\[
F = \sum_{W} \frac{1}{\sin \alpha \cos \alpha + (\sin \alpha \tanh \beta) (F)}
\]

\[
k \left( \frac{\partial^2 h}{\partial x^2} + k \frac{\partial^2 h}{\partial z^2} \right) = \frac{c'B + (W-uB) \tan \beta}{\alpha}
\]
<table>
<thead>
<tr>
<th>Page</th>
<th>Title</th>
<th>DOI</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>A comparative evaluation of the impact of average speed enforcement (ASE) on passenger and minibus taxi vehicle drivers on the R61 in South Africa</td>
<td>10.17159/2309-8775/2016/v58n4a1</td>
</tr>
<tr>
<td>11</td>
<td>Experimental and analytical investigation into the stiffness of composite steel-reinforced timber beams with flexible shear connectors</td>
<td>10.17159/2309-8775/2016/v58n4a2</td>
</tr>
<tr>
<td>21</td>
<td>A risk and cost management analysis for changes during the construction phase of a project</td>
<td>10.17159/2309-8775/2016/v58n4a3</td>
</tr>
<tr>
<td>29</td>
<td>Wind loading on catenary vault structures</td>
<td>10.17159/2309-8775/2016/v58n4a4</td>
</tr>
<tr>
<td>48</td>
<td>The effect of type, concentration and volume of dispersing agent on the magnitude of the clay content determined by the hydrometer analysis</td>
<td>10.17159/2309-8775/2016/v58n4a5</td>
</tr>
<tr>
<td>55</td>
<td>The possible rate of transition to lower-carbon housing</td>
<td>10.17159/2309-8775/2016/v58n4a6</td>
</tr>
<tr>
<td>62</td>
<td>Seismic evaluation of the northbound N1/R300 bridge interchange</td>
<td>10.17159/2309-8775/2016/v58n4a7</td>
</tr>
</tbody>
</table>
A comparative evaluation of the impact of average speed enforcement (ASE) on passenger and minibus taxi vehicle drivers on the R61 in South Africa

N A Ebot Ena Akpa, M J Booyse, M Sinclair

Average speed enforcement (ASE) is an emergent alternative to instantaneous speed limit enforcement to improve road safety, and is used to enforce an average speed limit over a road segment. This paper presents a study on the response of passenger vehicles and minibus taxis to ASE on the R61 in South Africa. A spatio-temporal quantitative study of speed compliance was conducted, where metrics such as speed variability, average speed and 85th percentile speed measured prior to, and during enforcement, were analysed for two prominent modes of transport – passenger vehicles and minibus taxis. These measurements were taken on the enforcement route and on control routes adjacent to and further away from the enforcement route. A qualitative study was also conducted to evaluate the relationship between speed compliance and driver understanding of the system. The impact of the system on crash risk and injury severity was also examined before and during enforcement. For passenger vehicles, results showed that the introduction of ASE was followed by a reduction in mean speed on the enforcement route and adjacent control route. For minibus taxis, it was found that ASE appears to have little influence on improving speed compliance, which is likely associated with a lack of driver understanding of how the system operates.

INTRODUCTION

With road transport becoming an increasingly integral part of societal activities in South Africa, and Sub-Saharan Africa in general, the need for efficient road safety measures is growing. The World Health Organisation (WHO 2013) indicates that the African region has only about 2% of the world’s registered vehicles, but accounts for almost 20% of global traffic deaths, averaging fatality rates of 24 deaths per 100 000 inhabitants. Speeding is often cited as the leading human factor responsible for these fatalities. Studies have also shown that there is a direct relationship between vehicle speed, crash risk and crash severity (Aarts & Van Schagen 2006). According to South Africa’s 2011 road traffic report (RTMC 2011), speeding contributed to about 40% of fatal crashes due to human error. As a result, modern Intelligent Transport System (ITS) safety measures, such as average speed enforcement (ASE), are geared towards regulating human factors, such as speeding, with the ultimate goal of improving safety.

Various countermeasures have been used to reduce speed-related fatalities and injuries in South Africa. While some countermeasures, such as rumble strips and speed humps, are aimed at managing vehicle speeds, other countermeasures, such as instantaneous speed cameras, are aimed at enforcing compliance with posted speed limits. However, these countermeasures are usually only effective around the vicinity of the intervention infrastructure, and most of them are impractical and costly for use over long distances. This paper focuses on ASE implemented through an average speed over distance (ASOD) system deployed on the R61 in South Africa.

The ASE system is a technology that ideally promotes both speed management and compliance with posted speed limits by using camera pairs with automatic number plate recognition (ANPR) functionality, strategically placed along a road section. Licence plates are captured at an initial camera location (the entry cabinet) and also at any subsequent camera location (the exit cabinet). The known distance between both cameras, and the travel time between them, are used to calculate the average speed of the vehicle. A fine is issued if the...
calculated average speed is higher than the legal speed limit for the vehicle type on the given road. Camera visibility is enhanced through roadside notifications at the entry and exit cabinets.

A number of studies in Australia and Europe have proved the effectiveness of ASE. Most of its effects on crash risk, injury severity and speed violations are undeniably positive. However, these effects may vary from region to region (Sussman 2008). Existing ASE studies have only investigated the enforced road sections, and have largely neglected the effects on adjacent and other control sections. Moreover, research to date has not distinguished between different modes of transport.

Apart from South Africa, there is no documented literature on the implementation of ASE in Sub-Saharan Africa. The majority of countries in Sub-Saharan Africa rely on police patrols, rumble strips and speed humps to control speed (Afukaar 2003). South Africa launched one of its first ASE systems in November 2011 on the R61 – a 71.6 km stretch of road between Beaufort West and Aberdeen in the Western Cape Province. Media reports on the system claim that it has been effective in road safety improvement, a claim apparently substantiated by reported traffic injuries. Evaluating the effectiveness of ASE systems is, however, a relatively new research area in the African context with its unique transport modes and challenges, where ASE systems have been running for less than half a decade. Hence there is still a general lack of a credible body of research on the extent of its effects on speed management in different regions, and the availability of concrete evidence to substantiate its benefits for different modes of transport in those regions.

The minibus taxi industry

The minibus taxi industry is a vibrant and mostly unregulated sector of public transport in South Africa, which has been largely associated with speed-related road fatalities. It is well known that the minibus taxi industry in South Africa constitutes the bulk of public transport and is generally characterised by substandard vehicles, overloading and high-risk driving behaviour, such as speeding and reckless driving (Sukhai et al 2004).

South Africa has at least 150 000 minibus taxis (Arrive Alive 2015) which serve about 67.9% of the public transport market share (Trans-Africa Consortium 2008). According to a study by Trans-Africa (Trans-Africa Consortium 2008), most taxi owners in Sub-Saharan Africa manage to cover their operation costs, but can barely afford to maintain or upgrade their fleets satisfactorily, hence compromising on quality and safety.

Minibus taxis mainly function as vehicles for urban and long-distance transport. The role of minibus taxis in urban transport falls somewhere between that of metered taxis (cabs) and urban buses in the developed world, while long-distance taxis perform a function similar to that of coaches in the developed world. Although the logistics of these two functions (long-distance and urban) are different, the vehicles and drivers involved are the same. It is common for a taxi driver who ferries passengers to work and back from Monday morning to Friday afternoon, to also complete a long-distance route over the weekend (Booysen et al 2013). Details on how the minibus taxi industry conducts long-distance trips are presented in Booysen et al (2013), and Booysen and Ebot Ena Akpa (2014).

This paper focuses on long-distance transport where most ASE systems are encountered. A typical long-distance route is from Cape Town in the Western Cape Province to Mthatha in the Eastern Cape. The route, along the N1 and R61, is frequently used by minibus taxis over weekends and holiday seasons. Since no official information could be found to confirm the number of minibus taxis that complete the Cape Town to Mthatha route every weekend, a traffic count was performed. Figure 1 shows the number of minibus taxis taking long-distance trips along the N1 over a typical weekend during the festive season. More than 1 700 taxis were identified in twelve hours. From interviews with taxi operators it became clear that the vast majority of long-distance taxis using this stretch of the N1 are bound for the Eastern Cape, with significantly fewer heading to the Free State and Gauteng.

