity of manpower and equipment.

**LITERATURE REVIEW**

A few studies on the modelling of manpower and equipment productivity for tall residential building projects which have direct relevance to the current study are discussed in this section.

Zayed and Halpin (2005) estimated the productivity and cost of pile construction using the regression technique in the United States. They designed 52 regression models to assess piling process productivity and cycle time. They validated the models to assure their appropriateness in the assessment process by developing several sets of charts that represented productivity, cycle times and cost. They concluded that productivity was directly related to the achievement of cycle time using piling equipment. They further observed that variation happened if the cycle time of piles on one piece of equipment was not achieved.

Al-Zwainy et al. (2013) used the Multivariable Linear Regression Technique for modelling productivity of construction, with a focus on marble finishing works in Iraq. They used parameters like age, experience, number of assisting workmen, height of the floor, size of the marble tiles, security conditions, health status of the work team, weather conditions, site condition and availability of construction materials as independent variables. They developed a model based on 100 sets of data collected in Iraq from different types of construction, such as residential, commercial and educational projects. They concluded that the size of finished marble blocks had the most significant effect on the productivity of marble finishing works for floors, while the other input variables had a moderate impact on the productivity.

Gupta and Kansal (2014) examined the factors affecting labour productivity in construction sites in India. They invited professionals such as project managers, project engineers, site engineers, architects, assistance project managers, assistance project engineers and others who worked on the project, from management to execution level, to participate in a questionnaire survey. The questionnaire had four primary groups of factors, i.e. management, technological, human/labour and external. They found that ten factors of clarification in technical specifications (labour supervision, method of construction, delay in payment, labour fatigue, lack of construction managers’ leadership, extent of variations/change in order during execution, late arrival, early quitting and frequent unscheduled breaks, labour skill, and availability of experienced labour) affected construction labour productivity in the Chambal Region.

An examination of the available literature indicated that very few studies had developed models for estimating productivity on tall building projects. It further showed that research was not carried out in the context of a combination of both manpower and equipment resources. Finally, it was observed that the research outcomes were based on area-specific studies and had no scope for application in tall residential building projects. Hence, this study explored the options of developing models for estimating productivity in tall residential building projects. In addition, it attempted to examine various factors influencing productivity when construction was carried out with a combination of manpower and equipment resources.

**OBJECTIVES OF THE STUDY**

The objectives of this research are:

- To develop prediction models for manpower and equipment productivity, using multiple linear regression for the main activities of tall residential building construction like concrete, reinforcement, formwork, blockwork and plastering, considering the resources as independent variables, and for conditions prevailing in developing countries.

- To validate the developed models by comparing theoretical and actual quantities of each item, so that the models can be used for planning and utilisation of resources in projects.

- To examine and classify the factors affecting productivity of manpower and equipment in tall residential building projects.

- To rank the factors affecting productivity in descending order of their intensity in terms of frequency indices.

**RESEARCH METHOD**

This is an empirical study of tall residential building projects in India. First, it identified the tall buildings located in different parts of India and prepared a database for the selection of projects and the collection of data. It regularly collected data pertaining to the five basic activities over a period of 18 months. Multiple linear regression analysis was used to develop manpower and equipment productivity models. Independent field data was used to validate the models developed in this study. Thereafter questionnaires were used to collect information regarding the factors affecting tall building projects from 96 respondents involved in 52 projects.

**DATA BASE DEVELOPMENT**

**Project selection criteria**

A set of criteria was employed to select the 52 tall building projects:

- The final height of the building should be more than or equal to 30 m.
- The building should be a framed Reinforced Cement Concrete (RCC) structure with column-slab design and it should not involve any composite construction.
- All the five selected basic activities should be on-going at various height levels in the project.
- The floor height in each level should be 3 m – 4 m, which is the normal floor to floor height in India.
- The project sites should be distributed throughout the country.

Based on these criteria, 52 tall residential building construction projects were chosen for the study. This in turn determined the distribution of two sets of questionnaires to 130 persons involved in different trades among consultants, execution teams, planning personnel, etc.
Data collection

Once the activities and projects had been selected, the manpower input and equipment usage for each activity were recorded on a monthly basis for every project. In the case of equipment like tower cranes and hoists, which were shared for many activities in the given project, a percentage of time allocation was considered for different activities. For example, tower crane time was distributed as 35% for concrete, 35% for reinforcement and 30% for formwork. Similarly, the distribution of time for all equipment was determined for this study. The distribution of percentages of equipment time to different activities was based on the experience of the project personnel and the allocation log sheets maintained by the plant and equipment (P&E) departments in the 52 project sites.

The heights of the building, i.e. the point from where the work commenced at the start of the month to the point to where work had progressed by the end of the month, were recorded. The mean of these two heights was considered as the height at which the activity was carried out during the month.

The range of data collected from the 52 tall building construction projects is given in Table 1.

Factor selection for factor analysis

Twenty-one factors affecting the productivity of manpower and 20 factors affecting equipment productivity were identified for this study from various sources:

- Studies available from construction industry and other projects.
- The opinion of personnel involved in the construction of tall building projects, based on their experience.
- Empirical data on losses due to productivity from a multinational organisation in the area of study.
- The opinions of contract managers while estimating the costs of tall residential building projects.

The list of factors considered for this study, with their classification in respective categories, is shown in Table 2(a). The critical literature review of the factors affecting productivity is shown in Table 2(a).
<table>
<thead>
<tr>
<th>Factor</th>
<th>Factor no</th>
<th>Affecting equipment productivity</th>
<th>Factor no</th>
<th>Affecting manpower productivity</th>
</tr>
</thead>
<tbody>
<tr>
<td>System factors</td>
<td>ES1</td>
<td>Improper access and egress</td>
<td>MS1</td>
<td>Improper access and egress</td>
</tr>
<tr>
<td></td>
<td>ES2</td>
<td>Excess travel/ lifting</td>
<td>MS2</td>
<td>Long lead</td>
</tr>
<tr>
<td></td>
<td>ES3</td>
<td>Non-payment of dues</td>
<td>MS3</td>
<td>Excess lift</td>
</tr>
<tr>
<td></td>
<td>ES4</td>
<td>Excess travel/ lifting</td>
<td>MS4</td>
<td>Lack of standard procedures</td>
</tr>
<tr>
<td>Environmental factors</td>
<td>EE1</td>
<td>Extreme weather conditions</td>
<td>ME1</td>
<td>Extreme climatic conditions</td>
</tr>
<tr>
<td></td>
<td>EE2</td>
<td>Lack of water/ hygiene</td>
<td>ME2</td>
<td>Lack of proper illumination</td>
</tr>
<tr>
<td>Resource/ equipment factors</td>
<td>ER1</td>
<td>Lack of support equipment</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>ER2</td>
<td>Non-availability of fuel</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>ER3</td>
<td>Non-availability of spares</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>ER4</td>
<td>Lead time</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>ER5</td>
<td>Delay in installing the equipment</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>ER6</td>
<td>Two or more gangs sharing a piece of equipment</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>ER7</td>
<td>Equipment breakdown</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>ER8</td>
<td>Sub-standard spares, etc</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>ER9</td>
<td>Use of high-end equipment where not required</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

Table 2(b) Summary of literature for factors affecting productivity

<table>
<thead>
<tr>
<th>S no</th>
<th>Author(s)</th>
<th>Year</th>
<th>Study method</th>
<th>Area of study</th>
<th>Number of factors</th>
<th>Major factors affecting productivity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Dai et al</td>
<td>2009</td>
<td>Questionnaire survey and regression analysis</td>
<td>USA</td>
<td>83</td>
<td>Construction equipment, materials, tools and consumables, engineering drawing management, direction and coordination, project management, training, craft worker qualification, superintendent competency, and foreman competency</td>
</tr>
<tr>
<td>2</td>
<td>Enhassi et al</td>
<td>2007</td>
<td>Questionnaire survey</td>
<td>Gaza</td>
<td>45</td>
<td>Material shortage, lack of labour experience, lack of labour surveillance, misunderstandings between labour and superintendent, and drawings and specification alteration during execution</td>
</tr>
<tr>
<td>3</td>
<td>Mistry and Bhatt</td>
<td>2013</td>
<td>Questionnaire survey</td>
<td>India</td>
<td>27</td>
<td>Delay in payments, skill of labour, clarity of technical specification, shortage of materials, and motivation of labour</td>
</tr>
<tr>
<td>4</td>
<td>Chigara and Moyo</td>
<td>2014</td>
<td>Questionnaire survey</td>
<td>Zimbabwe</td>
<td>40</td>
<td>Non-availability of materials, late payment of salaries and wages, suitability/adequacy of plant and equipment, supervisory incompetence, and lack of manpower skills</td>
</tr>
<tr>
<td>5</td>
<td>Robles et al</td>
<td>2014</td>
<td>Questionnaire survey</td>
<td>Spain</td>
<td>35</td>
<td>Shortage or late supply of materials, non-clarity of drawings and project documents, lack of clear and daily task assignment, tools or equipment shortages, and poor level of skill and experience of labourers</td>
</tr>
<tr>
<td>6</td>
<td>Shashank et al</td>
<td>2014</td>
<td>Questionnaire survey and regression analysis</td>
<td>India</td>
<td>34</td>
<td>Manpower group, managerial group, motivation group, material/equipment group, safety group, and quality group</td>
</tr>
<tr>
<td>7</td>
<td>Gupta and Kansal</td>
<td>2014</td>
<td>Questionnaire Survey</td>
<td>India</td>
<td>45</td>
<td>Clarification in technical specifications, labour supervisions, method of construction, delay in payment, labour fatigue, lack of leadership from construction managers, extent of variations/change of order during executions, late arrival, early quitting and frequent unscheduled breaks, labour skills, and availability of experienced labour</td>
</tr>
<tr>
<td>8</td>
<td>Nguyen and Nguyen</td>
<td>2013</td>
<td>Case study</td>
<td>Vietnam</td>
<td>–</td>
<td>Labour productivity of the formwork activity, and labour productivity of the rebar activity</td>
</tr>
</tbody>
</table>
manpower and equipment productivity is given in Table 2(b).

QUANTITATIVE ANALYSIS AND DEVELOPMENT OF MODELS

The data from the 52 projects was collected and sorted for each of the five basic construction activities. Data pertaining to every third month from the start date of data collection was separated as out-of-sample for the purpose of validation, and the remaining months’ data was used as sample for the formulation of the models. Models were developed in this study for estimating productivity of the five activities: concrete, reinforcement, formwork, blockwork and plastering. Further, the effect of monsoon was also studied, and models were developed for each activity in monsoon and non-monsoon periods separately. According to the Indian Meteorological Department, India receives rain from two monsoons, namely the Southwest Monsoon from the end of May to the beginning of September every year, and the Northeast Monsoon from the beginning of October to the end of December. Four categories/areas of monsoon were considered, with reference to the onset of the two monsoon periods in India:

A. Northern and western regions – June, July and August
B. Eastern region – July, August and September
C. Southeastern region – October and November
D. Southwestern region – May, June and July

The data was separated as sample for modelling and as out-of-sample for validation. Overall, the data separated for the purpose of validation was 30% of the total data set.

Concrete quantity model
In this model, concrete quantity was the dependent variable, whereas the parameters of height ($H$), man-days ($MD$), tower crane hours ($TCH$), pump hours ($CPH$), transit mixer hours ($TMH$) and batching plant hours ($BPH$) were independent variables.

The best model obtained using multiple linear regression analysis is:

$\text{Conc} = -448.751 - 1.249 H + 0.402 MD + 0.902 TCH + 1.371 CPH + 2.155 TMH + 5.735 BPH$

($R^2 = 0.953; N = 624; SE = 528.58$) (Model 1)

The plot between actual quantity (from out-of-sample data) and the derived quantity (sample data) for concrete validates the model, as shown in Figure 1. The alignment of plotted points along the line of equality indicates the robustness of the model.

The model for concrete quantity in the monsoon period is:

$\text{Conc monsoon} = -674.443 - 0.322 H + 0.328 MD + 1.127 TCH + 1.291 CPH + 2.804 TMH + 5.262 BPH$

($R^2 = 0.957; N = 136; SE = 520.34$) (Model 2)

Reinforcement quantity model
In this model, reinforcement quantity was the dependent variable, whereas the parameters of height ($H$), man-days ($MD$), tower crane hours ($TCH$), bar cutting hours ($BCH$) and bar bending hours ($BBH$) were independent variables.

The best model obtained using multiple linear regression analysis is:

$\text{Rft} = -210.221 - 0.119 H + 0.047 MD + 0.997 TCH + 0.438 BCH - 0.370 BBH$

($R^2 = 0.932; N = 624; SE = 106.69$) (Model 4)

The plot between actual quantity (from out-of-sample data) and the derived quantity (sample data) for reinforcement validates the model, as shown in Figure 2. The alignment of plotted points along the line of equality indicates the robustness of the model.

The model for reinforcement quantity in the non-monsoon period is:

$\text{Rft non-monsoon} = -266.949 - 2.229 H + 0.358 MD + 0.809 TCH + 1.382 BCH - 0.341 BBH$

($R^2 = 0.946; N = 486; SE = 569.22$) (Model 3)
The model for reinforcement quantity in the non-monsoon period is:

\[
R_{ft\ non-monsoon} = -188.669 + 0.005 \times H + 0.052 \times MD + 0.872 \times TCH + 0.391 \times BCH - 0.339 \times BBH
\]

\((R^2 = 0.931; N = 486; SE = 112.54)\) (Model 6)

An examination of the reinforcement models (Models 4, 5 and 6) exhibits a negative sign on the resource of bar bending machine hours, implying that the increase in bar bending hours reduces the quantity of reinforcement. In most cases, reinforcement is provided as straight bars or cut to length. Bending is required in all the bars. Hence, this negative sign implies that, with increase in bar bending instead of being laid straight, there will be a reduction in quantity. On the other hand, Model 6 for reinforcement work in the non-monsoon period shows a positive sign towards the independent variable height. This is due to the use of a higher quantity of long bars on three floors, as a 12 m bar is useful for floors of 4 m height in specific cases.

**Formwork quantity model**

In this model, formwork quantity is the dependent variable, while the parameters of height \((H)\), man-days \((MD)\), tower crane hours \((TCH)\) and hoist hours \((HH)\) are independent variables.

The best model obtained using multiple linear regression analysis is:

\[
FW = -519.279 + 0.192 \times H + 0.454 \times MD + 7.223 \times TCH + 4.856 \times HH
\]

\((R^2 = 0.950; N = 624; SE = 2609.25)\) (Model 7)

The plot between actual quantity (out-of-sample data) and the derived quantity (sample data) for formwork validates the model, as shown in Figure 3. The alignment of plotted points along the line of equality indicates the robustness of the model.

The model for formwork quantity in the monsoon period is:

\[
FW_{\ monsoon} = 41.864 + 1.828 \times H + 0.437 \times MD + 5.974 \times TCH + 5.373 \times HH
\]

\((R^2 = 0.950; N = 136; SE = 2438.09)\) (Model 8)

The model for formwork quantity in the non-monsoon period is:

\[
FW_{\ non-monsoon} = -313.592 - 2.609 \times H + 0.417 \times MD + 7.879 \times TCH + 4.795 \times HH
\]

\((R^2 = 0.961; N = 51; SE = 2056.62)\) (Model 9)

It is observed from the coefficients of the variables in formwork models (Models 8 and 9) that the formwork quantity is higher for the monsoon period than for the non-monsoon period. This may be due to the fact that major quantities of formwork are being done with system formwork and climbing formwork, which involve a continuous process of work, even in the monsoon period but with protection against the rains.

**Blockwork quantity model**

In this model, blockwork quantity is the dependent variable, whereas the parameters of height \((H)\), man-days \((MD)\), tower crane hours \((TCH)\), and hoist hours \((HH)\) are independent variables.
The best model obtained using multiple linear regression analysis is:

\[
BW = 49.662 - 0.514H + 0.183MD + 2.887TCH + 1.221HH \quad (R^2 = 0.932; N = 480; SE = 374.08) \quad \text{(Model 10)}
\]

The plot between actual quantity (out-of-sample data) and the derived quantity (sample data) for blockwork validates the model, as shown in Figure 4. The alignment of plotted points along the line of equality indicates the robustness of the model.

The model for blockwork quantity in the monsoon period is:

\[
BW_{\text{monsoon}} = 15.621 - 1.263H + 0.176MD + 2.529TCH + 1.404HH \quad (R^2 = 0.936; N = 103; SE = 370.76) \quad \text{(Model 11)}
\]

The model for blockwork quantity in the non-monsoon period is:

\[
BW_{\text{non-monsoon}} = -61.466 + 0.572H + 0.234MD + 2.753TCH + 1.213HH \quad (R^2 = 0.933; N = 376; SE = 385.63) \quad \text{(Model 12)}
\]

**Plastering quantity model**

In this model, plastering quantity is the dependent variable, while the parameters of height (H), man-days (MD) and hoist hours (HH) are independent variables.

The best model obtained using multiple linear regression analysis is:

\[
P = 929.613 - 0.491H + 0.430MD + 1.863HH \quad (R^2 = 0.861; N = 480; SE = 1667.05) \quad \text{(Model 13)}
\]

The plot between actual quantity (out-of-sample data) and the derived quantity (sample data) for plastering validates the model, as shown in Figure 5. The alignment of plotted points along the line of equality indicates the robustness of the model.

The model for plastering quantity in the monsoon period is:

\[
P_{\text{monsoon}} = 591.723 + 7.521H + 0.571MD + 1.607HH \quad (R^2 = 0.865; N = 103; SE = 1546.55) \quad \text{(Model 14)}
\]

The model for plastering quantity in the non-monsoon period is:

\[
P_{\text{non-monsoon}} = 901.529 + 1.739H + 0.385MD + 1.881HH \quad (R^2 = 0.854; N = 376; SE = 1688.85) \quad \text{(Model 15)}
\]

**Statistical validity of the models**

Table 3 shows the statistical validity of the models and the significance of the variables, verified through validity checks with reference to the standard parameters of “t” values, “P values” and variance influencing factor (VIF) for each independent variable considered in the models.

The acceptable “student’s t” statistic value for 95% confidence level is 1.645, but Table 3 reveals “student’s t” values of more than 1.645 for all the resource parameters, implying a normal distribution across observations. The acceptable P values is 0.05 but the table shows that “P values” are less than 0.05, implying that the variables included for model development are significant to the model.

Interestingly, the coefficient of the variable height in each activity, except formwork, is negative, i.e. for activities concrete, reinforcement, blockwork and plastering. The negative value of height...
indicates that, as height increases, productivity decreases. This is logical and understandable, given the increased time for lifting materials from the base for each additional floor. In the four activities the materials have to be lifted from the bottom every time for an additional height, and hence the productivity decreases with increase in height. However, in formwork activity we have a positive coefficient, which defies the above logic. The increase in formwork productivity with height is due to the increased number of repetitions from floor to floor, with the same material being lifted without disturbing the standard configuration. This increase in productivity is attributed to the specialisation that the resources imbibe, due to repetitive nature of the work.

The results of regression statistics and ANOVA for the models are shown in Tables 4 and 5, respectively.

It is observed from Table 4 that multiple R is more than 0.9 for all five the models, implying correlation between the observed and predicted value, which is adequate to comply with construction requirements. The value of significance F in Table 5 is less than 0.05 for all five the models, which implies that the developed models are significant. The collinearity statistics for all the independent variables is established indicating that the variance influencing factor (VIF) is greater than ‘1’ in each case. However, multi-collinearity between the independent variables exists, as the VIF is more than ‘2’. For example, in concrete work the two independent variables of concrete pump hours and transit mixer hours have strong multi-collinearity because, without a transit mixer to deliver concrete, the concrete pump resource will not be useful.

**Inference from quantitative analysis**

In all the above models, the variables such as man hours, tower crane hours, hoist hours and other equipment hours appear

<table>
<thead>
<tr>
<th>Table 4 Regression statistics of the developed models</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regression statistics</td>
</tr>
<tr>
<td>Multiple R</td>
</tr>
<tr>
<td>R Square</td>
</tr>
<tr>
<td>Adjusted R square</td>
</tr>
<tr>
<td>Standard error</td>
</tr>
<tr>
<td>Observations</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 5 ANOVA results of the models</th>
</tr>
</thead>
<tbody>
<tr>
<td>df</td>
</tr>
<tr>
<td>----</td>
</tr>
<tr>
<td><strong>Concrete</strong></td>
</tr>
<tr>
<td>Regression</td>
</tr>
<tr>
<td>Residual</td>
</tr>
<tr>
<td>Total</td>
</tr>
<tr>
<td><strong>Reinforcement</strong></td>
</tr>
<tr>
<td>Regression</td>
</tr>
<tr>
<td>Residual</td>
</tr>
<tr>
<td>Total</td>
</tr>
<tr>
<td><strong>Formwork</strong></td>
</tr>
<tr>
<td>Regression</td>
</tr>
<tr>
<td>Residual</td>
</tr>
<tr>
<td>Total</td>
</tr>
<tr>
<td><strong>Blockwork</strong></td>
</tr>
<tr>
<td>Regression</td>
</tr>
<tr>
<td>Residual</td>
</tr>
<tr>
<td>Total</td>
</tr>
<tr>
<td><strong>Plastering</strong></td>
</tr>
<tr>
<td>Regression</td>
</tr>
<tr>
<td>Residual</td>
</tr>
<tr>
<td>Total</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 6 Summary of responses – manpower productivity</th>
</tr>
</thead>
<tbody>
<tr>
<td>S no</td>
</tr>
<tr>
<td>------</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
</tbody>
</table>
with positive coefficients, indicating that an increase in these variables leads to an increase in productivity. On the other hand, the variable height appears with a negative coefficient, showing that, with the increase in height, the production decreases, except for the formwork model. In all the models the plotted lines aligned along the line of equality, indicating the robustness of the models.

**QUALITATIVE ANALYSIS**

**Questionnaire responses**

Questionnaires with the factors presented in Table 2 were distributed to 130 persons and responses collected from 96 persons who were involved in the execution of the 52 selected tall residential building projects. The responses were tabulated in order to understand the importance of various factors affecting productivity. The responses to the factors affecting manpower productivity and equipment productivity have been separately analysed and are shown in Tables 6 and 7, respectively.

The relative important index (RII) was calculated for the summary of responses for both manpower and equipment productivity. The RII in Tables 6 and 7 indicates that planning factors are the most important, followed by human factors in both cases.

**Figure 6** Frequency index for factors affecting manpower productivity

![Figure 6](image)

**Table 7 Summary of responses – equipment productivity**

<table>
<thead>
<tr>
<th>S No</th>
<th>Factors</th>
<th>Affects strongly</th>
<th>Affects significantly</th>
<th>Affects moderately</th>
<th>Does not affect</th>
<th>Cannot say</th>
<th>RII</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Human factors</td>
<td>30</td>
<td>42</td>
<td>22</td>
<td>2</td>
<td>0</td>
<td>0.808</td>
</tr>
<tr>
<td>2</td>
<td>Planning factors</td>
<td>48</td>
<td>32</td>
<td>13</td>
<td>2</td>
<td>1</td>
<td>0.858</td>
</tr>
<tr>
<td>3</td>
<td>System factors</td>
<td>22</td>
<td>41</td>
<td>23</td>
<td>6</td>
<td>2</td>
<td>0.744</td>
</tr>
<tr>
<td>4</td>
<td>Resource/equipment factors</td>
<td>27</td>
<td>38</td>
<td>23</td>
<td>6</td>
<td>2</td>
<td>0.771</td>
</tr>
<tr>
<td>5</td>
<td>Environmental factors</td>
<td>17</td>
<td>37</td>
<td>33</td>
<td>8</td>
<td>2</td>
<td>0.729</td>
</tr>
</tbody>
</table>

The quantitative and qualitative studies of the research, based on the data collected from 52 tall residential building construction projects across India and various analyses conducted on them, reveal several findings.

**DISCUSSIONS OF THE FINDINGS**

The quantitative and qualitative studies of the research, based on the data collected from 52 tall residential building construction projects across India and various analyses conducted on them, reveal several findings.

**A. Quantitative study**

1. Fifteen multiple linear regression models, three for each of the five basic activities of tall building construction (concreting, reinforcement, formwork, blockwork and plastering) were developed in this study for the productivity of manpower and equipment for tall residential building construction projects covering the monsoon season and the non-monsoon season.
Figure 7 Frequency index for factors affecting equipment productivity

2. The 15 developed models were evaluated and proven, with reference to the out-of-sample data, and were found to satisfy resource estimation requirements adequately.

3. In all the models, man-days are found to be the most significant explanatory variable influencing the quantity of different activities.

4. Tower crane hours and hoist hours are the next significant explanatory variables influencing the quantities.

5. The effect of monsoon on the quantities of different activities taken for the study is not significant, with a reduction of 2.15% for concreting quantity, 3.58% for formwork quantity and 3.90% for plastering quantity.

6. These models can be used as an effective tool in the estimation and monitoring of manpower and equipment resources in tall building construction projects at different stages of work in India and other South Asian developing countries.

B. Qualitative study

1. Based on expert opinion, 21 factors affecting manpower productivity were identified and were segregated into four groups: human factors, planning factors, system factors and environmental factors. Twenty factors affecting equipment productivity were identified and were segregated into five groups: human factors, planning factors, system factors, environmental factors and resource/equipment factors.

2. The frequency indices for the 21 factors affecting manpower productivity, and the 20 factors affecting equipment productivity were calculated from the responses received from 96 respondents to two questionnaires. The range of frequency indices for manpower productivity is between 0.555 and 0.820, and the range of frequency indices for equipment productivity is between 0.535 and 0.805.

3. The three important factors affecting manpower productivity are improper planning of work with a frequency index of 0.82, non-availability of materials with a frequency index of 0.80, and lack of skill of workman with a frequency index of 0.80.

4. The three important factors affecting equipment productivity are improper planning of work with a frequency index of 0.81, lack of skill of operator with a frequency index of 0.79, and equipment breakdown with a frequency index of 0.78.

CONCLUSIONS

Productivity of manpower and equipment resources in construction projects is usually calculated using empirical methods, experience of personnel and in some cases the manufacturer’s recommendation. Using models for the estimation and monitoring of productivity provides leverage to the project team to scientifically estimate and monitor productivity for a given set of resources and a given set of conditions in construction projects, particularly tall buildings. In this study, 15 robust models were developed for the five basic activities of construction for three conditions, and these models were validated using out-of-sample data. These models cover the resources to be used for each activity in tall building residential projects. Similarly, this study also identifies the factors affecting manpower and equipment productivity, and also the intensity of their effect. The planning factors affect the productivity more than other factors and need to be addressed in construction projects, particularly tall residential building projects.

LIMITATIONS AND SUGGESTIONS FOR FURTHER STUDY

Firstly, the models developed in this study can be used for tall residential buildings with RCC framed structure of column-beam-slab construction. However, they cannot be used for the construction of composite structures with structural steel.

Secondly, these models will be useful in construction involving a combination of resource input in the form of manpower and equipment. However, these will not be useful for fully mechanised projects or fully manpower-oriented projects without equipment.

Thirdly, these models can be best applied in South Asian developing countries which follow a certain pattern of construction, with a combination of manpower
and equipment resources in similar environmental and climatic conditions, especially with similar periods of monsoon and non-monsoon. If the operational and climatic conditions are different, then they may not have much applicability. Further, it is possible to extend the benefit of the models to other construction activities like façade work, interior works, etc, for which the equipment and labour requirement will be different from those of the activities covered in this study.

CONFLICT OF INTEREST
The authors declare that there is no conflict of interest regarding the publication of this paper.

REFERENCES


Mistry, S & Bhatt, R 2013. Critical factors affecting labor productivity in construction projects: Case study of South Gujarat Region of India.


Can detrimental carbonation of cement or lime-stabilised road base layers, and the occurrence of biscuit layers as a result of carbonation, be controlled by proper construction techniques only?

C J Semmelink, N J J Jooste, J E Raubenheimer

According to international standards, the majority of paved roads in South Africa are considered as low-volume roads. For this reason, the use of stabilised base layers under thin surfacings is still extensively used in the rehabilitation of these roads and the construction of new road pavements. In this spectrum of roads, problems have occurred with regard to the cementation of such layers. In most cases, these issues arise at the very top of these base layers, and the most common explanation has usually been described as “detrimental carbonation” or similar wording. It is especially the early problems within the construction period and soon thereafter that are under scrutiny in this paper.

This paper presents an update on the experience with chemically stabilised base layers in South Africa, and shows how the so-called “detrimental carbonation” is not a situation that can be eliminated during construction. The paper further argues the reason why the problem is material-related, and that a water-driven reaction probably causes it. Other opinions are that carbonation is the only cause and that carbonation is unaffected by moisture. A further opinion is that standard prescribed compaction techniques, curing and the application of prescribed bituminous types of prime can also be the cause of so-called “detrimental carbonation”. The conclusion is that the problem is moisture or “water-driven”, as water vapour is required for the carbonation reaction to take place.

INTRODUCTION

Ever since the onset of the construction of cement-stabilised base layers, problems have occurred with regard to the cementation of such layers, especially at the top of these base layers under thin surfacings. The most common explanation is usually described as “detrimental carbonation” or similar wording. While a cemented base option may be considered on low-volume roads, this type of design has inherent risks associated with it, both from a design and construction perspective. Crushing calculations assuming a lightly cemented C4 base and using a design tyre pressure of 750 kPa suggest that crushing should be expected in less than a year. Even a C3 base layer may be a risk, since the TRH4 Catalogue assumes a design tyre pressure of 520 kPa, which is no longer applicable to South African roads (GAUTRANS 2004). A soft layer at the top of a chemically stabilised base layer may well also be the result of a number of factors, or a combination of factors. These other related factors that may also contribute to the formation of a soft layer at the top of a cemented layer may include:

- Stabilisation design, especially with regard to satisfying the initial consumption of stabiliser
- The addition of too much water during the compaction process
- Premature spraying of water as part of the curing process, which may have the same effect as above
- Wet-dry cycles caused by the specified curing process
- Type of prime and application rate.

The focus of this paper is carbonation, cement, relative humidity, moisture, biscuit layer

Keywords: carbonation, cement, relative humidity, moisture, biscuit layer
after opening during the contract’s retention period. General knowledge about the problem, and some of the other documents mentioned in this paper, are of the opinion that carbonation referred to as “detrimental carbonation” can be avoided through the construction and curing processes. This paper shows why the authors consider the problem to be moisture or “water-driven”, as water vapour is required for the carbonation reaction to take place.