**Figure 1** Traffic counts of minibus taxis driving through the ASE route over a period of twelve hours

**RELATED WORK**

ASE systems have been operating in certain regions for over a decade. The first instance was a trial system installed in 1997 in the Netherlands, which ran for five years before permanent installation in 2002. In 2000, England launched its first permanent system after running trial versions for a year. Due to the lack of literature on ASE system studies in Africa, this section summarises research outcomes predominantly carried out in Europe, where the impact of ASE systems has been evaluated in detail.

A number of studies have been conducted to evaluate the impact of ASE on speed and crash rates. Soole et al (2013) compiled a concise literature review of ASE evaluation in Europe. The aim of their research was to monitor compliance with posted speed limits on enforcement routes. They also investigated the effectiveness of ASE systems on driver perception, including comparison with other countermeasures. Previous studies on some enforcement routes revealed that ASE reduced mean and 85th percentile speeds by up to 33%. In addition, speed variations from posted speed limits were reduced, with speeds typically below or at the posted speed limits. Their findings support ASE as a complementary measure to existing speed compliance measures, particularly suitable on roads with historically high crash rates. They nevertheless conclude that ASE systems are a more reliable and cost-effective approach to speed enforcement, and are widely accepted by road users. The main outcomes of evaluation studies reviewed by Soole et al (2013) are presented in the following paragraphs. These studies were conducted in the Netherlands, Italy, and England.

In the Netherlands a study was conducted in 2005 on the A13 in Rotterdam, which had a posted speed limit of 80 km/h.
During enforcement, average speed on the enforcement route reduced to 80 km/h from 100 km/h. Reduction in speed variance and 85th percentile speed was also observed. Moreover, offence rates dropped by 4%, all crashes reduced by 47% and fatalities reduced by 25% (Stefan 2005).

In Italy an evaluation of all enforcement routes was conducted in 2009. Average speeds reduced by 16 km/h (corresponding to a 15% reduction) during the first year of operation. After the first year, average speeds reduced further by 9.1 km/h. Fatalities also reduced by 50.8%, while serious-injury crashes reduced by 34.8%. In 2011, a one-week pre-installation and post-installation comparative study conducted on an 80 km/h road in Naples also showed positive impact. Average speed dropped by 9 km/h and speed variance dropped from 18.1 km/h to 12.1 km/h. For crash outcomes, eight-month pre-installation and post-installation periods were compared. Serious and minor injuries reduced from 116 to 71, while fatal crashes reduced from four to zero (Cascatta & Punzo 2011).

In 2011, a series of evaluations were conducted by relevant stakeholders at 13 locations in England (data was provided by stakeholder consultation). Speed profiles three years before enforcement were compared with speed profiles for three years during enforcement. Posted speed limits of enforcement routes were between 30 mph and 50 mph. The 85th percentile speed dropped by about 14.4% at 11 locations, but increased at one. Average speed reduced by an average of 12.5% at 10 locations, increased at two and remained unchanged at one. The proportion of vehicles travelling above the speed limit reduced by an average of 30%. Across all routes, crashes reduced by an average of 51.6% and casualties reduced by an average of 41.8%.

According to the government of the Western Cape Province in South Africa, ASE systems also have positive effects on speeding (Safely Home 2012). In 2012, a macroscopic evaluation of the system was conducted on the R61 by using only data captured through the ASE system. Prior to enforcement a total of 509 crashes had been reported, 75 of which involved fatalities. The specific time frame before ASE implementation, during which these crashes occurred, was not reported. During enforcement, between November 2011 and November 2012, no fatal crashes were reported. The proportion of vehicles driving above the speed limit of 120 km/h dropped from 39% to 26%, and the percentage of vehicles driving below the speed limit increased from 61% to 74%.

**Figure 2 R61 evaluation routes**

**Contribution of this work**

Although much research exists for ASE implementations in the developed world, the impact of ITS safety interventions vary from region to region (Sussman 2008). Moreover, to the best of the authors’ knowledge, no research exists that considers the impact of ASE on different modes of transport with differentiated speed limits. Furthermore, existing literature on ASE does not evaluate the impact on adjacent road segments, or compare the results with control routes. This paper examines the impact of ASE on speeding patterns (and crash rates) on the R61 in South Africa – a bidirectional single carriageway with no central reservation. The paper evaluates the impact on two prevalent transport modes, namely passenger vehicles and minibus taxis (the dominant forms of private and public transport in Sub-Saharan Africa) with respective speed limits of 120 km/h and 100 km/h on the R61. Time differentiation and spatial differentiation analyses were performed to establish the impact on the ASE route, and also on control routes at various distances from the enforcement route. The paper analyses the behavioural changes observed, or lack thereof, and presents explanations for anomalous effects not seen elsewhere in the literature, including a qualitative study that was motivated by the high violation rates observed from trips completed by minibus taxis (Booyseen & Ebot Eno Akpa 2014). Crash outcomes (fatalities, serious and minor injuries) are also analysed on the enforcement route by comparing two years of pre- and post-installation effects of the ASE system.

The investigation transends macroscopic effects presented by local authorities (Safely Home 2012) to address macroscopic effects such as reductions in average speed and speed variability.

**EXPERIMENTAL SETUP**

**Quantitative analysis**

The aim of the quantitative analysis was to investigate the impact of ASE systems on speed limit compliance and crashes for two modes of transport. This section focuses on the compliance, while methods pertaining to the crashes are presented in a subsequent section. To obtain detailed effects on speed compliance, one enforcement route (ER) with an ASE, and three control routes (CR I, CR II, and CR III) without ASE were evaluated. CR I was chosen since it shares similar characteristics with the enforcement route, while CRs II and III were chosen to observe speeding patterns further away from the enforcement route, and were frequently used by passenger vehicles equipped with TomTom devices. Figure 2 shows the enforcement and the control routes, while Table I shows the geometric and traffic characteristics of each route. CR III (between Hanover and Colesberg) is situated 240 km from the ER, north of the N1. Evaluation dates ran from June 2009 to June 2011 before enforcement, and from December 2011 to December 2013 during enforcement. With regard to the state of enforcement on these routes before ASE, it should be noted that there were no permanent ITS interventions, and speed enforcement was carried out exclusively by mobile police units.

Time differentiation was performed on the enforcement and control routes. This involved a ‘before’ and ‘during’ enforcement analysis for each route. Results from time
Differentiation on the enforcement route were expected to show reduction in travel speeds during enforcement. Similar results were also expected of CR I, considering its proximity to the enforcement route, while CR II and CR III was expected to show little or no impact due to enforcement.

Spatial differentiation was also performed with the aim of determining the impact of the system on control routes relative to the enforcement route. This involved ‘in’ and ‘out’ of ASE section analysis before and during enforcement. Comparing the enforcement route and CR I, results from spatial differentiation before enforcement were expected to be similar, while results during enforcement were expected to be slightly different. Between the enforcement route and CR II, spatial differentiation results were expected to be similar before implementation, but different after implementation. Similar results were expected between the enforcement route and CR III. It was well understood that, despite these expectations, the riding quality, general traffic patterns of the routes over time, policing, etc, could lead to different results.

**Data sets**

Two independent data sets were considered for the quantitative study. Firstly, TomTom traffic statistics obtained from tracking devices, TomTom navigation devices and TomTom fleet management devices were used. This data set represented fleet monitoring for passenger vehicles mainly used for private transportation. TomTom devices are considered a luxury item.

The second data set was obtained from nine minibus taxis registered under the Eastern Cape were obtained between November 2013 and May 2014, these covering a total distance of more than 50 000 km. There was no data for minibus taxis before ASE. Due to this data availability constraint, only spatial differentiation analysis during enforcement was performed for the minibus taxis. In addition, minibus taxis rarely travel along CR III. As a result spatial differentiation analysis was not possible for minibus taxis on CR III.

**Data capturing and validation, with further analysis**

Although the tracking devices were programmed at a minimum transmission frequency of 1Hz, not all consecutive records were captured at this frequency, due to filtering and data loss. Despite the accuracy of the GPS as a measurement device, it is still subject to systematic and random errors, which could be out by as much as 15 m per sample (Gates et al. 2004). The reasons for this include the following:

- Systematic errors that affect accuracy may occur due to a low number of satellites in view, a high horizontal dilution of precision (HDOP) which relates to satellite orientation on the horizon and its impact on position precision, and other factors such as poor antenna placement.
- Random errors may occur due to signal blockage, atmospheric effects, multipath signal reflection, satellite orbit, and other factors such as receiver defects.

Systematic error effects were minimised by removing GPS records with less than five satellites in view and HDOP’s greater than one. On the other hand, the effects of random error were difficult to address. Statistical smoothing techniques or visual inspection of data can be used to identify random errors (Jun et al. 2006). Polygons surrounding each route were used to minimise the effects of random error. Only records within the polygons were used.

To validate the minibus tracking data, average speeds captured by the ASE system were compared with average speeds calculated from the GPS traces. The ASE system’s speeds were obtained from twelve fines levied on minibus taxi drivers between December 2013 and March 2014. Time stamps on each fine, with their corresponding average speeds, were mapped against GPS-calculated average speeds with the same time stamps. A maximum percentage error of 0.85% was measured between ASE and GPS average speeds. Two GPS reference records closest to the entry and exit cabinets respectively were selected from the list of GPS records defining a trip. For each trip, a 2 km radius was defined around each camera to minimise wide variations in the location of reference records. Trips with no GPS records in the specified radius were excluded from the analysis. This ensured a maximum deviation of 4 km in travel distance from the fixed travel distance of 71.6 km. The GPS average speed for each trip was calculated using the known distance and travel time between the reference records.

Average speeds calculated from reference records were also used to conduct further analyses on minibus taxis. These were used to detect if a given trip violated the ASE system. The 402 valid trips through the enforcement route were identified and analysed. Each taxi was examined separately.

**Crash risk and injury severity**

High crash rates on a particular road are often the reason behind ASE system deployment. Reduction in crash rates due to ASE rest on the assumption that their effect on vehicle speed is equally worthwhile. It is therefore necessary to investigate their impact on crash rates. To this end, crash data within the enforcement route between January 2008 and September 2014 was provided for analysis by the Western Cape Department of Transport. Time-based analysis around the enforcement date of November 2011 was applied with pre-implementation and post-implementation periods of two years. The analysis was conducted for minibus taxis and passenger vehicles for crashes primarily linked to human error due to speeding.

**Qualitative analysis**

Although the trips in the study were captured from nine vehicles, multiple taxi drivers were involved, as more than one taxi driver is employed to drive each vehicle. In all, a total of 20 minibus taxi drivers were interviewed to determine their level of understanding of ASE. Only those drivers who frequently drive through the R61 enforcement routes between Cape Town and the Eastern Cape Province
were interviewed. Based on their understanding of ASE, taxi drivers were grouped into three categories. The first category represented drivers who understood how the ASE system operates and where it had been deployed along the route. The second category represented drivers who understood how the system operates, but were unaware of its location along the route. The third category represented drivers who neither understood how the system operates nor where it had been deployed along the route.

**RESULTS**

**Speed compliance results**

More than 6 000 vehicles were identified from TomTom traffic queries making complete trips through the respective evaluation routes in the time frames considered. For minibus taxis, 402 trips identified from GPS records were analysed.

The results are presented in Figures 3 and 4, and Tables 2, 3 and 4, in which $V_{85}$ represents the 85th percentile speed, and $V_{120}$ represents the percentile crossing at 120 km/h. In Table 2, delta ($\Delta$) represents differences between ‘during’ and ‘before’ implementation parameters, while in Table 3 it represents differences between control and enforcement route parameters.

**Passenger vehicles**

The time differentiation results for passenger vehicles, illustrated in Figure 3 and Table 2, show a reduction of 5 km/h in both the mean speed (to 105 km/h) and 85th percentile speed (to 124 km/h) on the ER, and a change from 66% to 75% in speed limit compliance (similar to 124 km/h) on the CR I. Its mean and 85th percentile speed were 3.6 km/h and 1 km/h lower than that on CR II and III, and was in line with the results mentioned in the “Related Work” section above, which suggests that more time was spent driving below the legal speed limit on the enforcement route. However, these changes are not limited to the enforcement route, with an even greater improvement apparent in the adjacent CR I – here mean speed reduced by 7 km/h and the 85th percentile speed reduced by 13 km/h, corresponding to a 10% reduction. Interestingly the 85th percentile of CR I was 7 km/h higher than that of the ER before ASE, at 136 km/h, and reduced to within 1 km/h after introduction of ASE. A 4 km/h improvement is also noticeable in the 85th percentile speed of CR II, but to a relatively high 134 km/h, with a similar trend on the mean speeds for CR II. Although the 85th percentile for CR III improved by 3 km/h, the mean speed was 4 km/h higher. These results indicate an improvement after introduction of the ASE, but similar improvements in the adjacent and farther away control sections.

Moreover, these improvements on the ER and CR I occurred despite the fact that their speed profiles before enforcement were already significantly lower than those of CR II and III. Together with Table 3, Figure 3 also gives insight into spatial differentiation results. During enforcement, the ER and CR I had similar mean and percentile speed profiles. CR II and CR III also had similar profiles. At any given percentile, speed margins between the enforcement route and CR I to CR II and CR III were about 10 km/h. Before enforcement, however, these margins were lower and inconsistent, suggesting a higher degree of similarity and the absence of average speed-related enforcement. Coupled with observations from time differentiation results, it was observed that the ASE system appeared to have influenced passenger vehicle drivers to comply with speed limits along the enforcement route and on CR I, but not on control routes further away, such as CR II and CR III.

Two concerns arise from the time and spatial differentiation results. Firstly, between the enforcement route and CR I, it was observed that during enforcement CR I showed a slightly better level of compliance with the speed limit. Its mean and 85th percentile speeds were 3.6 km/h and 1 km/h lower than that of the enforcement route respectively. Speed profiles on CR I were expected to improve, but not to the point where they would be better than the enforcement route. This unexpected result may be due to routine maintenance during the enforcement period on CR I. During maintenance, which typically lasts for two months in a year, speed restrictions are set at 100 km/h, with occasional Stop/Go closure delays (SANRAL 2011; SANRAL 2013). The second point of concern is that CR II and III unexpectedly also showed slight reduction in speeds. This could be due to road safety campaigns carried out across the country on roads with high death tolls. Following this trend, factors responsible for this could to some extent be responsible for reduction in speeds along the enforcement route, which may have nothing to do with the ASE system. Nevertheless, the reduction in speed along the enforcement route was better than that on CR II and III, and was in line with the results mentioned in the “Related

| Table 2 Spatio-temporal comparison for passenger vehicles |
|---------------------|-----|-----|-----|-----|-----|-----|
| Enforcement | Trips | Mean | $V_{85}$ | $V_{120}$ | $\Delta_{\text{mean}}$ | $\Delta_{\text{85}}$ | $\Delta_{\text{120}}$ |
| Before | 306 | 110 | 129 | 66 | 20 | –5 | –5 | 9 |
| During | 1 389 | 105 | 124 | 75 | 30 | –7 | –13 | 16 |
| CR I | Before | 101 | 109 | 136 | 64 | 20 | –7 | –13 | 16 |
| During | 528 | 102 | 123 | 80 | 28 | –4 | –4 | 10 |
| CR II | Before | 2 000 | 121 | 138 | 38 | 6 | –4 | –4 | 10 |
| During | 3 500 | 117 | 134 | 48 | 13 | –4 | –4 | 10 |
| CR III | Before | 94 | 111 | 137 | 46 | 21 | –4 | –3 | 1 |
| During | 200 | 115 | 134 | 47 | 13 | –4 | –3 | 1 |

![Figure 3 Time differentiation results for passenger vehicles](image-url)
Passenger vehicles and minibus taxis

The spatial differentiation results after introduction of ASE are presented in Figure 4 and Table 3, for both passenger vehicles and minibus taxis. Percentiles in Table 3 show that only about 14% of all recorded taxi speeds were within their legal speed limit of 100 km/h. Furthermore, besides lower variations in speed, their speed profiles were very similar to, or higher than, those of passenger vehicles. This finding conforms to a previous study (Booysen & Ebot Eno Akpa 2014), which presents similar results for three other road sections.

The passenger vehicles generally showed rising mean and 85th percentile speeds (+12 and +10 km/h respectively), and falling percentiles for the 100 km/h and 120 km/h crossings (–17 and –27 percentage points respectively), from ER to CR II. Conversely, the minibus taxis exhibited an increase in mean speed of only 4 km/h and no change in 85th percentile crossing. Similarly, for the minibus taxis the 120 km/h crossing was at 5 percentage points higher, and no change for the 100 km/h crossing. These results support the hypothesis that the ASE has an impact on drivers of normal passenger vehicles, but suggest further that there is no significant change for minibus taxis in the ASE section.