PERSPECTIVES FROM LITERATURE
The main arguments in some other published papers on the subject of carbonation are the following:

- Goodbrake et al (1979) determined that the percentage of calcium silicate reacting with carbon dioxide reduces significantly below a relative humidity of 50% and is relatively constant (about 75%) at a relative humidity in excess of 50%.
- Roberts (1981) stated that “…carbonation is greatest at ordinary temperatures in the relative humidity range of 50 to 75 percent.” (Page 15)
- Paige-Greene et al (1990) stated: “The influence of relative humidity on the rate of carbonation, although extremely important as a source of moisture necessary for the reaction to occur, is not clearly defined.”
- Ballim and Basson (in Fulton 2002 page 150) stated that no carbonation takes place when the pores are completely dry or when they are fully saturated, and that the rate of carbonation also increases with increasing ambient temperature.
- Botha et al (2005) presented a paper at the TREMTI Conference in Paris in 2005 indicating that many of the problems in South Africa that were attributed to carbonation, were caused by ‘water-driven reactions’ and were thus material-related and not construction-related.
- Wikipedia, the free online encyclopaedia, defines humidity as a term for water vapour in the air. Relative humidity measures the current absolute humidity relative to the maximum humidity and is expressed as a percentage (Wikipedia n.d.A.).
- Wikipedia, the free online encyclopaedia, also states the following about the effect of carbonation on phenolphthalein: “The acid-base indication abilities of phenolphthalein also make it useful for testing for signs of carbonation reactions in concrete. Concrete has naturally high pH due to the calcium hydroxide formed when Portland cement reacts with water. The pH of the ionic water solution present in the pores of fresh concrete may be over 14. Normal carbonation of concrete occurs as the cement hydration products in concrete react with carbon dioxide in the atmosphere, and can reduce the pH to 8½ to 9, although that reaction usually is restricted to a thin layer at the surface. When a 1% phenolphthalein solution is applied to normal concrete, it will turn bright pink. If the concrete has undergone carbonation, no colour change will be observed.” (Wikipedia n.d.B.)
- In an abstract in a TREMTI 2010 paper, Paige-Greene (2010) states the following: “During the early 1980s, a number of problems related to the loss of stabilisation and disintegration of stabilised layers in roads (lime and cement) were reported in South Africa. This led to many comprehensive investigations, and it was shown without any doubt that the problems were related to carbonation of the stabilised materials. A paper was presented by Botha et al (2005) at the TREMTI Conference in Paris in 2005 indicating that many of the problems in South Africa that were attributed to carbonation, were actually caused by ‘water-driven reactions’ and were thus material-related and not construction-related. This paper assesses the fundamental principles of each of the processes and draws conclusions as to their likelihood and the increasing occurrence of stabilisation problems. It is concluded that, although there is indubitable proven field and laboratory evidence for carbonation of stabilised layers, there is no solid scientific evidence for the occurrence of the ‘water-driven reactions’ in soil stabilisation for roads.”

The paper also states that the carbonation reaction depends on the solubility and diffusion of the components. “The diffusion is controlled by the concentration differences and is an inward diffusion of CO₂ gas and carbonate ions (Lagerblad 2005). The gas diffusion is much faster than ion diffusion. Thus the rate of reaction is controlled by the humidity in the material, i.e. how much liquid fills the connected pore system. In dry material, the CO₂ can penetrate well, but there is insufficient water for the reaction to take place. In the saturated condition, only the carbonate ions move, and carbonation is slow. Typically, the reaction is most likely and rapid at humidity levels of 40% to 70% (Lo & Lee 2002; Ballim & Basson 2001; Gjer & Oppsal 1998).”

ARGUMENTS WHY CARBONATION IS WATER-DRIVEN
Assume for the moment that “poor curing techniques cause detrimental carbonation” of the stabilised base surface during and after the construction thereof.

As everyone involved in road construction should know, all cement-stabilised layers are compacted at optimum moisture content (OMC). However, priming of the base is done at roughly 50% of OMC (i.e. the moisture content where the inter-particle suction forces of remaining water in the layer peak in order to ensure proper adhesion of the road-prime to the cement-stabilised layer). From the earlier statements it is clear that there are only two points in cement-treated materials’ moisture regimes (including concrete) where no carbonation takes place, namely 0% (i.e. totally dry) or 100% saturated so that CO₂ cannot be taken up in the pores of the material.

From the above-mentioned construction facts it is clear that the cement-stabilised layer is never totally dry or 100% saturated at any time during its life cycle. If one were to try and prevent carbonation from taking place by saturating the layer, the layer would start pumping under traffic, because inter-particle stresses are reduced by the pore water pressure in the saturated voids. So this is not a workable solution. If hypothetically the layer could be dried back rapidly to 0% moisture content, the layer would probably ravel because of the lack of inter-particle suction forces (which is clearly seen on any very dry gravel road), as the added amount of cement is insufficient to ensure chemical bonds between all particles. Furthermore, it is impossible to compact a totally dry material effectively. It is also clear that water vapour is needed for the carbonation reaction to take place (i.e. water-driven reaction).

Because the cement-stabilised layers are primed at about 50% of the OMC, a substantial amount of the inter-particle
voids will not be filled with water at the time of priming. The moisture regime of these voids is situated right in the middle of the so-called active carbonation range of 40% to 60% of humidity. As the layer surface heats up during the day, the temperature of the layer also increases, causing the humidity in the inter-particle voids to increase. As the surface is sealed by the prime and surfacing layers, very little of this moisture in suspension as moisture vapour is lost to the atmosphere. Prior to priming, the surface layer’s moisture content also fluctuates between totally wet and totally dry each time the stabilised layer is sprayed with water for curing as specified. This causes a number of carbonation cycles in the stabilised curing as specified. This causes a number of carbonation cycles in the stabilised layer each day, as no contractor can keep the layer in a totally saturated or totally dry condition all the time.

However, the carbonation reaction does not stop after the curing period, but continues as long as there is free lime available in the material, and the moisture content of the stabilised layer is in the active carbonation range of 40% to 60% humidity. As the TREMTI 2005 paper showed (Botha et al 2005), the loose surface material was broomed off before priming and surfacing of the cement-stabilised layer (i.e. normal practice). Later pictures (see photographs of the project in the Appendix on pages 38–39) of the same layer surface show that a loose (dry) powdery layer had formed on top of the surface of the cement-stabilised layer just below the prime and the surfacing. This dry layer is very unusual, as the pavement layers under a surface road are normally moist below the road surfacing. What is more, the dark pink colour of the sprayed phenolphthalein solution on the material shows that there is still abundant free lime (i.e. cement) available in the material, which rules out detrimental carbonation.

Despite the presence of free lime from the cement, the surface ravelled under the surfacing because there was no inter-particle suction force between the particles, due to the dry material. However, because it is usually moist (damp) below the surfacing, one can reason in this case as well that the dryness of the material was not caused by evaporation of moisture through the prime and bituminous road surfacing, but was possibly caused by exceedingly high intra-particle suction forces of certain minerals (e.g. illite-smectite) in the material matrix that sucked the inter-particle water into the mineral particles themselves to form part of the chemically bound water, which can only be driven off at exceedingly high temperatures in the region of 600°C. Thus normal oven-drying at 100°C will not show the presence of the chemically bound water at all. This is why the authors are convinced that this damage is caused by a water-driven reaction, despite the fact that no proven chemical reaction or proven physical explanation for this dry powdery layer could be given in this case.

Other evidence from literature

Paige-Greene et al (1990)

Paige-Greene et al (1990) stated: “Distress or failure assessed to be primarily due to loss of stabilisation has occurred in at least 100 cases in southern Africa over the last 30 years.” [See Table 1 in Appendix.] “This represents an average rate of about three per year. The most common form of distress is surface disintegration of the primed base during construction and scabbing of the seal in service due to inadequate bond with the base. In many cases a loose layer of disintegrated base course material was noticed between the surfacing and the base. A less frequent occurrence has been a partial or even complete loss of cementation, and a large decrease in strength, leading to rutting, cracking and shearing or pumping. In a few cases an increase in the plasticity index has been found. Lime, cement and lime-slag were used as stabilisers, in amounts ranging between 2% and 5% by mass. Carbonation has been confirmed, or is strongly indicated, to have been a factor in about half of the cases. In the remainder of cases the available information does not permit an assessment of whether or not carbonation was involved.” (Page 2)

In the same document (Paige-Greene et al 1990) they also state: “The influence of relative humidity on the rate of carbonation, although extremely important as a source of moisture is necessary for the reaction to occur, is not clearly defined. Addis (1986) includes a figure showing that carbonation shrinkage is a maximum at a relative humidity of about 50 per cent and decreases to zero at 100 per cent. Goodbrake et al (1979) have determined that the percentage of calcium silicate that reacted with carbon dioxide declines steeply below a relative humidity of 50 per cent and is more or less constant (about 75 per cent) at a relative humidity in excess of 50 per cent. Roberts (1981) states that carbonation is greatest at ordinary temperatures in the relative humidity range of 50 to 75 per cent.” (Page 15)

The following techniques can be used to ascertain whether carbonation has occurred (Paige-Greene et al 1990 pp 18–20):

1. **Phenolphthalein** (Netterberg 1984 in Paige-Greene et al 1990)
   
   $$ \text{pH} < 8.4 \quad \text{pH} \leq 11 $$

   Colourless . . . . . . . . . . . . . Red

2. **Dilute HCl (5N)**

   $$ \text{Ca(OH)}_2, \text{CSH}, \text{CAH}: \text{no effervescence} $$
   $$ \text{CaCO}_3: \quad \text{effervescence} $$

3. **pH (paste or low water: soil ratio)**

   $$ \text{Ca(OH)}_2, \text{OPC}: \quad \geq 12.4 $$
   $$ \text{CSH}, \text{CAH}: \quad = 11.0 \text{ to } 12.4 $$
   $$ \text{CaCO}_3: \quad \pm 8.3 $$

As the reaction products are only stable at a pH of 11 to 12.5, further work is necessary to find a reliable field method of measuring such pH levels. Of all the indicators tried, only phenolphthalein appears to be reliable in all cases. However, this can only indicate when the pH is in excess of about 10 (and less than about 8.4).

De Wet and Von Solms (1985)

De Wet and Von Solms (1985) stated: “The best humidity for the test has not yet been ascertained. Carbonation should be fastest at 40–60%. However, this is an air-dry condition which will take time to achieve in the pores of the specimen (which are about 75% saturated with water at OMC) ... If vacuum soaking is carried out, the specimen should be placed in the vacuum chamber, a vacuum of 80 kPa applied slowly over a period of 10 minutes, the vacuum tap closed, and water slowly allowed to enter the chamber over a period of 10 minutes to completely cover the specimen and to release the vacuum. Allow to soak for 10 minutes and remove specimens.”

The carbonation reaction is therefore shown here to be water-driven.

Lowe and Von Solms (1986)

Lowe and Von Solms (1986) concluded:

1. “In South West Africa [Namibia today] lime stabilisation of calcrete has generally been successful, but on occasion unsuccessful. It seems that the quality of the end product depends on the composition of the calcrete and the reaction of the lime with the material.” (Point 4.1)
There is strong evidence that carbonation occurred the pH was generally below 12. pH is high. The records show that in most cases of dispute, one cannot assume that, because the material seems suitable and the laboratory design is workable, the road builder is responsible when the required results are not achieved. This is not a balanced approach, as it is also laboratory-design-based (i.e. using accelerated material testing techniques not reflecting actual construction constraints experienced on site), and not only construction-related. From the evidence of the original report by Paige-Greene et al (1990) it is clear that the authors of that report found that detrimental carbonation had only been confirmed, or was strongly indicated to have been a factor, in about 50% of the cases investigated. Twenty years later in the 2010 TREMTI paper by the same author (Paige-Greene 2010) on the same subject, the conclusion was made that this loose powdery layer underneath the surfacing and prime is caused by carbonation, which implies poor curing techniques, thus making it a construction-related problem only.

The authors of this paper are of the opinion that the statement in the 2010 TREMTI paper by Paige-Green (2010) is incorrect, as the dark pink colour of the sprayed phenolphthalein solution on the so-called carbonated material found in the survey done in the investigation as set out in the Appendix, contradicts this statement. One would further expect the whole surface of the "poorly cured" stabilised layer evaluated (see Appendix) to be affected by the detrimental "carbonation", which is not the case either. In actual fact, this particular road shown in the Appendix has been trafficked for more than ten years without any major surface failures. Keep in mind that all loose surface material had been removed from the stabilised road surface before priming and surfacing the road. From the evidence presented it should be clear that in at least 50% of the cases this loose powdery layer is probably caused by an as yet undefined water-driven reaction which is beyond the control of the road builder and therefore not construction-related. Lowe and Von Solms (1986) came to a similar conclusion.

Furthermore, material-related construction problems will only be identified in the laboratory if the laboratory test procedures simulate site construction conditions.

"Not everything in a TRH or other document is necessarily valid indefinitely: it only represents the state-of-the-practice or else the best knowledge available at that time. As an interim measure the normal TRH 14 (NITRR 1985b) and TRH 13 (NITRR 1986a) requirements should be applied to the residual values." (Paige-Greene et al 1990) (p 34).

It is evident that more research is required to fully understand the cause of this phenomenon.

SUMMARY OF EVIDENCE

1. Water or water vapour is required for the carbonation reaction to take place (i.e. a water-driven reaction).
2. There are only two points in the moisture regime of a cement-stabilised layer where carbonation does not take place, namely totally dry or totally saturated.
3. The structural layers of a road below the prime layer are never totally dry or totally wet.
4. Phenolphthalein colours the cement-stabilised material pink when the material has not carbonated and remains colourless when carbonated, and therefore shows that the road investigated in 2001 had not carbonated (see Appendix).
5. Carbonation was identified to have played a part in the detrimental condition of the cement-stabilised layers in only 50% of sites investigated by Paige-Greene et al (1990). The other 50% was therefore probably caused by another as yet unidentified water-driven reaction.
6. The evidence clearly shows that both detrimental reactions are water-driven, which is beyond the control of road builders and therefore not a construction-related issue only.

7. The safest curing process is therefore to do no curing on site with a water cart, but rather to leave it as is.

REFERENCES


Fulton 2002. Fulton's Concrete Technology. Johannesburg: Concrete Institute of South Africa.


Lowe, E H & Von Solms, C L 1986. Experience and practice with modified basecourses in South West Africa. Paper presented at the meeting at which TRH 13 [Cementitious Stabilizers in Road Construction] (1986) was introduced to practice.


APPENDIX

Photographs taken during an investigation by Semmelink and Botha in 2001

Photograph 1 General condition of the road surface (2001)

Photograph 2 Close-up of the road surface in a next-to-the-wheel track (2001)

Photograph 3 The dry powdery layer just below the surfacing; note the dark pink colour of phenolphthalein indicating free lime (2001)

Photograph 4 Dark pink colour of phenolphthalein visible through the whole depth of the stabilised layer, indicating free lime through the whole depth (2001)
Photographs taken in March 2012 of the same road

Table 1 Known cases of loss of stabilisation in southern Africa (1957–1986) which have led to distress\(^1\) (Paige-Greene et al 1990)

<table>
<thead>
<tr>
<th>Area</th>
<th>Lime</th>
<th>Cement</th>
<th>Lime-slag</th>
<th>Loose prime and/or surfacing scabbing</th>
<th>General loss of strength</th>
<th>Return of PI</th>
<th>Carbonation(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cape</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>3</td>
<td>1</td>
<td>–</td>
<td>1</td>
</tr>
<tr>
<td>Natal</td>
<td>10</td>
<td>2</td>
<td>–</td>
<td>7</td>
<td>7</td>
<td>1</td>
<td>8</td>
</tr>
<tr>
<td>Orange Free State</td>
<td>1</td>
<td>2</td>
<td>2</td>
<td>10</td>
<td>1</td>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>South West Africa</td>
<td>11</td>
<td>–</td>
<td>–</td>
<td>9</td>
<td>2</td>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>Transvaal</td>
<td>3</td>
<td>9</td>
<td>6</td>
<td>19</td>
<td>9</td>
<td>2</td>
<td>13</td>
</tr>
<tr>
<td>Total</td>
<td>26</td>
<td>14</td>
<td>9</td>
<td>48</td>
<td>20</td>
<td>5</td>
<td>28</td>
</tr>
<tr>
<td>Botswana</td>
<td>3</td>
<td>2</td>
<td>–</td>
<td>4</td>
<td>3</td>
<td>1</td>
<td>6</td>
</tr>
<tr>
<td>Lesotho</td>
<td>1</td>
<td>–</td>
<td>–</td>
<td>2</td>
<td>–</td>
<td>–</td>
<td>1</td>
</tr>
<tr>
<td>Malawi</td>
<td>2</td>
<td>7</td>
<td>–</td>
<td>9</td>
<td>8</td>
<td>–</td>
<td>7</td>
</tr>
<tr>
<td>Swaziland</td>
<td>1</td>
<td>–</td>
<td>1</td>
<td>2</td>
<td>–</td>
<td>–</td>
<td>1</td>
</tr>
<tr>
<td>Zambia</td>
<td>–</td>
<td>1</td>
<td>–</td>
<td>1</td>
<td>27</td>
<td>–</td>
<td>27</td>
</tr>
<tr>
<td>Zimbabwe</td>
<td>1</td>
<td>1</td>
<td>–</td>
<td>3</td>
<td>–</td>
<td>–</td>
<td>1</td>
</tr>
<tr>
<td>Total</td>
<td>8</td>
<td>11</td>
<td>10</td>
<td>69</td>
<td>31</td>
<td>6</td>
<td>16</td>
</tr>
<tr>
<td>Grand total</td>
<td>34</td>
<td>25</td>
<td>10</td>
<td>69</td>
<td>31</td>
<td>6</td>
<td>44</td>
</tr>
</tbody>
</table>

1 Compiled from DRTT records, discussions, and a survey of stabilisation durability in South Africa. A “case” may be an airport or length of road from about 800 m to over 20 km.

2 Confirmed by a decrease in pH to 8–9 and an increase in CaCO\(_3\) content and similar symptoms to known cases of distress associated with carbonation.
**Impact of low viscosity grade bitumen on foaming characteristics**

S S Kar, A K Swamy, D Tiwari, P K Jain

The construction of highway infrastructure requires large quantities of construction materials and is an energy-intensive activity. Issues like scarcity of construction materials, increasing cost of materials, increasing cost of construction processes, and the emphasis on less polluting construction procedures have led to advancements in pavement construction practices. One such example is foamed bitumen technology. In this process, cold water is injected into a hot bitumen medium, which leads to drastic expansion of bitumen, forming a fine mist or foam. The performance of foam bitumen mixtures is influenced by the bitumen properties, as well as by the foaming process. This paper deals with research on the rheology of three different types of binders having low viscosity (in the range of 500 to 3,000 Poise at 60°C) on foaming characteristics. The water content and temperature also have significant effects on foam decay and the expansion ratio of foam. Results show that a low viscosity bitumen consumes less energy during the foam process compared to high viscosity bitumen. The conclusion drawn from this study is that the expansion ratio and half-life of foam bitumen depend on the rheology of the binder, as well as on the chemical composition of the bitumen. It was concluded that, with an increase in carbonyl and sulphoxide compound, the expansion ratio decreases and the half-life increases.

**INTRODUCTION**

The increase in road infrastructure around the world and its impact on the environment require serious attention to the development of more sustainable pavement construction. The quest for sustainability in pavement construction constitutes a strong incentive towards the use of cold mix asphalt technology. Foamed asphalt is an attractive cold asphalt mixture, and it is becoming an important subject area within sustainable pavement construction practices. It is reported that this mixture has been successfully implemented in many roads across the world, especially in cold recycling.

Foamed bitumen (also known as foamed asphalt or expanded asphalt) is a mixture of air, water and bitumen. When injected with a small quantity of cold water, the hot bitumen expands within an expansion chamber to about fifteen times its original volume and forms a fine mist or foam (Figure 1) (Wirtgen 2004). Foamed bitumen stabilisation (FBS) offers considerable advantages. The use of these mixtures conserves aggregates and bitumen, decreases energy usage, minimises waste and reduces fuel consumption and greenhouse gas emissions. This mixture can therefore significantly reduce the cost of construction (Morton et al. 2004). Engineering advantages include the possibility to use a wide variety of aggregates (Morton et al. 2002). The foamed binder increases strength when compared to a granular material, exhibiting more flexibility when compared to cement-treated materials, gives faster strength gains compared to cold emulsion mixtures and facilitates possible early opening to traffic (Jenkins et al. 2001).

The mechanical properties of FBS depend on the rheological properties of the foamed asphalt binder during production, during construction and when in service. To date the characteristics of foamed bitumen and their effect on mixture properties have not been critically analysed. It is necessary to select the best foam prior to mixing with aggregate materials. This is achieved by investigating the foaming properties of the bitumen. Researchers have reported that the foaming process limits the ageing of the binder and the mixtures (Xiao et al. 2013; Martinez-Arjuelas et al. 2017). In contrast, research by Nivedya et al. (2013) indicated that the foamed binder at lower shear rates exhibited viscosity close to the RTFO binder. This necessitates comparing...
the properties of foamed bitumen with that of the short-term aged binder.

It is necessary to investigate the effect of binder composition and binder rheology on foaming characteristics, which will provide a better idea of bitumen-stabilised material behaviour. This paper describes a detailed study related to the effect of binder viscosity on foaming characteristics. It deals with the study of bitumen properties, which includes the ageing parameter prior to foaming and its effect on foaming characteristics. Besides traditional testing (penetration, ring and ball softening point), advanced tests were conducted on bitumen before foaming, using equipment such as the Brookfield rotational viscometer and the dynamic shear rheometer (DSR). Three varieties of asphalt binder with lower viscosity were used in this work.

**LITERATURE REVIEW**

**Parameters determining characteristics of foam**

The production of effective foam bitumen for pavement recycling applications can be achieved when the base bitumen possesses acceptable characteristics. The main parameters used to determine the quality of the final foamed bitumen product are expansion ratio and half-life. Expansion ratio (ER) is calculated as the ratio of the maximum volume of foam relative to its original volume. ER is an indirect measure of the viscosity of the foam and is an indicator of dispersive characteristics. Bowering and Martine (1976) reported that expansion of the foamed bitumen is approximately 15 to 20 times the volume of the original bitumen. The half-life (HL) is measured in seconds and usually lies between 10 and 15 seconds. As the percentage of added water is increased, the HL and ER develop in opposite directions (Asphalt Academy 2009; Sunarjono 2008). Jenkins (2000) developed the concept of a foam index (FI) to measure the combination of expansion ratio and half-life. Saleh (2007) used viscosity of foam also as an indicator of foaming characteristics. Various researchers have indicated that viscosity is a better indicator of foam quality, mixing properties and the workability of FBM. The foam index needs to be analysed in combination with the expansion ratio and the half-life in order to optimise the foaming characteristics of tar for a specified application (Morton 2001). As per Morton et al (2003) optimum foaming results are generally achieved at temperature ranges of 110°C and 125°C, with foamanant water content between 1.5% and 2.5%.

**Acceptable range and optimisation of foaming characteristics**

Previous research on foamed bitumen indicated that acceptable values of ER and HL depend on various parameters (Csanyi 1960; Jenkins et al 1999). Accepted minimum values for ER and HL for stabilising material are 10 and 8 respectively (Bowering & Martin 1976; Asphalt Academy 2009) when the temperature aggregate is between 10–25°C. When aggregate temperature is higher than 25°C, the recommended values of ER and HL by various researchers are summarised in Table 1.

**Factors affecting foamed bitumen characteristics**

The characteristics of foamed bitumen are highly influenced by various factors like foamanant water content, bitumen temperature, viscosity of bitumen, type and composition of bitumen and the temperature of the vessel in which the foamed bitumen is collected (Jenkins & Van de Ven 2001). Foamed bitumen with a higher expansion ratio and longer half-life has better dispersion through granular materials, resulting in greater strength improvement (Wahhab et al 2012). With increasing water content and temperature, the expansion ratio is expected to increase with a simultaneous decrease in the half-life (Bowering & Martin 1976; Brennen et al 1983; He & Wong 2006). This can be attributed to more water availability, and subsequent steam generation. A higher quantity of steam generated leads to the formation of more bubbles. Optimised foamanant water content, along with optimum other parameters like air pressure and temperature of bitumen, were established by many researchers (Middleton & Forfylow 2009; Xiao 2011; Iwański & Chomicz-Kowalska 2012). Brennen et al (1983) found that the half-life and expansion ratio of foam were affected by the volume of the foam produced, foamanant water content and bitumen temperature at which the foam is produced. According to Nivedya et al (2013), while measuring the apparent viscosity of the foamed binder, the time taken to reach the steady state is independent of the water content and temperature of the foamed binder.

**Table 1 Summary of ER and HL reported in literature**

<table>
<thead>
<tr>
<th>Half-life (seconds)</th>
<th>Expansion ratio</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>≥ 20</td>
<td>8 to 15</td>
<td>Ruckel et al (1983)</td>
</tr>
<tr>
<td>≥ 7</td>
<td>≥ 7</td>
<td>Asphalt Academy (2002)</td>
</tr>
<tr>
<td>≥ 8</td>
<td>≥ 10</td>
<td>Wirgen (2004)</td>
</tr>
</tbody>
</table>
The relationship between the viscosity of foam with respect to time has been developed using a viscometer (Jenkins 2000; Saleh 2007). It is a known fact that the viscosity of bitumen decreases with increase in temperature. The subsequent thinning effect of bitumen on bubbles leads to a reduced half-life. When small bubbles coalesce with larger bubbles, the bitumen film thickness surrounding the bubbles reduces (Saleh 2007; Wirtgen 2004). This reduction in foamed bitumen viscosity leads to a simultaneous decrease in the surface tension in the bitumen film. This in turn leads to excessive steam pressure within the bubbles, leading to the final collapse of the bubbles (Csányi 1960).

Table 2 Properties of BEO

<table>
<thead>
<tr>
<th>Properties</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density, d415</td>
<td>0.98</td>
</tr>
<tr>
<td>°API</td>
<td>11.8</td>
</tr>
<tr>
<td>Kinematic viscosity, cSt at 100°C</td>
<td>92.8</td>
</tr>
<tr>
<td>Flash point, °C</td>
<td>242</td>
</tr>
<tr>
<td>Asphaltene content, % wt</td>
<td>0.62</td>
</tr>
<tr>
<td>Aromaticity, (by NMR)</td>
<td>0.4</td>
</tr>
<tr>
<td>Molecular weight (by VPO method)</td>
<td>660.5</td>
</tr>
</tbody>
</table>

**Figure 2** Effect of BEO on VG 10 bitumen

Table 3 Properties of LGB, VG 10 and VG 30

<table>
<thead>
<tr>
<th>Properties</th>
<th>Test data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetration (25°C, 100 g, 5s, 0.1 mm)</td>
<td>42.6 81 47</td>
</tr>
<tr>
<td>Softening point (ring and ball), °C</td>
<td>39 44 51</td>
</tr>
<tr>
<td>Viscosity at 60°C, Poise</td>
<td>564 1 150 3 230</td>
</tr>
<tr>
<td>Viscosity at 135°C, cSt</td>
<td>225 350 475</td>
</tr>
<tr>
<td>Temperature of 1.01 kPa G'/Sinδ of un-aged bitumen, °C</td>
<td>60.2 66 74</td>
</tr>
<tr>
<td>Temperature of 2.2 kPa G'/Sinδ of RTFOT residue, °C</td>
<td>64.0 73 75</td>
</tr>
</tbody>
</table>

**Effect of bitumen viscosity on foam characteristics**

The relationship between the viscosity of foam with respect to time has been developed using a viscometer (Jenkins 2000; Saleh 2007). It is a known fact that the viscosity of bitumen decreases with increase in temperature. The subsequent thinning effect of bitumen on bubbles leads to a reduced half-life. When small bubbles coalesce with larger bubbles, the bitumen film thickness surrounding the bubbles reduces (Saleh 2007; Wirtgen 2004). This reduction in foamed bitumen viscosity leads to a simultaneous decrease in the surface tension in the bitumen film. This in turn leads to excessive steam pressure within the bubbles, leading to the final collapse of the bubbles (Csányi 1960).

*Figure 2* Effect of BEO on VG 10 bitumen

---

**Effect of bitumen chemical composition on foam characteristics**

Studies by Barinov (1990) illustrated that bitumen containing a higher percentage of asphaltenes lead to a higher expansion ratio and half-life. This can be attributed to asphaltenes acting as surfactants. While acting as surfactants, asphaltenes reduce the surface tension in the lamellae of the bubbles, as well as the plateau border suction. Thus collapse of the foam is delayed. Crispino et al (2014) studied the foaming properties of chemical modified binder based on oleic acid diethanolamine, showing about 10 times better properties compared to the near binder. Other authors also support this finding (Namutebi et al 2011; Martinez-Arguelles et al 2015). Kendall et al (1999) found that silicones present in the bitumen reduced the foaming ability of bitumen. Thus the presence of silicones necessitates the use of a foaming agent for higher quality foam (Kendall et al 1999). According to Ivánskí et al (2015), an increase of wax content up to 2.5% induced an improvement in bitumen foamability, i.e. increase of expansion ratio and half-life. The effects of binder chemical composition on foaming using Fourier transform infrared spectroscopy and X-Ray radiography techniques have been investigated by various authors (Namutebi et al 2011; Hailesilassie et al 2015). Lesueur et al (2004) reported that bitumen composition did not influence foaming properties much. Saleh (2007) reported that temperature susceptibility of the binder plays an important role in the foaming process.

**EXPERIMENTAL WORK**

**Materials**

VG 10 and VG 30 grade bitumen conforming to Indian Standard (15 2013) produced at the Indian Oil Corporation Limited (IOCL) refinery were used in this study. A typical bitumen extract oil (BEO) was used to reduce the viscosity of VG 10 bitumen. In this study, lower grade bitumen (LGB) was obtained by blending of VG 10 bitumen and BEO in a stirrer for 15 minutes at 150°C. The physical properties of BEO used for lowering the viscosity of VG 10 bitumen are shown in Table 2. The effect of BEO on the viscosity of VG 10 bitumen is shown in Figure 2. The physical properties of bitumen are given in Table 3.
Experimental programme

The experimental study was conducted in two stages, namely:
A. Evaluation of physical and rheological properties of VG 30, VG 10 and low grade bitumen (LGB)
B. Assessment of the foaming properties using foaming pilot plant at laboratory.

The different laboratory tests conducted for these two stages are shown in Figure 3. The tests conducted for Stage A are shown in the upper dotted area and for Stage B in lower dotted area.

Study on the ageing of bitumen

Ageing is known to be a major phenomenon which influences the performance of bitumen. Many factors contribute to the hardening of bitumen, such as oxidation, volatilisation and polymerisation. The use of bitumen binder as paving material is dependent on its resistance to change in physical properties over the range of temperatures. In general, bitumen ageing takes place in two stages, namely short-term ageing at high temperature during asphalt mixing, storage and laying, and long-term ageing at ambient temperature while in service (seven to ten-year period). Therefore, bitumen (VG 10, VG 30 and LGB) were subjected to ageing by the rolling thin film oven (RTFO) test to simulate its short-term ageing, and the pressure ageing vessel (PAV) test to simulate its long-term ageing. The effects of ageing on the rheological and physical properties of bitumen binders were investigated using the dynamic shear rheometer test (DSR) and the Brookfield viscometer. The ageing index (ratio of viscosity after RTFOT and PAV test to viscosity before RTFOT and PAV test) is one of the most acceptable parameters used to determine the ageing resistance of bitumen.

Rheological test methods

The viscosity profile of the bitumen was measured using a Brookfield rotational viscometer. The viscosity of VG 10, VG 30 and LGB was measured at 120°C, 135°C, 150°C, 170°C and 180°C at a rotational speed of 20 rpm.

The viscoelastic response of the bitumen was evaluated using DSR with parallel plate geometry by measuring the complex shear modulus (G*) and the phase angle (δ) values. A 25 mm steel plate was used with a gap width of 1 mm, and measurements were taken at a frequency of 10 rad/sec with a strain level of 1%. Frequency sweep was conducted in the frequency range of 0.1–25 Hz at different temperatures (15–90°C), at an increment of 15°C. The samples were prepared by pouring the bitumen into silicon moulds with the appropriate geometry for the type of material to be tested.

The bending beam rheometer (BBR) was used to measure the stiffness and the rate of change of stiffness, i.e. the m-value of the binders at low temperatures. The BBR test was carried out as per AASHTO T313 specifications.

Temperature susceptibility determination

All types of bitumen display temperature dependent properties, i.e. they become softer when heated and hard when cooled. Several equations have been proposed by researchers to relate viscosity (or consistency) with temperature (Airey 2002; Roberts et al 1991). In this study three approaches are used to determine the temperature susceptibility of the LGB, VG 10 and VG 30.