Figure 5, which shows speed distribution plots on all routes, also confirms this finding – mean speeds were at 110 km/h on the enforcement route, 112 km/h on CR I and 114 km/h on CR II. Standard deviations were at 14.7 km/h on the ER, 13.1 km/h on CR I, and 11.9 km/h on CR II.
and 13.7 km/h on CR II. From these results, it appears that minibus taxis were not influenced by the presence of ASE along the R61 at all. Also, the similarity between minibus taxi speeds during enforcement and passenger vehicle speeds before enforcement along the ER and CR I is an indication that time differentiation analysis on minibus taxis showed little or no significant change.

**Further investigation on minibus taxis**

Investigations of individual trips along the enforcement route were conducted for each taxi. Table 4, which is a summary of system violations detected from GPS data, shows that most drivers did not conform to the 100 km/h limit. Results are expressed as the percentage of trips with an average speed beyond a specified threshold. Thresholds start at the 100 km/h speed limit and end at 120 km/h, with 5 km/h increments. N denotes the number of trips completed through the ASE system, and SL denotes the speed limit of 100 km/h.

For average speed, results show that at least 70% of trips taken by each taxi violated their 100 km/h speed limit, and for some taxis close to 34% of their trips violated the 120 km/h speed limit of passenger vehicles. While these results show that ASE has little or no impact on minibus taxis, they also support previous findings (Bester & Marais 2012) on the impracticality and enforcement difficulties associated with differentiated speed limits. Interviews with the taxi drivers revealed that, although they are all aware of the 100 km/h speed limit, they nevertheless consider 120 km/h as the limit that governs their choice of speed.

**Effect on crash risk and injury severity**

It is well known that speeding increases the risk of crash occurrence and severity. However, the specific cause of a crash may be due to several human factors, and not exclusively due to speeding. The data used in this study classified crashes based on their specific causes, none of which were attributed to speeding. As a result, statistics presented here only refer to crashes with specific causes linked to driver error/negligence.

Comparing two years before enforcement to two years during enforcement on the enforcement route, all crashes increased by 9.6% (from 83 to 91). Despite the increase in reported crashes, fatalities reduced by 79.5% (from 39 to 6), serious injuries reduced by 58.5% (from 53 to 22) and minor injuries reduced by 50% (from 106 to 53). Crash severity involving the two vehicle types considered in this study were queried separately, and the results are shown in Figure 6.

For passenger vehicles, the number of reported crashes decreased by 2% (from 49 to 48). Fatalities reduced by 57.1%, serious injuries reduced by 78.3%, and minor injuries reduced by 18.9%. For minibus taxis, the number of reported crashes increased by 38.1% (from 21 to 29). Nevertheless, a notable decrease in severity was observed – fatalities reduced by 90.6%, serious injuries reduced by 57.7% and minor injuries reduced by 79.7%. From these results it is probable that ASE had a significant role to play in crash severity, considering the reduction in mean speed during enforcement, and the known proportionality between speed and crash severity. However, it should be noted that the fatality results presented were measured over a fixed period of time. As such, effects due to regression-to-the-mean in road accident data were not taken into consideration. While results show that the deployment of the ASE system was effective, subsequent measurements may reveal different statistics which are not necessarily or solely linked to the ASE system.

**Driver perception and awareness**

This section presents outcomes of the survey related to ASE systems. Twenty drivers who regularly travel along the R61 were interviewed. All drivers were aware of their legal speed limit of 100 km/h and of the location of speed cameras along the route. Eighty percent of the drivers claimed that the presence of cameras caused them to adhere to speed limits within the vicinity of the camera, while 20% claimed not to be influenced by the presence of cameras, because they usually adhered to speed limits. Drivers were then asked if they understood how ASE systems work. Only two (10%) of the twenty drivers understood the concept of ASE and knew how ASE systems operate. The drivers who understood how the system operates also knew where it was deployed along the road. Eighteen drivers (90%) neither knew about the deployment of such a system nor how it worked. Four of these eighteen drivers admitted that they were advised by traffic officers to spend more than a minimum travel time on the road, below which they will get fined. These drivers were nevertheless placed in the third category of oblivious drivers since they...
neither understood how the system works nor knew the enforcement sections. The high percentage of ASE unawareness suggests that cameras at the beginning and end of the enforcement section were viewed as instantaneous speed cameras, which measure instantaneous speed just in the vicinity of the camera, and not over a longer distance. This was verified by examining taxi GPS speeds within three hundred metres of Camera B (camera between Beaufort West and Aberdeen). Camera A (just outside Beaufort West) was not included in this analysis due to comparatively low speeds which can be attributed to its proximity to residential areas. Figure 7 shows normalised results for the speed distribution within 300 m of Camera B against the speed distribution on the enforcement route. The mean speed within 300 m of Camera B was 60 km/h, which is 50 km/h less than the mean speed on the enforcement route, despite no noticeable differences in the road condition. Moreover, in the vicinity of Camera B, over 95% of speed records were below the 100 km/h speed limit. Thus, despite the proven advantage of ASE systems to improve speed uniformity along enforcement routes (Soole et al. 2013), most trips completed by minibus taxis proved otherwise, due to an apparent misunderstanding of average speed enforcement.

**DISCUSSION**

Average speed enforcement along the R61 is currently the primary intervention to counter speeding between Beaufort West and Aberdeen. Questions may arise as to whether improvements in speed compliance of passenger vehicles should be attributed to the system. Answers to these questions are especially relevant since a net decrease in overall speed was observed not only on the enforcement route, but on the control routes as well, although by varying degrees. It should be noted that high death tolls on provincial routes before enforcement have led to the systematic intensification of existing countermeasures and the launching of road safety campaigns during the enforcement period, which may have directly influenced speed compliance. However, this is impossible to quantify. Ad hoc police patrols were the most common countermeasure on this route before ASE. Despite these patrols, mean speeds and 85th percentile speeds were high before ASE, coupled with high crash rates and injury severity. Evidence of the impact of the ASE system can be seen from the fact that, during enforcement, speed compliance on the enforcement route is better than compliance on CR II with a lower 120 km/h percentile and a mean only 3 km/h lower than the speed limit. Also, despite the complementary nature of the ASE system amidst other countermeasures, the fact that these changes occur during enforcement indicate that the system could be actively responsible for speed compliance. The main advantage of ASE over other countermeasures is the reduction in mean speed, 85th percentile speed and low speed variability over long distances. As with other countermeasures, it is also associated with a reduction in crash rates and injury severity.

According to Elvik et al. (2009), studies that evaluate the effects of road safety measures by only relying on measures that influence driver behaviour, rather than crash rates or injuries, have less of an impact for two reasons. Firstly, for many forms of behaviour their relationship with crash occurrence is unknown, and secondly, the ultimate objective of all road safety measures is to reduce the expected number of crashes or injury severity. On the other hand, behavioural studies become more relevant when specific causes need to be identified or verified. For passenger vehicles, improvement in speed compliance is depicted by a corresponding decrease in crash rates and injury severity. With the introduction of the ASE system, its combined effect with other countermeasures along the enforcement route has led to a decrease in fatalities and injury severity for both passenger vehicles and minibus taxis. While it has been assumed that ASE is primarily responsible for the significant improvements on the enforcement route and CR I, two observations undermine this explanation. Firstly, with regard to mean and 85th percentile speeds, CR I performs better than the enforcement route. Secondly, before ASE implementation, mean speeds on the enforcement route and CR I were already lower than the legal speed limit of 120 km/h. From previous ASE evaluations, it was observed that such systems reduced speeds, causing drivers to drive around the enforced speed limit. As a result, reductions in speed observed for passenger vehicles on the enforcement route and CR I could be due to a higher visibility of police enforcement and awareness campaigns during enforcement.

![Figure 7 Speed distribution: Vicinity of Camera B against enforcement route](image-url)
CONCLUSION
General effects of ASE on road safety were already evident from reports of low crash rates and fatalities from local authorities. This study supplements these reports with driver behavioural patterns obtained from speed measurements. In summary, the introduction of ASE along the R61 coincided with reduced passenger vehicle speed and crash rates on the enforcement route and its immediate vicinity, but concrete evidence as to whether these reductions can be primarily attributed to ASE is still uncertain. Also, a lack of understanding of how ASE operates can greatly limit its benefits for different transport modes. With separate analyses conducted for each mode of transport, minibus taxi drivers were identified as habitual offenders of the system, exceeding their speed limit often, and having similar speed profiles on the enforcement route, its immediate vicinity, and beyond. Such unsafe driving behaviour on the enforcement route could potentially be mitigated by educating taxi drivers on how the system operates, since they displayed extremely low levels of understanding.