Penetration Index (PI)

Pfeiffer and Van Doormaal developed an equation for the temperature susceptibility that assumes a value of about zero for road bitumens (Van der Poel 1954). They defined the penetration index (PI) as:

\[ PI = \frac{20(1 - 25A)}{1 + 50A} \]  

(1)

Where A is temperature susceptibility and the PI is an unequivocal function of A. If the logarithmic function of penetration P is plotted against temperature T, a straight line of the following form is obtained:

\[ \log P = KT + A \]  

(2)

The PI of each bitumen can be calculated using (1) and (2). Low PI values indicate high temperature susceptibility. In this study PI is calculated using penetration values of 15 and 25°C. In this study the PI system has been used to give a good

---

**Figure 3** Laboratory experimental study

- Penetration, softening point and temperature susceptibility (PVN, VTS and PI)
- Viscosity (120, 135, 150, 170 and 180°C)
- BBR at –14, –18, –20, –22, –24 and –28°C
- Rheological properties (5, 15, 30, 45, 60, 75 and 90°C)
- FTIR analysis
- Aging (RTFO and PAV)
- With variation bitumen temperature (120°C, 140°C, 160°C, 180°C)
- With variation of water content (2%, 4%, 6%, 8%, 10% and 11% by weight of bitumen)
- Effect of binder on foaming characteristics

---
approximation of the behaviour of bitumen to be expected, but its confirmation has been done by using viscosity measurements.

**Penetration-Viscosity Number (PVN)**

The penetration- viscosity (Mcleod 1972) number is another measure used to determine the temperature susceptibility of bitumens. This number is based on penetration at 25°C and viscosity at 60°C, using Equation 3.

\[
PVN = \frac{6.491 - 1.5 \log P_{25} - \log \eta_{60}}{1.05 - 0.22 \log P_{25}}^{(-1.5)}
\]

Where: \(P_{25}\) is penetration at 25°C and \(\eta_{60}\) is viscosity at 60°C.

**Viscosity Temperature Susceptibility (VTS)**

The viscosity temperature susceptibility (VTS) value for measuring temperature susceptibility for any particular bitumen sample was determined using Equation 4:

\[
VTS = \frac{(\log(\log \eta_1)) - (\log(\log \eta_2))}{\log T_1 - \log T_2}
\]

Where: \(\eta_1\) is the viscosity at \(T_1\) and \(\eta_2\) is the viscosity at \(T_2\). \(T_1\) and \(T_2\) in this study are 100°C and 135°C respectively.

**FTIR analysis**

Fourier transform infrared spectroscopy (FTIR) analysis was used for identification and quantification of functional groups present in bitumen. FTIR spectra were collected for binders using a Bruker-Alpha FTIR spectrometer equipped with a zinc-selenide (ZnSe) series and attenuated total reflectance (ATR). Bitumen samples were placed directly on the ZnSe window, and the spectra were obtained. Forty scans at a resolution of 0.8 cm\(^{-1}\) were taken for each sample, and background scans were between wave numbers 3 000 to 500 cm\(^{-1}\). The resultant spectra were corrected using both ATR and baseline correction functions in the OPUS software.

**Foaming procedure**

A bitumen foaming process was undertaken in the laboratory pilot plant to determine the optimum foaming water content and optimal foaming temperature according to the following test operational conditions:

- Air pressure: 550 kPa
- Water pressure: 600 kPa
- Temperature of bitumen: 120°C, 130°C, 140°C, 160°C and 180°C
- Water content: 2%, 4%, 6%, 8%, 10% and 11%
- Amount of bitumen being foamed during each test: 500 gm at the rate of 50 gm/sec.

**Sample preparation and strength characterisation**

To check the effective binding of foam using LGB in the mix, foam mix samples, termed as bitumen stabilised mix (BSM), were cast with three different foam binder contents (varying from 1.8 to 2.2% at an increment of 0.2% with 80% of recycled aggregate). In this study, BSM Level 1 mix design was carried out for all binders (LGB, VG 10 and VG 30) as per TG2 guidelines (Asphalt Academy 2002; 2009). The tensile properties of BSM were evaluated using indirect tensile strength (ITS).

Six specimens were compacted, of which three were used to determine ITS under unconditioned conditions and the rest were tested after proper conditioning. After proper curing, the unconditioned specimens were tested for ITS at 25°C. For conditioning, the specimens were placed in a water bath at 40°C for 24 hours and then placed in an environmental chamber maintained at 25°C for two hours. Using failure load, and specimen dimensions, dry and wet ITS were calculated for unconditioned and conditioned samples, respectively using Equation 5.

\[
\sigma_t = \frac{2P}{\pi Dt}
\]

Where:
- \(P\) = maximum load,
- \(D\) = diameter of the specimen, and
- \(t\) = thickness of the specimen.

## Table 4: Viscosity ratio of bitumen

<table>
<thead>
<tr>
<th>Bitumen</th>
<th>Viscosity ratio RTFOT aged</th>
<th>PAV aged</th>
</tr>
</thead>
<tbody>
<tr>
<td>LGB</td>
<td>2.3</td>
<td>4.0</td>
</tr>
<tr>
<td>VG 10</td>
<td>1.9</td>
<td>3.5</td>
</tr>
<tr>
<td>VG 30</td>
<td>1.5</td>
<td>3.0</td>
</tr>
</tbody>
</table>

## Table 5: Complex modulus values before and after RTFOT ageing of bitumen

<table>
<thead>
<tr>
<th>Bitumen</th>
<th>Complex modulus (G*) (kPa)</th>
<th>Complex Modulus Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Un-aged</td>
<td>RTFOT aged</td>
</tr>
<tr>
<td>LGB</td>
<td>7.2</td>
<td>20.4</td>
</tr>
<tr>
<td>VG 10</td>
<td>23.3</td>
<td>38.9</td>
</tr>
<tr>
<td>VG 30</td>
<td>31.4</td>
<td>52.3</td>
</tr>
</tbody>
</table>
ratio (ageing index) of the bitumen after the RTFOT and PAV ageing tests are given in Table 4. Results show that the viscosity ratio of LGB is higher than VG 10 and VG 30 indicating more stiffness after ageing. Due to the presence of BEO, it is more prone to oxidation compared to VG 10 and VG 30. $G'$ values at 45°C at 1.5 Hz before and after short-term ageing are given in Table 5. The complex shear modulus ratio shows similar results to the viscosity ratio. The $G'$ value of un-aged LGB is 7.2 kPa, which increases to 20.4 kPa after short-term ageing, which is nearly equal to the $G'$ value of un-aged VG 10 bitumen. It may be concluded that after short-term ageing the low grade bitumen can behave as VG 10 during the mix performance.

**Foaming characteristics with respect to expansion ratio and half-life**

Expansion ratio (ER) and half-life (HL) were determined for all binders at different foaming temperatures (120°C, 130°C, 140°C, 160°C and 180°C) and different water contents (2%, 4%, 6%, 8%, 10% and 11%). HL was determined as the time in seconds that the foam required to collapse from maximum expansion to half of expansion. Figure 5 shows the foaming properties of the different bitumen types, expressed in mean values of three replicates. It is observed that with increasing viscosity of bitumen, the HL increases and a higher temperature is needed for a better foaming process. The results indicate that LGB can produce better foam at 120°C compared to VG 10 and VG 30, whereas it will also require less energy. In Table 1 the minimum requirements of ER and HL for acceptable foam were summarised and compared to the global scenario; the bitumen used in this study shows higher half-life and expansion ratio, which may be due to the addition of higher water content.

**Optimisation of water content and temperature**

The water content in the foaming process is optimised for each temperature and is given in Table 6. Optimum water content is determined where the HL and ER curves meet, showing optimum ER and HL values (Figure 5) (Xiao et al 2011; Raffaelli 2004). From the results it is observed that, with increasing temperature of the foaming process, the water required to obtain the maximum half-life and expansion ratio decreases. For example, at 120°C, the

---

*(Figure 5 Foaming characteristics of different bitumen types)*
Expansion ratio and half-life are 11 and 15 for both VG 10 and LGB respectively, whereas for VG 30 at the same temperature, foaming is not obtained. For VG 30, the maximum ER and HL is 160°C.

Ageing characteristics of bitumen and the effect on foaming characteristics
Ageing of bitumen occurs during mixing, placement and compaction, as well as during the service life of the road. To simulate the ageing process under short-term and long-term conditions, RTFO ageing and PAV ageing were carried out. The viscosity ratio was determined for identification of the oxidation behaviour of the binder. The viscosity ratio was calculated for RTFO aged at 163°C, RTFO aged at optimum foaming temperature PAV aged, and foamed binder. The viscosity based ageing ratio was calculated using Equation 5. The computed viscosity ageing ratio (VR) for all binders are presented in Figure 6.

The lower temperatures for RTFO were selected in accordance with the temperatures at which the foaming of different binders is optimised, as shown in Table 6. Since foam mix and HMA are subjected to similar conditions in the field, the PAV ageing conditions were not altered and were kept the same for both foam and HMA binders. The viscosity ageing ratio shows that LGB is more prone to ageing when compared to VG 10 and VG 30 through the RTFOT and PAV test. However, ageing indices show similar values for all binders when RTFO is done at optimum binder temperature for foaming. After foaming, the foamed sample is kept still for two hours and water is removed by pouring out; the remaining bitumen is subjected for testing and termed as foamed binder. The viscosity ratio of foam shown in Figure 6 is defined as the ratio of viscosity of virgin binder to the viscosity of foamed binder and is closer to unity, showing a negligible difference in the viscosity of binders before and after foaming.

Rheological properties of bitumen and effect on foaming characteristics
Figure 7 shows the isochronal plots for the different bitumen binders used in this work. From these plots it is observed that, as the frequency increases, the shear complex modulus (G*) also increases. Furthermore, it is also found that the shear complex modulus decreases with increase in temperature.

**Table 6 Optimisation of water content and temperature**

<table>
<thead>
<tr>
<th>Bitumen</th>
<th>Temperature (°C)</th>
<th>Water content (%)</th>
<th>ER (%)</th>
<th>HL (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LGB</td>
<td>120</td>
<td>7</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>130</td>
<td>6</td>
<td>18</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>140</td>
<td>4</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>160</td>
<td>4</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>180</td>
<td>2.5</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td>VG 10</td>
<td>120</td>
<td>9</td>
<td>11</td>
<td>11</td>
</tr>
<tr>
<td></td>
<td>130</td>
<td>8.5</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>140</td>
<td>6.5</td>
<td>14</td>
<td>14</td>
</tr>
<tr>
<td></td>
<td>160</td>
<td>4.5</td>
<td>14</td>
<td>14</td>
</tr>
<tr>
<td>VG 30</td>
<td>120</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>130</td>
<td>10</td>
<td>16</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>140</td>
<td>10</td>
<td>18</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>160</td>
<td>6</td>
<td>19</td>
<td>19</td>
</tr>
<tr>
<td></td>
<td>180</td>
<td>3</td>
<td>18</td>
<td>18</td>
</tr>
</tbody>
</table>

**Table 7 Complex modulus and phase angle values of bitumen**

<table>
<thead>
<tr>
<th>Bitumen types</th>
<th>Temperature (°C)</th>
<th>15°C</th>
<th>45°C</th>
<th>90°C</th>
<th>120°C</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>G* (kPa)</td>
<td>G* (kPa)</td>
<td>δ°</td>
<td>G* (kPa)</td>
<td>δ°</td>
</tr>
<tr>
<td>LGB</td>
<td>241</td>
<td>74.0</td>
<td>72</td>
<td>84.0</td>
<td>0.06</td>
</tr>
<tr>
<td>VG 10</td>
<td>1 770</td>
<td>61.0</td>
<td>23.3</td>
<td>80.4</td>
<td>0.14</td>
</tr>
<tr>
<td>VG 30</td>
<td>2 460</td>
<td>63.0</td>
<td>31.4</td>
<td>81.8</td>
<td>0.12</td>
</tr>
</tbody>
</table>

**Figure 6 Viscosity ratio of aged, un-aged and foam binders**
Table 7 presents the results of $G^*$ and phase angle ($\delta$) with varying temperatures for different bitumen types. It was observed that at temperature 45°C, the bitumen types LGB, VG 10 and VG 30 experience $G^*$ in increasing order, whereas the phase angle ($\delta$) is not substantially different. The substantial difference was observed at low temperatures only. Results indicate that LGB demonstrates more viscous behaviour compared to VG 10 and VG 30, showing a higher phase angle at lower temperatures. At 120°C there is no substantial difference in the $G^*$ and $\delta$ values for all three types of bitumen. Hence, complex modulus and phase angle do not have much effect on foaming characteristics.

It can be seen that all three types of bitumen behave as sol between 45°C to 120°C, as phase angle values are high and comparable, but there is considerable difference in phase angle values at 15°C, which indicate more elastic behaviour of VG 10 and VG 30 compared to LGB bitumen. BBR was used to evaluate low temperature properties of PAV aged binders. Creep stiffness and m-value were obtained from this testing procedure. The BBR test was conducted at different temperatures, i.e. –14°C, –18°C, –20°C, –22°C, –24°C and –28°C. Figures 8(a) and (b) show the stiffness and m-value for all binders at different temperatures, respectively. As per the Asphalt Institute’s Superpave Series No 1 (SP 1), the creep stiffness of specified grade temperature should be less than or equal to 300 MPa at 60 s, with an m-value greater than or equal to 0.3 at 60 s. Hence, the low cracking of LGB, VG 10 and VG 30 has been determined to be –20, –18 and –14°C, respectively. Therefore, LGB is likely to demonstrate better resistance to cracking at low temperature. Due to the sol behaviour of LGB at lower temperatures, it may show better coating during mixing with aggregates.

**Temperature susceptibility of bitumen and its effect on foaming**

Due to the sudden change of bitumen temperature upon its contact with cold water in the foaming process, the temperature susceptibility might have an effect on the foamability and the quality of the produced foam. Therefore three approaches were used to evaluate and characterise this property – penetration index (PI), penetration-viscosity number (PVN) and viscosity temperature susceptibility (VTS) (Roberts et al 1991). Low PI, low PVN,
and high VTS values are indicators of a bitumen that is highly susceptible to temperature changes, and vice versa indicates that the bitumen has lower temperature susceptibility. Variations of PI, PVN and VTS with respect to different grades of bitumen are presented in Figures 9(a), (b) and (c) respectively. Figure 9(a) shows that the PI values are between –1.41 and –1.81. According to Roberts et al (1991), PI values for most good paving bitumen types are between +1 and –1. High temperature susceptibility occurs when the bitumen has a PI less than –2. LGB shows the lowest PI value while VG 30 exhibits the highest PI in the range of 15 to 25°C.

Figure 9(b) shows that all PVN values were between 3.65 and 0.57. According to Roberts et al (1991) most paving bitumen types have a PVN between 0.5 and –2.0. Bitumen class VG 30 showed the lowest PVN value, while LGB exhibited the highest. Thus, VG 30 is the most temperature-susceptible bitumen and LGB is the least temperature-susceptible bitumen. Therefore, LGB exhibits the highest resistance to cracking at a low temperature range compared to VG 30 and VG 10. Figure 8(c) shows that all VTS values are between 3.69 and 2.92. LGB bitumen shows the highest value, while VG 30 exhibits the lowest. Thus, LGB is the most temperature-susceptible bitumen at the temperature range of 100°C to 135°C.

Foaming of bitumen is done at higher temperature ranges, i.e 120°C to 180°C. At 120°C, VG 30 could not produce acceptable foam. Hence, it can be correlated that, due to high temperature-susceptibility, LGB shows better foaming characteristics at 120°C compared to VG 10 and VG 30. This is due to a decrease in viscosity of LGB at 120°C, compared to VG 10 and VG 30 showing better foaming at lower temperatures.

It is obvious that VG 30 is the most temperature-susceptible bitumen according to the results obtained from PI and
PVN, and produces unacceptable foam at lower temperature. VG 10, which is also comparatively less temperature-susceptible, shows reasonable foaming characteristics similar to those produced by LGB (which is the least temperature-susceptible bitumen). The conclusion is that the use of temperature-susceptible bitumen types at temperatures below 120°C does not have a direct effect on the foaming properties. Many literature studies are available supporting such results (Saleh 2007; Iwánski et al. 2015; Roberts 1991; Pengcheng & John 2007).

**Effect of viscosity of bitumen on foaming characteristics**

To check the effect of viscosity on ER and HL, numerical values of ER and HL were plotted against viscosity. Plots of ER vs viscosity, and HL vs viscosity are presented in Figures 10 and 11, respectively. These plots account for all grades of binder together. A best-fit curve was determined for these values while keeping the water content constant. The equations thus obtained are presented in Equations 6 and 7 for ER and HL, respectively. The regression coefficients and \( R^2 \) values for different water content are tabulated in Table 8.

\[
ER = A \times \ln(\eta) + B \tag{6}
\]

\[
HL = A \ln(\eta) + B \tag{7}
\]

Where: A and B are coefficients, and \( \eta \) is the viscosity of bitumen at foaming temperature.

In general, ER vs viscosity, and HL vs viscosity exhibited negative exponential and logarithmic trends, respectively. The correlation coefficient was lower than 0.6 for all cases. As specified by TG2 (Asphalt Academy 2009), the minimum criteria of ER and HL are chosen to be 8 and 6 s. Figure 10 shows that for an expansion ratio greater than or equal to 8, viscosity must be lower than 700 Poise at foaming temperature. Similarly, it can be concluded from Figure 11 that viscosity must be greater than 250 Poise for half-life greater than or equal to 6 s. Hence, the optimum range of viscosity for acceptable foaming characteristics was found to be between 250 and 700 Poise. A bitumen having viscosity less than 250 Poise shows better expansion ratio, but foam is unstable due to a lower half-life. It is a well-known fact that the viscosity of bitumen decreases with increase in temperature, and subsequently the thinning effect of the bitumen film of bubbles leads to reduced HL. Similar observations on reduced HL with lower viscosity have been made by other researchers (Saleh 2007; Wirtgen 2004).

**Effect of chemical composition on foaming characteristics**

The foaming effects on binder chemical composition using Fourier transform infrared spectroscopy and X-Ray radiography techniques have been investigated by various authors (Namutebi et al. 2011; Haileslassie et al. 2015), but the effect of binder structure on foaming has not been studied yet. Figure 12 shows the sulphoxide, carbonyl, aromatics and aliphatic

![Figure 10 Effect of viscosity on expansion ratio](image1)

![Figure 11 Effect of viscosity on half-life](image2)

**Table 8 Coefficient and \( R^2 \) values**

<table>
<thead>
<tr>
<th>Equation no</th>
<th>Foaming water content (%)</th>
<th>Coefficient</th>
<th>( R^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>5.1</td>
<td>10</td>
<td>-78</td>
<td>6.61</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>-5.5</td>
<td>4.69</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>-3.1</td>
<td>2.82</td>
</tr>
<tr>
<td>4.2</td>
<td>10</td>
<td>7.6</td>
<td>0.001</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>11.6</td>
<td>0.001</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>16.3</td>
<td>0.001</td>
</tr>
</tbody>
</table>
fraction of all the binders. Asphaltene contains some nitrogen, sulphur and oxygen in addition to carbon and hydrogen. Sulphoxide and carbonyl content increases with increase in asphaltene content, hence results in an increase in the viscosity of the binder. Within the bitumen fluid system, sulphoxide and carbonyl components act as surfactants (Hung et al. 2012). Surfactants, sulphoxide and carbonyl reduce the surface tension in the surface of the bubbles and hence collapse of the foam is delayed. For the same reason, a decrease in ER was observed with increasing sulphoxide and carbonyl content. It can be correlated that with an increase in carbonyl and sulphoxide compound, the expansion ratio decreases and half-life increases.

**Strength characteristics of BSM**

The optimum moisture content (OMC) of this untreated blend was found to be 6.2% by using the moisture-density relationships according to AASHTO T180 specifications. Figures 13(a) and (b) show the variation of dry ITS and wet ITS, respectively, with respect to binder quantity and type. For a particular foamed bitumen content, a higher ITS value was observed with mixtures prepared with a higher viscosity grade. This can be attributed to the higher stiffness provided by the bitumen. A mix prepared with LGB gives more than the required ITS values in both dry and wet conditions (dry ITS > 225 kPa and wet ITS > 100 kPa), which show the effective binding properties of LGB in mix design consideration.

**CONCLUSION**

There is an increasing trend worldwide to use foamed bitumen. Most of the studies reporting on foamed bitumen have focused on the effect of water content and bitumen temperature during foaming. The conclusions drawn from the laboratory study are as follows:

1. The temperature susceptibilities of the different types of bitumen were investigated using VTS, PVN and PI. According to PVN and PI, VG 30 is most temperature-susceptible in the temperature range 15°C to 60°C compared to VG 10 and LGB. But at higher temperatures, LGB shows the highest temperature-susceptibility. Use of temperature-susceptible bitumen at temperatures below 100°C does not have a direct effect on the foaming properties. LGB is the least temperature-susceptible bitumen. Therefore LGB exhibits the highest resistance to cracking at a low temperature range compared to VG 30 and VG 10.

2. Foaming is done above 120°C. At 120°C and higher temperatures there is no substantial difference in the values of $G^*$ and $\delta$ for all six binders. Hence, complex modulus and phase angles have little effect on foaming characteristics.
3. From various tests and analyses it has been found that foaming characteristics depend on the viscosity of bitumen, along with the foaming temperature and foamed water content.

4. The performance of foamed bitumen depends not only on the rheology of bitumen, but also on the chemical composition of bitumen. From FTIR results it has been concluded that, with an increase in carbonyl and sulphoxide compound, the expansion ratio decreases and half-life increases, due to decrease in surface tension of the binder.

5. Viscosity of bitumen is an important factor. Foaming temperature may be decided on the basis of viscosity value. The optimum range of viscosity for acceptable foaming characteristics is found to be between 250 and 800 Poise.

6. LGB consumes low energy during the foaming process compared to high viscosity bitumen. If LGB is used in place of VG 30 bitumen for foaming technology, foam can be produced at 40°C lower, which can result in a huge saving in energy consumption and greenhouse gas emissions.

ACKNOWLEDGEMENTS

The authors are thankful to the Director, CSIR—Central Road Research Institute, New Delhi 110025, for permission to publish this paper. The authors would also like to thank Shri Satish Pandey (Scientist, CSIR—Central Road Research Institute) and Ms Writgen for their support during the installation of the WLBS 10 Foaming Plant.

REFERENCES


Determination of base and shaft resistance factors for reliability-based design of piles


This paper aims to propose a procedure for calculating separately the resistance factors for ultimate base and shaft resistances for the reliability-based design of piles. The proposed procedure can clearly explain the different sources of uncertainties of the bearing capacity, including those from ultimate base and shaft resistances. The study evaluates the convergence of the proposed procedure, and the effects of relevant parameters on resistance factors. Finally, two examples are used for comparison and application of the presented method for determining ultimate base and shaft resistance factors. Convergence analysis proves that the final resistance factors can be rapidly obtained, and maintain good stability using the iteration algorithm included in the proposed procedure. A parametric study indicates that the ratio of ultimate shaft resistance factors is conservative compared to the results by Kim et al. (2011), due to the consideration of more uncertainties. The recommended ultimate shaft and base resistance factors for the reliability-based design of piles can be obtained conveniently using the proposed procedure in the application example.

NOTATION
The following symbols are used in this paper:

- \( b \): ratio of \( R_{ult,b} \) to \( (L_D + L_L) \)
- \( \text{COV}_{R_{ult,b}} \): coefficient of variation for \( R_{ult,b} \)
- \( \text{COV}_{R_{ult,s}} \): coefficient of variation for \( R_{ult,s} \)
- \( \text{COV}_{\lambda_D} \): coefficient of variation for \( \lambda_D \)
- \( \text{COV}_{\lambda_L} \): coefficient of variation for \( \lambda_L \)
- \( \theta \): load factor for live load
- \( \lambda_D \): bias factor for \( L_D \)
- \( \lambda_L \): bias factor for \( L_L \)
- \( \lambda_{ult,b} \): bias factor for \( R_{ult,b} \)
- \( \lambda_{ult,s} \): bias factor for \( R_{ult,s} \)
- \( \rho \): ratio of \( L_D \) to \( L_L \)
- \( \phi_{ult,b} \): resistance factor for \( R_{ult,b} \)
- \( \phi_{ult,s} \): resistance factor for \( R_{ult,s} \)
- \( \phi_{ult,s}^{(0)} \): initial value of \( \phi_{ult,s} \)
- \( \gamma_L \): limit value of calculation accuracy
- \( \delta \): limit state function
- \( L \): load
- \( L_D \): dead load
- \( L_L \): live load
- \( P_f \): failure probability
- \( R_{ult} \): ultimate resistance of pile
- \( R_{ult,b} \): ultimate base resistance of pile
- \( R_{ult,s} \): ultimate shaft resistance of pile
- \( s \): ratio of \( R_{ult,s} \) to \( (L_D + L_L) \)
- \( \beta \): reliability index
- \( \beta_T \): target reliability index
- \( \gamma_D \): load factor for dead load

INTRODUCTION
Load and resistance factor design (LRFD) is conceptually a more advanced design method than the existing working stress design (WSD). The key improvements of LRFD over the traditional WSD are the ability to provide a more consistent level of reliability and the possibility of accounting for load and resistance uncertainties separately (Foye et al. 2006). Successful implementation of LRFD
in geotechnical engineering contributes to an economical and safe design.

Many researchers and practitioners are now recognising the great advantages of LRFD in practice, and more and more relevant research is being incorporated into LRFD for driven piles based on reliability analysis (Zhang et al 2001; Paikowsky 2004; AASHTO 2007). Many countries and regions, such as the United States, Canada, South Africa, China mainland, Japan, Korea, Singapore, Europe and Hong Kong, are replacing or have already replaced WSD with LRFD for structural design. However, LRFD in geotechnical engineering has not been fully developed yet (Kim et al 2011).

Against this background, a rational framework for LRFD development should be established for the replacement of resistance factors calculated based on factors of safety with those calculated based on reliability analysis. The LRFD framework is conducive to maintaining the same levels of load factors for all loads under different conditions. A number of studies have looked at calculating and calibrating resistance factors for geotechnical engineering. Zheng et al (2012) presented a Bayesian optimisation approach to determine the resistance factor of piles, and recommended values for the resistance factors of driven piles. Bian et al (2015) incorporated the serviceability limit state requirements into LRFD for the ultimate limit states of piles to determine the resistance factors for reliability-based design of piles. Phoon and Kulhawy (2002), and Phoon et al (2003) proposed a multiple resistance factor design concept for foundations and studied the uplift resistance factors for uplift side resistance, uplift tip resistance and dead weight of foundation against uplift force. Honjo et al (2002) established a procedure for the calculation of partial factors for dead load, seismic load, base resistance and shaft resistance of axially-loaded cast-in situ piles. Kim et al (2011) contributed to the development of LRFD for axially-loaded driven piles in sands, the evident feature of which is that the resistance factors for base and shaft resistances were calculated separately to account for their different uncertainty levels. Basu and Salgado (2012) developed resistance factors for drilled shafts for a design method based on soil parameters.

However, methods to determine the ultimate base and shaft resistance factors are not well developed. This paper will present a novel method to calculate the ultimate base and shaft resistance factors for the reliability-based design of piles. First, an iterative algorithm to estimate the ultimate base and shaft resistance factors will be presented using the reliability theory and LRFD criteria. Second, the convergence of the proposed procedure will be analysed. Third, the effects of relevant parameters – ratio of dead load to live load, initial value of base (or shaft) resistance factor, and target reliability index – on resistance factors will be evaluated. Finally, the validation and practical application of the presented method will be shown with two examples to illustrate the feasibility and availability of the presented method.

**LOAD AND RESISTANCE FACTOR DESIGN**

**Design criterion**

Considering an axially-loaded driven pile, the ultimate pile resistance (or bearing capacity) \( R_{\text{ult}} \) is generally expressed as the summation of ultimate base resistance (or end resistance) \( R_{\text{ult},b} \) and ultimate shaft resistance (or shaft friction) \( R_{\text{ult},s} \) (Poulos & Davis 1980):

\[
R_{\text{ult}} = R_{\text{ult},b} + R_{\text{ult},s} \quad (1)
\]

The key advantage of the LRFD approach is that significant uncertainties (e.g. load and material resistance) can be incorporated quantitatively into the design process. If only dead load \( L_D \) and live load \( L_L \) are considered, the LRFD design formula for an axially-loaded driven pile can be written as (AASHTO 2007):

\[
\phi_{\text{ult},b} R_{\text{ult},b} + \phi_{\text{ult},s} R_{\text{ult},s} \geq \gamma_D L_D + \gamma_L L_L \quad (2)
\]

where \( \phi_{\text{ult},b} \) and \( \phi_{\text{ult},s} \) are the resistance factors for \( R_{\text{ult},b} \) and \( R_{\text{ult},s} \) respectively; and \( \gamma_D \) and \( \gamma_L \) are the specified load factors for dead and live loads respectively.

**Resistance factors**

Suppose resistance \( R_{\text{ult}} \) and load \( L \) follow lognormal distribution and they are statistically independent (Ang & Tang 2007; Wang & Kulhawy 2008; Dithinde et al 2011). It should be pointed out here that the probability distribution for load \( L \) is certainly suitable to dead load \( L_D \) and live load \( L_L \), namely lognormal distribution. The limit state function \( (g) \) in accordance with LRFD framework is established:

\[
g = \ln(R_{\text{ult}}) - \ln(L) \quad (3)
\]

The reliability index \( \beta \), which is used to estimate the reliability of piles and reflect the safety status of piles, can be calculated using the following formula (Federal Highway Administration 2001; Bian et al 2016):

\[
\beta = \frac{\ln\left[\phi_{\text{ult},b} R_{\text{ult},b} + \phi_{\text{ult},s} R_{\text{ult},s} \sqrt{1 + \text{COV}_L^2} \right]}{\sqrt{\ln(1 + \text{COV}_{R_{\text{ult},b}}^2 + \text{COV}_{R_{\text{ult},s}}^2) + \ln(1 + \text{COV}^2_{L_D} + \text{COV}^2_{L_L})}} \quad (4)
\]

where \( \phi_{\text{ult},b} \) and \( \phi_{\text{ult},s} \) are the bias factors for \( R_{\text{ult},b} \) and \( R_{\text{ult},s} \) respectively; and \( \text{COV}_{R_{\text{ult},b}} \), \( \text{COV}_{R_{\text{ult},s}} \), \( \text{COV}_{L_D} \) and \( \text{COV}_{L_L} \) are the coefficients of variation (COVs) for \( R_{\text{ult},b} \), \( R_{\text{ult},s} \), \( L_D \) and \( L_L \) respectively. Here the bias factor includes the net effect of various sources of errors, such as inherent soil variability, measurement error, and transformation uncertainty.