ACKNOWLEDGEMENTS
The authors would like to acknowledge MTN, MiX Telematics and TomTom for their financial and technical support.

REFERENCES
Experimental and analytical investigation into the stiffness of composite steel-reinforced timber beams with flexible shear connectors

W M G Burdzik, S Skorpen

Most of the current research on the design of timber composite beams involves either complex mathematical models which are not checked with experimental testing, or is purely based on experimental work with no attempt to model the behaviour. In a literature review the authors failed to find a practical way of designing composite timber beams, other than in the Eurocode 5. The equations in Eurocode 5 are unfortunately limited in their application. This paper looks at stiffening timber beams, with a known stiffness distribution, by screwing or nailing a steel strip to the underside of the beam. The modelled behaviour is compared with experimental test results and recommendations for the analysis and design of such members are given.

The experimental work involved determining the stiffness of twenty-four South African pine beams reinforced with metal strips. The spacing of the connectors was varied to ascertain the increase in stiffness of the composite with a reduction in the connector spacing. The analytical methods used were the Eurocode 5 method, as well as two finite element modelling methods, which may be used to determine the composite stiffness. The results of the three methods used show a remarkably good fit with the lower-bound experimental results.

INTRODUCTION

Timber design codes SANS 10163:2003 (SANS 2003) and BS EN 1995-1-1:2004 (EN 2004) call for beams to be designed for strength criteria, as well as serviceability criteria. The most important serviceability criterion is deflection, which is not only influenced by creep, but also by the modulus of elasticity of the beam. Sometimes beams that meet the strength criteria deflect too much under permanent loading only. To overcome a lack of stiffness, the second moment of area can be increased by either adding timber with a similar modulus of elasticity or by adding less material, such as steel, with a much higher modulus of elasticity.

Current research done on timber-steel composite beams is often either largely experimental work, with little attempt made to compare the results with the analytical design method given in Eurocode 5 (EN 2004), or complex mathematical modelling which is not checked with experimental results. The Eurocode 5 design method is also very limited in its application, as it applies only to flexural members where the stiffening element stretches from support to support. Examples of current research include timber-steel-hybrid beams in high-rise buildings (Winter et al 2012), work on two-dimensional simply-supported composite beams (Xu et al 2006), the analysis of partial composite beams (Girhammer et al 2006) and a simplified analysis method for composite beams with interlayer slip (Girhammer 2009). The authors believe that, because of the highly varied nature of timber, experimental verification is a crucial part of all timber-related research. Practical methods for analysis are also important for design engineers who would not necessarily be designing composite timber structures on a regular basis.

With the above in mind the authors had a two-pronged approach to the problem. The initial approach was to see how well proposed finite element models and the Eurocode equations would be able to predict the stiffness of composite timber and steel flexural elements. This was done by measuring the stiffness of each piece of timber, the steel plate and the predicted stiffness of the connector (based on test values) prior to the assembly of the composite.

With the successful completion of the first set of tests, which showed remarkable correlation between the proposed finite element models, the Eurocode equations and the individual test results, the tests were...
expanded to include other connectors and thickness of steel plate.

Most design engineers would not have the facilities to measure the timber stiffness, the stiffness of the steel plate or the stiffness of the connectors, and would have to use published values. The authors felt it prudent to validate the theoretical values, based on the actual stiffness of the beams and steel plates, with test results. Predicted theoretical values agreed with the measured stiffness within acceptable experimental tolerances. The authors feel confident to recommend that design engineers use the theoretical design method, and base the stiffness on the published characteristic density and modulus of elasticity of the steel and timber.

**COMPOSITE BEAM CONNECTORS**

When adding steel to improve beam stiffness, the best solution is to glue the metal strip to the underside of the beam with some form of epoxy glue, although phenol resorcinol formaldehyde has been used before (Ebersohn 1994). The adhesive forms a very thin layer between the steel and the timber, and this layer has a very high stiffness. Very little or no slip occurs between the two materials. Practical problems, such as the proper cleaning of the steel and the clamping of steel to the timber, make this a fairly difficult and expensive option, especially on site. Furthermore, many engineers do not trust the bond between the steel and the timber, as even small impurities such as oil or milling scale cause de-bonding and subsequent stress peaks around the de-bonded areas, which can lead to complete de-bonding of the two materials. When the epoxy is subjected to elevated temperatures, such as would occur during a low-intensity fire, degradation of the epoxy layer takes place, which could cause failure (Mouritz 2002). High-temperature-resistant low-viscosity epoxies are available, but these would not necessarily be used in the construction industry.

Acceptable alternatives are mechanical fasteners, such as high-tensile screws or nails. Mechanical fasteners, however, need to undergo deformation before they are able to transfer loads (nails are preferable, as the pre-drilled holes in the steel plate can be much smaller than for screws). This relative movement between the materials leads to loss in stiffness and strength. The stiffness of the composite will lie somewhere between the sum of the individual stiffnesses and that of the fully composite element. Figure 1 shows the strain distributions for three conditions, namely the idealised condition

(12)

| Figure 1 Strain distribution in a composite member, subjected to bending moment, for various conditions of connectivity between the two materials, where:
| $N_{AT}$ is the position of the neutral axis for the timber on its own
| $N_{AS}$ is the position of the neutral axis for the steel on its own
| $N_{AC}$ is the position of the neutral axis of the ideally combined section. |

### Equation 1

$$EI_C = 0.5E_S I_S + 0.5E_T I_T + A_S E_S y_S^2 + A_T E_T y_T^2$$

Where:
- $EI_C$ is the combined stiffness of the section with no slip on the interface
- $E_S$ is the modulus of elasticity of the steel
- $I_S$ is the second moment of area of the steel about its own axis
- $y_S$ is the distance from the combined neutral axis to the neutral axis of the steel
- $E_T$ is the modulus of elasticity of the timber

With no positive connection between the timber and the steel, the stiffness of the combined section is reduced to:

$$EI_C = 0.5E_S I_S + 0.5E_T I_T$$

Many engineers who do not design timber structures on a regular basis would assume the idealised case, calculate the shear force on the interface and design the connection to transfer the load. This method ignores the deformation of the connector and the loss in stiffness as a result of that deformation. However, a simple beam or shell finite element model that can accurately predict the
stiffness of the composite member, taking the stiffness of the connector into account, will make it easier for designers to assess the consequences of using steel and timber composites with flexible connectors.

**EUROCODE 5 MODEL**

Eurocode 5, BS EN 1995-1-1:2004 Appendix B contains an equation that can be used to determine the effective flexural stiffness of mechanically joined beams. This equation is based on a simply-supported beam with uniform distributed loading where the composite section stretches from support to support. The basic assumptions are given, and the equation offered to the designer is as follows:

\[ EI' = \sum_{i=1}^{n} (EI_i + y_i E_i A_i d_i^2) \]  

(3)

The two-element composite timber and steel beam that was used for this investigation is shown in Figure 2.

For a two-element composite member, as shown in Figure 2, the equation may be simplified with:

\[ A_1 = a_1 \times h_1 \text{ and } A_2 = a_2 \times h_2 \]  

(4)

\[ I_1 = \frac{a_1 \times h_1^3}{12} \text{ and } I_2 = \frac{a_2 \times h_2^3}{12} \]  

(5)

\[ y_2 = 1 \]

\[ y_1 = \left[ 1 + \frac{\pi^2 \times E_1 \times A_1 \times 4}{K_{ser} \times L^2} \right]^{-1} \]  

(6)

\[ a_2 = \frac{y_1 \times E_1 \times A_1 \times (h_1 + h_2)}{2 \times (y_1 \times E_1 \times A_1 + y_2 \times E_2 \times A_2)} \]  

(7)

\[ a_1 = \frac{(h_1 + h_2)}{2} - a_2 \]  

(8)

Where:

- \( b \) is the width of the member in m
- \( h \) is the height of the member in m

**Figure 3** Assumed deflected shape of the connecting member

**Figure 4** The assembly of the composite using the beam element method

For the consequences of using steel and timber will make it easier for designers to assess the stiffness of the connector into account, based on a simply-supported beam with uniform distributed loading where the composite section stretches from support to support.

**Eurocode 5 also has equations that may be used to determine the stresses in the individual members, so that they may be checked for strength. From these equations it is possible to determine the bending moment and axial force components in each member.**

The theoretical stiffness of the connector can be calculated by using the equations given in Table 7.1 of BS EN 1995-1-1:2004.

- **Screws and nails with pre-drilling:**
  \[ K_{ser} = \frac{\rho_m^{1.5} \times d}{23} \]  

(9)

- **Nails (without pre-drilling):**
  \[ K_{ser} = \frac{\rho_m^{1.5} \times d^{0.8}}{30} \]  

(10)

Where:

- \( K_{ser} \) is the stiffness of the screwed connection in kN/m
- \( d \) is the screw diameter in mm
- \( \rho_m \) is the mean mass density in kg/m³

It should be noted that this equation is unit-dependent. Clause 3 of BS EN 1995 allows the stiffness for steel-to-timber connections \( K_{ser} \) to be multiplied by 2.0.

**FINITE ELEMENT MODELS**

The following two finite element methods can be used to predict the stiffness of the combined section:

- **Beam elements**, where the centreline of the timber beam and the centreline of the steel strip or plate are connected by means of elements that have the same flexural stiffness as the shear stiffness of the connector.

- **Shell elements**, where the timber beam that is divided into shell elements is connected to the steel elements by means of three-dimensional axial and moment springs that model the shear stiffness of the connector.

The shell element method will give the stiffness as well as the stresses in the members, whereas the beam element method will give the axial forces as well as the bending moments in each element.

**Beam element method**

The composite beam can then be assembled as shown in Figure 4. The connecting elements between the centreline of the timber beam and the centreline of the steel plate are given a stiffness of \( K \) (as calculated in Equation 12), and are spaced at the required spacing of the nails or screws. They also need to have a high axial stiffness to simulate the contact pressure between the plate and the timber beam and ensure that the steel plate is forced to the same deflection as the underside of the timber beam. The authors propose that the Eurocode connection stiffness \( K_{ser} \) be used for \( K \) in this method where testing of the connector stiffness is not possible.

The method that is used to define the connector must take cognisance of the way in which the connection element is deformed. The connector may be placed through a pre-drilled hole in the thin steel plate and screwed or nailed into the timber beam. When screws are used, the top of the hole may be bevelled to accommodate the screw head. This bevel will allow for some rotation of the screw head to take place, whereas the shaft into the timber can be seen as fixed to the centreline of the timber beam. The stiffness of the connector can then be defined in terms of the stiffness of the connecting spring. Figure 3 shows the assumed deflected shape of the connector.

The assumption is made that the connector bends in double curvature.

The stiffness of the connection \( K \) is given by \( F/\Delta \). Standard deflection tables in the
Steel Designer’s Manual (1983) may be used to calculate the deflection in terms of the stiffness of the connecting element AB. The deflection is given by:

\[ \Delta = \frac{F \times x^3}{12 \times EI} \]  

(11)

The stiffness of the connecting element EI is then given by:

\[ EI = \frac{F \times x^3}{12 \times \Delta} = \frac{K \times x^3}{3} \]  

(12)

Where:

- \( F \) is the longitudinal shear force at the interface between the two members in kN
- \( x \) is the distance between the centrelines of the connected members in m
- \( EI \) is the stiffness of the connector in kN.m²
- \( K \) is the shear stiffness of the connector in kN/m

\( \Delta \) is the displacement of the connector under the shear force in m

**Shell elements**

The composite beam can be assembled as shown in Figure 5. The timber beam is divided into shell elements, the dimensions keeping within the bounds of the standard element sizes as suggested by Brooker (2006). The width of the beam is the shell thickness. The steel plate is then also divided into the correct proportions. On the contact surface between the timber elements and the steel elements, springs are placed so that the shear stiffness of the springs \( K \) is the same as the expected stiffness of the screwed connection. The connector shear stiffness \( K \) is calculated in Equation 12 in the X direction, has a large stiffness in the Y direction and no moment stiffness. A high axial stiffness (Y direction) is required to simulate the contact pressure between the plate and the timber beam and ensure that the steel plate is forced to the same deflection as the underside of the timber beam. The authors propose that the Eurocode connection stiffness \( K_{ser} \) be used for \( K \) in this method where testing of the connector stiffness is not possible.

**INITIAL EXPERIMENTAL VALIDATION**

Six South African pine specimens (3.1 m long, 36 mm × 149 mm), marked as Grade 5 but having a stiffness closer to Grade 7, as well as six 30 mm × 5 mm thick steel plates were selected for the initial evaluation of the proposed finite element methods. The steel plates had screw holes drilled at a spacing of 50 mm and were to be screwed to the underside of the timber beams. The spacing of the connectors was varied between 400 mm and 50 mm to ascertain the increase in stiffness of the composite with a reduction in connector spacing.

The stiffness and properties of all the components of the reinforced timber beam were measured:
- The stiffness of the connectors was measured.
- The equivalent modulus of elasticity of the steel reinforcement strips was determined.
- The density and modulus of elasticity of each timber beam were measured.

**Measured stiffness of connectors (screw joints)**

Fifty double-screw connections were constructed, as shown in Photograph 1, and these were tested for stiffness by measuring the relative displacement of the two lines on the steel plates under load with the help of clip extensometers. The results of the screw...
tests are given in Graph 1. The connector stiffness is calculated by determining the slope of the linear portion of the load-deflection curve. The linear portion is taken as the load divided by deflection between 10% and 40% of the ultimate strength of the connection. The density was calculated by accurately determining the dimensions and mass weight of each specimen and by dividing the mass by the volume.

The equivalent modulus of elasticity of the steel reinforcement strip with pre-drilled connector holes was determined by obtaining the tensile load versus elongation of the strip. The equivalent modulus of elasticity was determined from the stress-strain graph (see Graph 2 and Table 1). The full cross-sectional area of 5 mm × 30 mm was used to determine the equivalent modulus of elasticity (shown in Table 2). Three strips were tested.

Photograph 2(a) shows the testing of a steel reinforcement strip, and Photograph 2(b) shows where the mill scale on the steel strip has come off the steel, as well as the distortion around the holes. It was noteworthy to observe how the mill scale popped off the strip as the loading was increased to the yield stress and beyond.

### Density and modulus of elasticity of the timber beam

The timber pieces were weighed so that the average density could be determined. The stiffness of the connectors depends to a large extent on the density of the timber. Four-point loading was applied, and the deflection measured in the middle of the member so that the average modulus of elasticity could be calculated. The properties of the specimens are shown in Table 1.

Once the modulus of elasticity and density of the timber beams were determined, the stiffness of the connectors was calculated using the following two methods:

- The measured connector stiffness \( K_m \) determined by experimental testing:
  \[
  K_m = -162.7 + 2.55 \rho_m
  \]  
  (13)

Where:

- \( K_m \) is the experimentally determined stiffness of the connector in kN/m
- \( \rho_m \) is a mean density of the timber beam in kN/m²

- Connector stiffness determined by Eurocode method (Table 7.1 of BS EN 1995):

### Table 1 Equivalent modulus of elasticity of the steel strip with bevelled screw holes based on the full cross-sectional area

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Modulus of elasticity based on gross area (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>179</td>
</tr>
<tr>
<td>2</td>
<td>176</td>
</tr>
<tr>
<td>3</td>
<td>182</td>
</tr>
<tr>
<td>Average test</td>
<td>179</td>
</tr>
</tbody>
</table>

**Graph 2** Stress strain curve for specimen 3

**Photograph 2(a)** Testing steel reinforcement strip

**Photograph 2(b)** Steel strip before and after testing
Table 2 Properties of initial timber specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Modulus of elasticity</th>
<th>Density</th>
<th>Measured screw stiffness</th>
<th>Eurocode* screw stiffness</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>E (MPa)</td>
<td>$\rho_m$ (kg/m³)</td>
<td>$K_m$ (kN/m)</td>
<td>$K_{ser}$ (kN/m)</td>
</tr>
<tr>
<td>1</td>
<td>9 357</td>
<td>511</td>
<td>1 140</td>
<td>1 758</td>
</tr>
<tr>
<td>2</td>
<td>12 747</td>
<td>602</td>
<td>1 372</td>
<td>2 248</td>
</tr>
<tr>
<td>3</td>
<td>10 327</td>
<td>522</td>
<td>1 168</td>
<td>1 815</td>
</tr>
<tr>
<td>4</td>
<td>12 315</td>
<td>514</td>
<td>1 148</td>
<td>1 773</td>
</tr>
<tr>
<td>5</td>
<td>13 993</td>
<td>670</td>
<td>1 546</td>
<td>2 639</td>
</tr>
<tr>
<td>6</td>
<td>13 195</td>
<td>509</td>
<td>1 135</td>
<td>1 747</td>
</tr>
</tbody>
</table>