LRFD is the limit state design (mainly including ultimate limit state and serviceability limit state for piles), and only the ultimate limit state requirements are focused on in this paper. As the critical state of the design formula (Equation 2) is necessary for the study, replacing the inequality sign in Equation 2 with an equality sign gives:

\[
\phi_{\text{ult},b} R_{\text{ult},b} + \phi_{\text{ult},s} R_{\text{ult},s} = \gamma_D L_D + \gamma_L L_L \quad (5)
\]

Then \( R_{\text{ult},b} \) and \( R_{\text{ult},s} \) can be expressed respectively as:

\[
R_{\text{ult},b} = \frac{\gamma_D L_D + \gamma_L L_L - \phi_{\text{ult},s} R_{\text{ult},s}}{\phi_{\text{ult},b}} \quad (6)
\]

\[
R_{\text{ult},s} = \frac{\gamma_D L_D + \gamma_L L_L - \phi_{\text{ult},b} R_{\text{ult},b}}{\phi_{\text{ult},s}} \quad (7)
\]

Substituting \( R_{\text{ult},b} \) and \( R_{\text{ult},s} \) expressed respectively by Equations 6 and 7, into Equation 4, and replacing \( \beta \) in Equation 4 with target
reliability index ($\beta_T$), gives the following expressions for $\phi_{ult,b}$ and $\phi_{ult,s}$:

$$\phi_{ult,b} = \frac{\lambda_{ult,b} \left[ \frac{\gamma_D \rho + \gamma_L}{\rho + 1} - \phi_{ult,s}^b \right]}{\lambda_D \rho + \lambda_L} \times \exp \left[ \beta_T \ln \left( \frac{1 + \text{COV}^2_{ult,b} + \text{COV}^2_{ult,s}}{\sqrt{1 + \text{COV}^2_{ult,b} + \text{COV}^2_{ult,s}}} \right) \right] - \lambda_{ult,s}^b \left( \frac{1 + \text{COV}^2_{ult,b} + \text{COV}^2_{ult,s}}{\sqrt{1 + \text{COV}^2_{ult,b} + \text{COV}^2_{ult,s}}} \right)$$

$$\phi_{ult,s} = \frac{\lambda_{ult,s} \left[ \frac{\gamma_D \rho + \gamma_L}{\rho + 1} - \phi_{ult,b}^s \right]}{\lambda_D \rho + \lambda_L} \times \exp \left[ \beta_T \ln \left( \frac{1 + \text{COV}^2_{ult,b} + \text{COV}^2_{ult,s}}{\sqrt{1 + \text{COV}^2_{ult,b} + \text{COV}^2_{ult,s}}} \right) \right] - \lambda_{ult,b}^s \left( \frac{1 + \text{COV}^2_{ult,b} + \text{COV}^2_{ult,s}}{\sqrt{1 + \text{COV}^2_{ult,b} + \text{COV}^2_{ult,s}}} \right)$$

in Equations 8 and 9 $\rho = \frac{L_D}{L_L}$, $s = \frac{R_{ult,s}}{R_{ult,b}}$, and $b = \frac{R_{ult,b}}{L_D + L_L}$.

Equations 8 and 9 indicate that $\phi_{ult,b}$ and $\phi_{ult,s}$ are functions of many parameters, such as $\rho$, $\beta_T$, $s$, $b$ and so on. Among these parameters $\rho$, $\beta_T$, $s$, $b$ are key factors influencing $\phi_{ult,b}$ and $\phi_{ult,s}$ due to great uncertainties for them. Here $\beta_T$ is a certain level of reliability, for which piles designed using the LRFD method will guarantee. In other words, the reliability index of a pile designed using LRFD is greater than or equal to the target reliability index.

**Probabilistic parameters**

Based on the foregoing discussion, there are two sets of information required to estimate resistance factors: load and resistance information (including load factor, bias factor, COV). A review of literature (AASHTO 2007) suggests that the following probabilistic parameters can be used for $L_D$ and $L_L$: $\lambda_D = 1.08$, $\text{COV}_{\lambda_D} = 0.13$, $\lambda_L = 1.15$, $\text{COV}_{\lambda_L} = 0.18$, $\gamma_D = 1.25$ and $\gamma_L = 1.75$. $\rho = \frac{L_D}{L_L}$ is structure-specific and changes with span length (Hansell et al 1971; Withiam et al 2001). Hansell et al (1971) also proposed an empirical formula, $L_D/L_L = (1 + l)(0.0132l)$, to relate $L_D/L_L$ with span length, where $l$ is the dynamic load factor (taken as 0.33 for LRFD loads), and $l$ is the span length in feet. $\rho = \frac{L_D}{L_L}$ spreads over from 0.576 to 5.184 when $l$ varies from 10 m to 90 m, and $\rho = 3.0$ is frequently used value.

Formula $p_f = \Phi(-\beta)$ expresses the relationship between failure probability ($p_f$) and $\beta$ (see Table 1). The acceptable $\beta_T$ is in essence the maximum acceptable failure probability. For example, determining acceptable $\beta_T = 3.0$ means the acceptable maximum failure probability is 0.001.

Barker et al (1991) reduced the target reliability index for driven piles to a value between 2.0 and 2.5, especially for a group system effect. Paikowsky (2004) suggested an initial target reliability index between 2.0 and 2.5 for a pile group, and 3.0 for a single pile. Paikowsky (2004) also recommended target reliability indices of 2.33 (corresponding to 1% probability of failure) and 3.00 (corresponding to 0.1% probability of failure) for representing redundant and non-redundant pile groups, respectively. As suggested by Barker et al (1991) and Paikowsky (2004), five levels (2.0, 2.5, 3.0, 3.5 and 4.0) of target reliability index will be considered in this study and the corresponding resistance factors calculated.

Probabilistic parameters for $R_{ult}$, $R_{ult,b}$ and $R_{ult,s}$ from literature are summarised in Table 2. Equations 8 and 9 demonstrate that $b (= \frac{R_{ult,b}}{(L_D + L_L)})$ and $s (= \frac{R_{ult,s}}{(L_D + L_L)})$ are two key parameters for evaluation of resistance factors. However, both $b$ and $s$ are difficult to determine, because $R_{ult,b}$ and $R_{ult,s}$ depend largely on site conditions and pile types. For example, $R_{ult,b}$ of friction piles is generally very small and may be ignored with respect to $R_{ult,s}$ which means that $R_{ult,s} \approx R_{ult,b}$ resulting in $b = 0$ and $s = \frac{R_{ult,s}}{(L_D + L_L)}$. Moreover, for a safe design $s = R_{ult,s}/(L_D + L_L) \geq 1.0$. For end-bearing piles $R_{ult,s}$ is generally very small, and may be ignored with respect to $R_{ult,b}$ which means that $R_{ult,b} \approx R_{ult,b}$ resulting in $s = 0$ and $b = \frac{R_{ult,b}}{(L_D + L_L)}$. Moreover, for a safe design $b = R_{ult,b}/(L_D + L_L) \geq 1.0$. For end-bearing friction piles and friction end-bearing piles, $b$ and $s$ are complex and need further study.

**PROCEDURE TO CALCULATE RESISTANCE FACTORS**

**Procedure flow chart**

Equations 8 and 9 indicate that $\phi_{ult,b}$ and $\phi_{ult,s}$ are functions of many parameters, such as $\rho$, $\beta_T$, $s$, $b$ and other parameters. Especially, $\phi_{ult,b}$ computed using Equation 8 will be submitted into Equation 9 to compute $\phi_{ult,s}$ and this $\phi_{ult,s}$ will be resubmitted into Equation 8 to compute $\phi_{ult,b}$ again. This computation process is in fact an iteration process, which contributes to build a procedure for resistance factors calculation, as shown in Figure 1 (Bian et al 2016). In Figure 1, $\phi_{ult,s}^{(0)}$ is

<table>
<thead>
<tr>
<th>$\beta$</th>
<th>1.0</th>
<th>1.5</th>
<th>2.0</th>
<th>2.5</th>
<th>2.33</th>
<th>3.0</th>
<th>4.0</th>
<th>5.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>$p_f$</td>
<td>0.16</td>
<td>0.07</td>
<td>0.023</td>
<td>0.006</td>
<td>0.001</td>
<td>0.00003</td>
<td>0.000003</td>
<td></td>
</tr>
</tbody>
</table>

Table 1 Relationship between reliability index and failure probability

<table>
<thead>
<tr>
<th>Resistance type</th>
<th>Bias factor</th>
<th>COV</th>
<th>Distribution type</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R_{ult}$</td>
<td>1.58</td>
<td>0.339</td>
<td>Lognormal</td>
<td>Paikowsky (2004)</td>
</tr>
<tr>
<td></td>
<td>2.0 (mean)</td>
<td>0.194</td>
<td>Normal</td>
<td>Michiyö et al (1993)</td>
</tr>
<tr>
<td>$R_{ult,b}$</td>
<td>1.023</td>
<td>0.201</td>
<td>Lognormal</td>
<td>Jardine et al (2005)</td>
</tr>
<tr>
<td>$R_{ult,s}$</td>
<td>1.088</td>
<td>0.287</td>
<td>Not stated</td>
<td></td>
</tr>
</tbody>
</table>
the initial value of $\phi_{ult,s}$ where $\theta$ is the limit value of calculation accuracy.

With reference to Figure 1, the proposed approach to determine resistance factors $\phi_{ult,b}$ and $\phi_{ult,s}$ for reliability-based design of piles is outlined in the following steps:

■ **Step 1** Input statistical parameters of load and resistance. Bias factors, COVs and load factors $L_D$ and $L_L$ can be determined referring to the previous literature. Bias factors and COVs of $R_{ult,b}$ and $R_{ult,s}$ are estimated using the load test database of piles.

■ **Step 2** Input combination parameters $p = L_D/L_L$, $s = R_{ult,s}/(L_D + L_L)$ and $b = R_{ult,b}/(L_D + L_L)$. For $p$, the commonly used values (such as 0.5, 1.0, 2.0, 3.0, 4.0 and 5.0) from the previously mentioned literature are welcome, while $s$ and $b$ need to be evaluated depending on site conditions and pile types.

■ **Step 3** Input $\beta_T$ and precision limit value $\theta$. For $\beta_T$, the commonly used values (such as 2.0, 2.5, 3.0, 3.5 and 4.0) will be accepted for further study. $\theta$ is set as 0.0001.

■ **Step 4** Determine initial value $\phi_{(0)}^{ult,b}$ (or $\phi_{(0)}^{ult,s}$). Six values, namely 0.0, 0.2, 0.4, 0.6, 0.8 and 1.0, will be suggested to study the influence of $\phi_{(0)}^{ult,b}$ (or $\phi_{(0)}^{ult,s}$) on the final results.

■ **Step 5** Calculate $\phi_{ult,b}$ and $\phi_{ult,s}$. Submit $\phi_{(0)}^{ult,b}$ into Equation 8 to compute $\phi_{ult,b}$, and denote as $\phi_{(1)}^{ult,b}$. Submit $\phi_{(0)}^{ult,b}$ into Equation 9 to compute $\phi_{ult,s}$ and denote as $\phi_{(1)}^{ult,s}$. Repeat this process to obtain $\phi_{(2)}^{ult,b}$ and $\phi_{(2)}^{ult,s}$.

■ **Step 6** Examine convergence. If $\phi_{(2)}^{ult,b}$ and $\phi_{(2)}^{ult,s}$ satisfy $|\phi_{(2)}^{ult,b} - \phi_{(1)}^{ult,b}| \leq \theta$, and $|\phi_{(2)}^{ult,s} - \phi_{(1)}^{ult,s}| \leq \theta$, $\phi_{(2)}^{ult,b}$ and $\phi_{(2)}^{ult,s}$ are taken as the final resistance factors $\phi_{ult,b}$ and $\phi_{ult,s}$. If $\phi_{(2)}^{ult,b}$ and $\phi_{(2)}^{ult,s}$ do not satisfy $|\phi_{(2)}^{ult,b} - \phi_{(1)}^{ult,b}| \leq \theta$ and $|\phi_{(2)}^{ult,s} - \phi_{(1)}^{ult,s}| \leq \theta$, repeat Step 5.

### Convergence analysis of calculation procedure

The validity and application conditions of the procedure are investigated in depth in this section. All related computation tasks will be completed using MS Excel. Convergence of the proposed procedure to calculate resistance factors was made using the following parameters: $p = 3.0$, $\phi_{ult,s} = 0.5$, $\beta_T = 3.0$ and $\theta = 0.0001$. Parameter $s$ was set as 0.5, 1.0, 2.0 and 3.0. For each value of $s$, $b$ was set as 0, 0.5, 1.0, 2.0, 3.0 and integer ≥ 4.0, respectively. Here statistics of $L_D$ and $L_L$ (namely $\lambda_D = 1.08$, $\gamma_D = 1.25$ and $\gamma_L = 1.75$) are provided by AASHTO (2007) and used in this section. Final resistance factors obtained for each combination and the number of iterations required to reach the set level of accuracy ($\theta = 0.0001$) are summarised in Table 2.

In Table 3, NM indicates that the combination ($b = 0$ and $s = 0.5$) is meaningless for the reliability-based design of piles, while NRC indicates that the iterative process could not reach convergence. Employing the information from Table 3, a bold judgement can be made that the iterative process proposed in this paper reached convergence only when the sum of $s$ and $b$ was less than 4. This requirement meets the demand for reliability-based design of piles satisfactorily. The LRFD criteria for piles, the most important limit state design method, has been expressed by Equation 2. For reliability-based design of piles, $R_{ult}$ does not excessively exceed the load effects. Due to this, and given that safe designs are those with $R_{ult}/(L_D + L_L) \geq 1.0$, $b = (R_{ult,b}/L_D + L_L)$ and $s = (R_{ult,s}/(L_D + L_L)$ will be limited to between 0 and 3.0. It is also pointed out that runs with $b = 0$ are done purely for comparison, as $b = 0$ implies a pure friction pile which is not possible.

From Table 3, the following preliminary conclusions can be drawn: for a given $s$, the number of iterative steps increases with increasing $b$, as shown from columns 10 to

---

**Figure 1 Flowchart of resistance factor calculation**

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Input statistical parameters of load and resistance. Bias factors, COVs and load factors $L_D$ and $L_L$ can be determined referring to the previous literature. Bias factors and COVs of $R_{ult,b}$ and $R_{ult,s}$ are estimated using the load test database of piles.</td>
</tr>
<tr>
<td>2</td>
<td>Input combination parameters $p = L_D/L_L$, $s = R_{ult,s}/(L_D + L_L)$ and $b = R_{ult,b}/(L_D + L_L)$. For $p$, the commonly used values (such as 0.5, 1.0, 2.0, 3.0, 4.0 and 5.0) from the previously mentioned literature are welcome, while $s$ and $b$ need to be evaluated depending on site conditions and pile types.</td>
</tr>
<tr>
<td>3</td>
<td>Input $\beta_T$ and precision limit value $\theta$. For $\beta_T$, the commonly used values (such as 2.0, 2.5, 3.0, 3.5 and 4.0) will be accepted for further study. $\theta$ is set as 0.0001.</td>
</tr>
<tr>
<td>4</td>
<td>Determine initial value $\phi_{(0)}^{ult,b}$ (or $\phi_{(0)}^{ult,s}$). Six values, namely 0.0, 0.2, 0.4, 0.6, 0.8 and 1.0, will be suggested to study the influence of $\phi_{(0)}^{ult,b}$ (or $\phi_{(0)}^{ult,s}$) on the final results.</td>
</tr>
<tr>
<td>5</td>
<td>Calculate $\phi_{ult,b}$ and $\phi_{ult,s}$. Submit $\phi_{(0)}^{ult,b}$ into Equation 8 to compute $\phi_{ult,b}$, and denote as $\phi_{(1)}^{ult,b}$. Submit $\phi_{(0)}^{ult,b}$ into Equation 9 to compute $\phi_{ult,s}$ and denote as $\phi_{(1)}^{ult,s}$. Repeat this process to obtain $\phi_{(2)}^{ult,b}$ and $\phi_{(2)}^{ult,s}$.</td>
</tr>
<tr>
<td>6</td>
<td>Examine convergence. If $\phi_{(2)}^{ult,b}$ and $\phi_{(2)}^{ult,s}$ satisfy $</td>
</tr>
</tbody>
</table>

---

**Table 1** Input bias factors, COVs and load factors of $L_D$ and $L_L$
For example, with $s = 0.5$, the iterative steps are 4, 5, 7 and 20 corresponding to $b$ with values of 0.5, 1.0, 2.0 and 3.0 respectively; with $s = 2.0$, the iterative steps are 3, 8 and 15 corresponding to $b$ with values of 0, 0.5 and 1.0 respectively.

**PARAMETER ANALYSIS AND DISCUSSION**

In this section, load statistical parameters (including $\lambda_D = 1.08$, $\text{COV}_{\lambda_D} = 0.13$, $\lambda_L = 1.15$, $\text{COV}_{\lambda_L} = 0.18$, $\gamma_D = 1.25$ and $\gamma_L = 1.75$) presented by AASHTO (2007), and resistance statistical parameters (including $\lambda_{ult,b} = 1.023$, $\lambda_{ult,s} = 1.088$, $\text{COV}_{R_{ult,b}} = 0.201$ and $\text{COV}_{R_{ult,s}} = 0.287$) presented by Jardine et al (2005) are used to compute the resistance factors $\phi_{ult,b}$ and $\phi_{ult,s}$.