* The Eurocode values are the values prior to multiplying by 2

Test procedure of the composite beams

The testing for stiffness was done by means of four-point loading and the beams were loaded to the maximum service stress, i.e. about 45% of the characteristic stress of the timber alone. Test specimens were estimated to be of Grade 7, so the beams were loaded to a stress of about 7 MPa. This was well within the elastic stress range of the composite beam. The 3.1 m long beams spanned 3.0 m, with the steel having a length of 2.9 m so that it would not rest on the supports. The authors did not want the plates to rest on top of the end supports, as they were worried that the friction forces induced at the supports could lead to a slightly higher stiffness, and none of the analytical models take friction into account. Deflection was measured in the middle of the span and the load-deflection increases were measured at 10 Hz. Initial fixing of steel plates was with a screw spacing of 400 mm, and the tests were then repeated on each specimen with the screws at 200 mm, 100 mm and 50 mm. This was accomplished by halving the spacing after each test had been completed. A total of six beams were used, giving a total of 24 test results. The experimental setup is shown in Figure 6 and Photograph 3.

The screws were tightened fully and then loosened a little so that frictional forces between the steel and the timber would be minimised. In the actual construction of the steel-reinforced timber composite, one would expect the steel plates to initially be fixed tightly to the timber beam. Friction between the interfaces could increase the stiffness. Over time, the cross-sectional dimensional changes of the timber as a result of moisture variation will create a gap at the connection interface, thereby eliminating the friction.

Each specimen was then tested in the same way until all six specimens of each configuration had been tested. The composite beam stiffness was then calculated using Equation 6 from the standard stiffness tables in the Steel Construction Handbook.

$$EI = \frac{23}{1 296} \times \frac{W \times L^3}{\delta} \quad (15)$$

Where:
- $W = \text{total point load, placed as W/2 at thirds of the span}$
- $\delta = \text{the deflection measured at midspan}$
- $L = \text{the span of the beam}$
- $EI = \text{the flexural stiffness of the composite beam}$

$$K_{ser} = \frac{\rho_m^{1.5} \times d}{23} \quad (screws \ and \ nails \ with \ pre-drilling) \quad (14)$$

$\rho_m$ is the mean density of the timber beam in kg/m³

$K_{ser}$ is the stiffness of the connector in kN/m

Table 2 shows the properties of the timber beams used, and the calculated connector stiffness using the two methods described above.
Comparison of test results with finite element analyses and Eurocode equations

Only the flexural rigidity of Specimen 1 is shown to illustrate the good correlation between the measured results, the two finite element models where the connector stiffness is modelled, and the Eurocode equations (see Table 3). Both the calculated connector stiffness \( K_{ser} \) and the measured connector stiffness \( K_m \) have been used for the Eurocode method. PROKON (2014) software was used for all the finite element models.

The following properties were used for the analysis:

- \( E = 9357 \text{ MPa} \)
- \( 30 \text{ mm} \times 5 \text{ mm} \) steel plate with \( E = 179 \text{ GPa} \)
- \( K_{ser} = 140 \text{ kN/m} \) from Eq 13 with density of 511 kg/m\(^3\)
- \( I_{connector} = 2.43 \times 10^{-10} \text{ m}^4 \) from Eq 12 with the element in double flexure
- \( K_m = 7588 \text{ kN/m} \) from Eq 14 and this may be multiplied by 2 (Section 7.7.3 (1) of BS EN 1995-1-1:2004)

As can be seen from Table 3, there is very good correlation between the measured stiffness \( EI \) and the Eurocode method, as well as the proposed finite element methods. However, the Eurocode screw stiffness equation, where the stiffness using steel plates is two times the timber-to-timber stiffness, could overestimate the stiffness by 20%. The other methods, using the Eurocode stiffness for timber-to-timber (i.e. without multiplying by 2), are in the region of 2% of the tested value.

FURTHER EVALUATION OF PROPOSED FINITE ELEMENT MODELS

In order to further validate the three proposed design methods, tests on a further 18 timber beams, reinforced with steel plates, were performed. The timber beam size was kept the same as the initial tests, and the connector type and steel plate thickness were varied. (Human 2013; Huang 2014; Meintjies 2014; Abdul 2014).

Three variables play a part in the stiffness of the composite member, these being the stiffness of the steel plate, the stiffness or spring constant of the connection, and the stiffness of the timber member. The modulus of elasticity of the steel plate has little variability, which reduces the variables that would influence the stiffness of the composite to two, namely timber stiffness and connection stiffness. By reducing the variables to two, it was felt that a steel-reinforced timber beam would give a good indication of whether the proposed analysis methods have any merit.

Table 3 Comparison of measured stiffness to modelled stiffness

<table>
<thead>
<tr>
<th>Screw spacing</th>
<th>Measured EI</th>
<th>Beam element method EI using ( K_{ser} ) (1 140 kN/m)</th>
<th>Shell element method EI using ( K_{ser} ) (1 140 kN/m)</th>
<th>Eurocode EI using ( K_m ) (1 140 kN/m)</th>
<th>Eurocode EI using 2( K_m ) (2 × 1 758)</th>
</tr>
</thead>
<tbody>
<tr>
<td>mm</td>
<td>kN.m²</td>
<td>kN.m²</td>
<td>kN.m²</td>
<td>kN.m²</td>
<td>kN.m²</td>
</tr>
<tr>
<td>400</td>
<td>104.18</td>
<td>107.80</td>
<td>108.9</td>
<td>106.33</td>
<td>125.5</td>
</tr>
<tr>
<td>200</td>
<td>115.97</td>
<td>116.59</td>
<td>117.7</td>
<td>116.67</td>
<td>142.5</td>
</tr>
<tr>
<td>100</td>
<td>129.74</td>
<td>130.03</td>
<td>130.9</td>
<td>131.58</td>
<td>160.0</td>
</tr>
<tr>
<td>50</td>
<td>145.85</td>
<td>148.58</td>
<td>147.4</td>
<td>149.25</td>
<td>174.4</td>
</tr>
</tbody>
</table>

Table 4 Summary of various test steel plate and connector variation

<table>
<thead>
<tr>
<th>Timber size width x height</th>
<th>No of specimens</th>
<th>Steel plate dimensions width x thickness</th>
<th>Connector type</th>
</tr>
</thead>
<tbody>
<tr>
<td>36 mm × 149 mm</td>
<td>6</td>
<td>30 mm × 5 mm</td>
<td>40 mm long, 3.5 mm particle-board screws</td>
</tr>
<tr>
<td>36 mm × 149 mm</td>
<td>6</td>
<td>30 mm × 4.5 mm</td>
<td>3 mm high-tensile fluted masonry nails</td>
</tr>
<tr>
<td>36 mm × 149 mm</td>
<td>6</td>
<td>30 mm × 8 mm</td>
<td>3 mm high-tensile fluted masonry nails</td>
</tr>
<tr>
<td>36 mm × 149 mm</td>
<td>6</td>
<td>30 mm × 10 mm</td>
<td>3 mm high-tensile fluted masonry nails</td>
</tr>
</tbody>
</table>

Table 5 Modulus of elasticity (MOE) and density of timber specimens used in experimental validation. Note that the MOE of the timber reinforced with 5 mm and 8 mm steel plates is very high. All the timber used was marked as Grade 5 and was obtained from a local supplier. The high variation in \( E \) value is noted and attributed to the timber being graded visually for strength.