**Effect of $\rho$ on resistance factors**

To study the effect of $\rho$ on $\phi_{ult,b}$ and $\phi_{ult,s}$, the following parameters were kept constant: $\phi_{(0)ult,s} = 0.5$, $s = 1.0$, $b = 1.0$, $\beta_T = 3.0$ and $\theta = 0.0001$. It is also well reasoned to set $\rho$ as 0.5, 1.0, 2.0, 3.0, 4.0 and 5.0 respectively. Using these parameters, $\phi_{ult,b}$ and $\phi_{ult,s}$ were calculated using the proposed procedure and plotted against $\rho$ in Figure 2.

Varying $\rho$ did not influence convergence, with convergence generally obtained within six iterative steps. It can be seen from Figure 2 that both $\phi_{ult,b}$ and $\phi_{ult,s}$ decrease slightly with increasing $\rho$. However, the difference between $\phi_{ult,b}$ and $\phi_{ult,s}$ is an approximate constant for all $\rho$. Under the given assumptions, $\phi_{ult,b}$ is larger than $\phi_{ult,s}$ and the average difference between $\phi_{ult,b}$ and $\phi_{ult,s}$ is about 0.025.

**Effect of $\phi_{(0)ult,s}$ on resistance factors**

To study the effect of $\phi_{(0)ult,s}$ on final resistance factors ($\phi_{ult,b}$ and $\phi_{ult,s}$), the following parameters were kept constant: $\rho = 3.0$, $s = 1.0$, $b = 1.0$, $\beta_T = 3.0$ and $\theta = 0.0001$. Parameter $\phi_{(0)ult,s}$ was set as 0, 0.2, 0.4, 0.6, 0.8 and 1.0 respectively, and the two resistance factors were calculated and plotted against $\phi_{(0)ult,s}$ in Figure 3.

Varying $\phi_{(0)ult,s}$ did not influence convergence significantly, with convergence

![Figure 2](image_url)

**Figure 2** Resistance factors $\phi_{ult,b}$ and $\phi_{ult,s}$ of driven piles versus $\rho$

![Figure 3](image_url)

**Figure 3** Resistance factors $\phi_{ult,b}$ and $\phi_{ult,s}$ of driven piles versus initial resistance factor $\phi_{(0)ult,s}$

<table>
<thead>
<tr>
<th>$\phi_{ult,b}$</th>
<th>$\phi_{ult,s}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$s = 0.5$</td>
<td>$s = 1.0$</td>
</tr>
<tr>
<td>$b = 0$</td>
<td>NM 0.3678</td>
</tr>
<tr>
<td>$b = 0.5$</td>
<td>0.3678</td>
</tr>
<tr>
<td>$b = 1.0$</td>
<td>0.3678</td>
</tr>
<tr>
<td>$b = 2.0$</td>
<td>0.3678</td>
</tr>
<tr>
<td>$b = 3.0$</td>
<td>0.3678</td>
</tr>
<tr>
<td>$b \geq 4.0$</td>
<td>NRC</td>
</tr>
</tbody>
</table>

**Note:** NM indicates that this combination of $b$ and $s$ are meaningless; NRC indicates that iterative process could not reach convergence.
reached within no more than seven iterative steps. It can be seen from Figure 3 that both \( \phi_{\text{ult,b}} \) and \( \phi_{\text{ult,s}} \) versus \( \phi^{(0)}_{\text{ult,s}} \) are approximate horizontal lines, which illustrate that convergence values \( \phi_{\text{ult,b}} \) and \( \phi_{\text{ult,s}} \) are both independent of \( \phi^{(0)}_{\text{ult,s}} \), as \( \phi_{\text{ult,b}} \) and \( \phi_{\text{ult,s}} \) are determined as 0.37 and 0.39 respectively. This conclusion provides support to the rationality of the proposed procedure for resistance factor calculation.

**Effect of \( \beta_T \) on resistance factors**

To study the effect of \( \beta_T \) on \( \phi_{\text{ult,b}} \) and \( \phi_{\text{ult,s}} \), the following parameters were kept constant: \( \rho = 3.0, \phi^{(0)}_{\text{ult,s}} = 0.5, s = 1.0, b = 1.0 \) and \( \theta = 0.0001 \). \( \beta_T \) was set as 2.0, 2.5, 3.0, 3.5 and 4.0. Calculated values of \( \phi_{\text{ult,b}} \) and \( \phi_{\text{ult,s}} \) are plotted against \( \beta_T \) in Figure 4.

In Figure 4, both \( \phi_{\text{ult,b}} \) and \( \phi_{\text{ult,s}} \) decrease sharply with an increase of \( \beta_T \), which shows that both \( \phi_{\text{ult,b}} \) and \( \phi_{\text{ult,s}} \) are very sensitive to \( \beta_T \). For example, when \( \beta_T \) increases from 2.0 to 4.0, \( \phi_{\text{ult,b}} \) decreases from 0.55 to 0.25, and \( \phi_{\text{ult,s}} \) decreases from 0.59 to 0.26. Varying \( \beta_T \) had a significant effect on iterative steps. For \( \beta_T = 2.0 \), the iterative steps were 12; for \( \beta_T = 2.5, 3.0 \) and 3.5 respectively, the iterative steps were all nearly 6; and for \( \beta_T = 4.0 \), the iterative steps were only 4.

In summary, engineers should very seriously consider a suitable \( \beta_T \) to conduct the reliability-based design of pile foundations. Selecting a small \( \beta_T \) will leave the piles designed using LRFD methods at risk. Selecting a large \( \beta_T \) will lessen the identified resistance factors, the design scheme will be conservative and the cost will be uneconomical. The analysis in this section indicates that the values of \( \beta_T \) between 2.5 and 3.0 are suitable. \( \beta_T \) between 2.5 and 3.0 indicates the acceptable maximum failure probability between 0.1% and 0.6%, which is low enough for general pile foundation engineering. Besides, the iterative steps are nearly 6 for \( \beta_T \) with values between 2.5 and 3.0, and the computational efficiency is good, too.

**VALIDATION AND APPLICATION**

Practical validation and application of the proposed method will be illustrated by the following two examples, respectively.

**Validation example**

According to Kim et al (2011), \( \phi_{\text{ult,b}} \) and \( \phi_{\text{ult,s}} \) values were calibrated using their proposed method for building and bridge structures, which are compatible with the ASCE/SEI 7-05 (ASCE 2005) load factors and the AASHTO (2007) load factors. For comparison and validation, the corresponding load statistical parameters are considered in this section (Kim et al 2011): in the case of ASCE/SEI 7-05, \( \lambda_D = 1.05, \text{COV}_{L_D} = 0.1, \lambda_L = 1.0 \) and \( \text{COV}_{L_L} = 0.25 \); while in the case of AASHTO, \( \lambda_D = 1.05, \text{COV}_{L_D} = 0.1, \lambda_L = 1.2 \) and \( \text{COV}_{L_L} = 0.205 \). Also, \( \gamma_Y = 1.25 \) and \( \gamma_L = 1.75 \) are selected for both ASCE/SEI 7-05 and AASHTO cases. The process of estimating the resistance factors for building and bridge structures must satisfy the following conditions:

1. Utilise \( \rho = L_L/L_D \) with four different values: 1.0, 2.0, 3.0 and 4.0
2. Utilise \( \beta_T \) with four different values: 2.0, 2.5, 3.0 and 3.5
3. Determine both \( s = R_{\text{ult,b}}/(U_D + L_L) \) and \( b = R_{\text{ult,b}}/(U_D + L_L) \) as 1.0
4. Select resistance statistical parameters referring to Jardine et al (2005), namely \( \lambda_{\text{ult,b}} = 1.023, \lambda_{\text{ult,s}} = 1.088,\)

![Figure 4 Resistance factors \( \phi_{\text{ult,b}} \) and \( \phi_{\text{ult,s}} \) of driven piles versus target reliability index \( \beta_T \)](image)

**Table 4 Summary of resistance factors**

<table>
<thead>
<tr>
<th>Case</th>
<th>( \beta_T )</th>
<th>Results of this paper</th>
<th>Results of Kim et al (2011)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \rho = 1 )</td>
<td>( \rho = 2 )</td>
<td>( \rho = 3 )</td>
</tr>
<tr>
<td>ASCE/SEI 7-05</td>
<td>2.0</td>
<td>0.62 0.66</td>
<td>0.58 0.62</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>0.50 0.53</td>
<td>0.47 0.50</td>
</tr>
<tr>
<td></td>
<td>3.0</td>
<td>0.40 0.43</td>
<td>0.38 0.40</td>
</tr>
<tr>
<td></td>
<td>3.5</td>
<td>0.32 0.34</td>
<td>0.30 0.32</td>
</tr>
<tr>
<td>AASHTO</td>
<td>2.0</td>
<td>0.58 0.62</td>
<td>0.57 0.60</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>0.48 0.51</td>
<td>0.46 0.49</td>
</tr>
<tr>
<td></td>
<td>3.0</td>
<td>0.39 0.41</td>
<td>0.38 0.40</td>
</tr>
<tr>
<td></td>
<td>3.5</td>
<td>0.32 0.34</td>
<td>0.31 0.33</td>
</tr>
</tbody>
</table>

58 Volume 60 Number 2 June 2018 Journal of the South African Institution of Civil Engineering
COVφult,b = 0.201 and COVφult,s = 0.287. Calculated results of φult,b and φult,s using the proposed method in this paper are shown in Table 4. For building structures with the ASCE/SEI 7-05 (ASCE 2005) load factors, the calculated φult,b and φult,s values vary within the ranges 0.55–0.62 and 0.58–0.66 for βT = 2.0; 0.45–0.50 and 0.48–0.51 for βT = 2.5; 0.36–0.39 and 0.39–0.41 for βT = 3.0; and 0.30–0.32 and 0.32–0.34 for βT = 3.5, respectively, depending on the ratio ρ = Lult/b/L (ρ = Lult/s/LUL range of 1.0–4.0).

For bridge structures with the AASHTO (2007) load factors, the calculated φult,b and φult,s values vary within ranges 0.55–0.58 and 0.58–0.66 for βT = 2.0; 0.45–0.48 and 0.48–0.51 for βT = 2.5; 0.36–0.39 and 0.39–0.41 for βT = 3.0; and 0.30–0.32 and 0.32–0.34 for βT = 3.5, respectively, depending on the ratio ρ = Lult/b/L (ρ = Lult/s/LUL range of 1.0–4.0).

For ease of comparison, the results of φult,b and φult,s from Kim et al (2011) are also given in Table 4 (columns 11 to 16). Comparison shows that the results of φult,b and φult,s for building and bridge structures compatible with load factors from ASCE/SEI 7-05 (ASCE 2005) and AASHTO (2007) computed in this paper, are smaller than the results from Kim et al (2011). These differences may be due to many probable reasons, but it should be pointed out here that some main uncertainty factors are not considered in Kim et al (2011), such as proportions of shaft (base) resistance to load (namely Rult,b/L (LD + LS) and Rult,s/L (LU,D + LU,S)), the correlation between φult,b and φult,s, and so on.

For example, in Kim et al (2011), φult,b and φult,s for building and bridge structures compatible with load factors from AASHTO (2007) vary within ranges of 0.82–0.87 and 0.70–0.75 for βT = 2.5; 0.76–0.79 and 0.63–0.66 for βT = 3.0; and 0.69–0.73 and 0.57–0.60 for βT = 3.5. These resistance factors in Kim et al (2011) seem to be very large for the reliability-based design of piles. The resistance factors proposed by AASHTO (2007) for strength limit state for shallow foundations are generally between 0.35–0.60, which perhaps more strongly support the results in this paper.

**Application example**

Luo (2004) compiled a database of pile load tests, including 151 driven pile load tests. From these databases only 128 driven pile load tests with sufficient information (measured ultimate bearing capacity, base resistance and shaft resistance) were analysed. The bias factors and COVs of pile resistances were calculated by authors referring to Luo (2004): λult,b = 1.18, λult,s = 1.21, COVφult,b = 0.34, and COVφult,s = 0.22, respectively. Computed resistance factors using the proposed procedure are summarised in Table 5 for the following conditions:

1. Set ρ = Lult/b/L at four different values: 1.0, 2.0, 3.0 and 4.0
2. Set βT at four different values: 2.0, 2.5, 3.0 and 3.5
3. Both sets s = Rult,b/L (LD + LS) and b = Rult,s/L (LU,D + LU,S) at 1.0
4. Accepted load statistical parameters presented by AASHTO (2007), namely λD = 1.08, COVγD = 0.13, λL = 1.15, COVγL = 0.18, γD = 1.25 and γL = 1.75.

Table 5 Summary of resistance factors (calculated by database from Luo (2004))

<table>
<thead>
<tr>
<th>βT</th>
<th>ρ = 1</th>
<th>ρ = 2</th>
<th>ρ = 3</th>
<th>ρ = 4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>φult,b</td>
<td>φult,b</td>
<td>φult,b</td>
<td>φult,b</td>
</tr>
<tr>
<td>2.0</td>
<td>0.62</td>
<td>0.68</td>
<td>0.59</td>
<td>0.63</td>
</tr>
<tr>
<td>2.5</td>
<td>0.49</td>
<td>0.51</td>
<td>0.47</td>
<td>0.48</td>
</tr>
<tr>
<td>3.0</td>
<td>0.39</td>
<td>0.40</td>
<td>0.38</td>
<td>0.39</td>
</tr>
<tr>
<td>3.5</td>
<td>0.32</td>
<td>0.32</td>
<td>0.30</td>
<td>0.31</td>
</tr>
</tbody>
</table>

CONCLUSIONS

Uncertainties regarding the bearing capacity of piles actually derives from the ultimate base and shaft resistances, and should be explained separately in the reliability-based design of piles. The way to solve this problem is by developing a method to evaluate and study the ultimate base and shaft resistance factors respectively. This is the major contribution achieved in this paper.

Convergence analysis demonstrates that the presented iteration algorithm to estimate ultimate base and shaft resistance factors converges rapidly and remains

Table 6 Recommended resistance factors (presented in this current study from database by Luo (2004))

<table>
<thead>
<tr>
<th>βT</th>
<th>2.0</th>
<th>2.5</th>
<th>3.0</th>
<th>3.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>φult,b</td>
<td>0.60</td>
<td>0.47</td>
<td>0.38</td>
<td>0.31</td>
</tr>
<tr>
<td>φult,s</td>
<td>0.61</td>
<td>0.49</td>
<td>0.39</td>
<td>0.31</td>
</tr>
</tbody>
</table>
stable. The condition of convergence (i.e. $b$ and $s$ between 0 and 3.0) can meet the demand of the reliability-based design of piles satisfactorily.

In addition, parameter analysis indicates that the ratio of dead to live loads has a limited influence on calculated resistance factors. The overall consideration of dead and live loads in the determination of ultimate base and shaft resistance factors is reasonable. Similarly, the initial seed resistance factor also has little effect on convergence of the final resistance factors. Any initial seed resistance factor could be selected in the reliability-based design of piles. However, the target reliability index significantly influences computed resistance factors, and an appropriate target reliability index is required for the reliability-based design of piles.

In a nutshell, the ultimate base and shaft resistance factors for the reliability-based design of piles can easily be obtained using the proposed procedure with an appropriate target reliability index. The application example has illustrated this point.

ACKNOWLEDGEMENTS
The authors would like to express their gratitude to the National Key Research and Development Program of China (2016YFC0800208) and the National Natural Science Foundation of China (51708428, 51408444, 51378404).

REFERENCES
Guidelines for the preparation of papers and technical notes

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

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 confirm 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 had been published elsewhere. Proof of such permission must be supplied.

SUBMISSION PROCEDURES AND REQUIRED FORMAT

- Online submission: Manuscripts must be uploaded as PDF files (http://journal.saice.org.za). Individual file sizes may not exceed 4 MB. Should you experience problems uploading your paper, please contact the editor (verelen@saice.org.za).

- 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 possible, 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 had 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 re-writes will be allowed, only essential minor corrections.