<table>
<thead>
<tr>
<th>Specimen number</th>
<th>4.5 mm thick plate</th>
<th>5 mm thick plate</th>
<th>8 mm thick plate</th>
<th>10 mm thick plate</th>
</tr>
</thead>
<tbody>
<tr>
<td>E (MPa)</td>
<td>Density (kg/m³)</td>
<td>E (MPa)</td>
<td>Density (kg/m³)</td>
<td>E (MPa)</td>
</tr>
<tr>
<td>1</td>
<td>11 037</td>
<td>551</td>
<td>9 695</td>
<td>511</td>
</tr>
<tr>
<td>2</td>
<td>10 267</td>
<td>607</td>
<td>12 745</td>
<td>602</td>
</tr>
<tr>
<td>3</td>
<td>6 815</td>
<td>481</td>
<td>10 625</td>
<td>322</td>
</tr>
<tr>
<td>4</td>
<td>5 684</td>
<td>478</td>
<td>12 305</td>
<td>514</td>
</tr>
<tr>
<td>5</td>
<td>8 665</td>
<td>519</td>
<td>14 220</td>
<td>670</td>
</tr>
<tr>
<td>6</td>
<td>7 093</td>
<td>495</td>
<td>13 480</td>
<td>508</td>
</tr>
</tbody>
</table>

Table 6 Mean and characteristic modulus of elasticity of the test specimens compared to the values given in SANS 10163

<table>
<thead>
<tr>
<th>Modulus of elasticity (tested timber)</th>
<th>SANS 10163 modulus of elasticity (Grade 5)</th>
<th>SANS 10163 modulus of elasticity (Grade 7)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MPa</td>
<td>MPa</td>
<td>MPa</td>
</tr>
<tr>
<td>Mean</td>
<td>9 148</td>
<td>7 800</td>
</tr>
<tr>
<td>Characteristic (5th percentile)</td>
<td>5 878</td>
<td>4 630</td>
</tr>
</tbody>
</table>

Table 4 shows the specimens with the different steel plate thicknesses and the type of connectors used.

To increase the number of test values, the authors decided to gradually decrease the spacing of the screws from a spacing of 400 mm, to 200 mm, to 100 mm and finally to 50 mm. This would then give four test results for every specimen. A total of 24 beams were used with a total of 96 test results. The experimental setup was similar to the setup in Figure 6.

Prior to assembly of the composites, the stiffness and density of each piece of timber
were determined (see Table 5). This was done using the standard four-point loading test on edge. The stiffness that was obtained from each individual test, with each spacing of the connector, was compared to the stiffness that had been obtained by using the three proposed design methods – using the characteristic modulus of elasticity of the tested timber, the stiffness of the steel based on the full area of the section, and the stiffness of the nail or screw using the Eurocode dowel connector stiffness for dowels through steel. Characteristic values were used, as a practising engineer cannot be expected to measure the modulus of elasticity of each individual piece of timber, or the reduced modulus of elasticity of the steel.

The method proposed by Leicester (1986) was used to determine the lower-bound characteristic value of the modulus of elasticity, as well as the density (see Table 6) from the experimental results. These values were used in the analytical Eurocode method and the two finite element methods to determine the characteristic stiffness of the composite beams. It is noted that the experimental values differ from the values in SANS 10163 for a Grade 5 timber, as they are closer to a Grade 7.

The mean density of the tested timber was 541 kg/m³ and the characteristic density was 444 kg/m³. SANS 10163 gives minimum densities for Grade 5 as 360 kg/m³ and Grade 7 as 425 kg/m³. More important is that the mean density for Grade 5 would be in the region of 425 kg/m³ and 500 kg/m³ for Grade 7.

The theoretical stiffness of the screws and nails can be calculated by using Equations (9) and (10) given in SANS 10163: Part 1 or Table 7.1 of BS EN 1995.

Clause (3) of BS EN 1995 allows the stiffness for steel-to-timber connections $K_{ser}$ to be multiplied by 2.0. However, using the initial test results it was decided to not multiply $K_{ser}$ by 2, as the stiffness of the initial tests was overestimated when this factor was applied.

For the 3.5 mm screw in mean density of timber = 516 kg/m³, the stiffness $K_{ser}$ will be 1 784 kN/m, which is the value used in the analyses.

For the 3.0 mm nail, without pre-drilled holes, in mean density of timber = 516 kg/m³, the stiffness $K_{ser}$ will be 941 kN/m, which is the value used in the analyses.

**COMPARISON**

Graphs 3, 4, 5 and 6 show the measured stiffnesses for the composite beams with various connectors, connector spacing and steel plate thicknesses. They also show the theoretical stiffnesses calculated using the three different proposed design methods,
and the stiffnesses calculated assuming full fixity and no connection.

All the theoretical values have been calculated using the measured MOE_{mean} and MOE_{5%} for the tested timber. The SANS 10163 MOE_{mean} and MOE_{5%} values for Grade 5 timbers are not compared, as the intent of this work was to compare test values against the proposed modelling methods. For the shell and beam elements, only the fifth percentile MOE has been used to avoid confusion between the data and the theoretical.

Where the test results are higher than the stiffness based on Eurocode 5 equations using MOE_{mean}, it is because the timber had a higher stiffness than the MOE_{mean} of the sample. Beam 6 of the sample using 8 mm steel plates (Graph 5) had an exceptionally high MOE and tested above the fully-fixed case. Only one test specimen had a modulus of elasticity that was less than the MOE_{5%} of the sample and that was beam 4 (Graph 6).

Comparing the four graphs, the following can be noted:

- The increase in stiffness of the composite beam as the spacing of the connectors decreases.
- Full fixity can theoretically be achieved when connector spacing tends towards zero; however, this would not be practical.
- There is a significant difference in predicted stiffnesses depending on whether MOE_{mean} or MOE_{5%} is used, and many of the test values fall below the predicted value for MOE_{mean} and connector stiffness based on the mean density.
- The MOE_{5%} calculated stiffnesses correlate with the lower-bound curve from the experimental results, indicating a safe design.
- There is a very good correlation between the two finite element models and the Eurocode spring model, as can be seen on the lower-bound curves of Graphs 1 to 4.
- The MOE value of the timber has a significant effect on the stiffness of the composite beam. The timber used for the 8 mm plate beams had high E values, and thus were stiffer composite beams when compared to the composite beam with the 10 mm thick plate which used timber with a lower MOE.

**CONCLUSION**

The objective of this investigation was to find suitable analytical methods that could be used by structural engineers to obtain an acceptable effective cross-sectional stiffness of a composite timber-steel beam, when using flexible connectors.

Three design methods were used in this paper and these were compared to test values. The methods in order of complexity are:

- Graph 5: Stiffness of composites with 3.0 mm nails and 30 mm x 8.0 mm steel plate
- Graph 6: Stiffness of composites with 3.0 mm nails and 30 mm x 10 mm steel plate; note that the E for Beam 4 was less than the E_{5%}
A spring model as given in Eurocode 5
A finite element model using beam elements
A finite element model using shell elements.

Engineers or designers who do not have that much experience with timber design can easily understand the methods presented. The finite element methods have an added advantage in that it makes it possible for the designer to specify non-linear behaviour for the connectors when investigating failure of the composite element.

The three design methods all use the following properties, which are easily obtained from design codes or other literature without testing:

The 5th percentile modulus of elasticity of timber
The mean density for the grade of timber
The Eurocode connector stiffness equation which is based on the mean density
The full cross-section of the steel member may be used with no loss of area or second moment of area due to holes.  

Note that the Eurocode 5, BS EN 1995-1-1:2004 method for the stiffness of screwed and nailed connections, which is given in Equations 9 and 10, should not be multiplied by 2 as recommended in the code when connecting a steel plate to timber, as the stiffness is then overestimated.

It was concluded that the stiffness, given by the Eurocode spring method and both the beam and the shell finite element methods, showed a remarkably good fit with the lower-bound experimental values if the characteristic modulus of elasticity is used. Using these methods for design would result in a conservative assessment of the stiffness of a composite timber and steel beam. It was also noted that the full fixity condition cannot be achieved in reality with a practical connector spacing.

The Eurocode 5 method for determining composite stiffness is specifically for a simply-supported member subjected to uniformly distributed loading, with the composite section stretching from support to support. The authors find it encouraging that it is possible to use the fairly simple finite element methods to obtain the composite stiffness with some measure of assurance. It gives the designer the option of designing for a specific loading case, as well as giving the designer the possibility of using the finite element methods to analyse continuous multiple-span members, and also members where the composite does not necessarily stretch from support to support.

LIST OF NOTATIONS

- **b** = the width of the member in m
- **d** = the screw diameter in m
- **E** = the modulus of elasticity
- **EI** = the flexural stiffness of the composite beam
- **EI_C** = the combined stiffness of the section with no slip on the interface
- **E_S** = the modulus of elasticity of the steel
- **E_T** = the modulus of elasticity of the timber
- **F** = longitudinal shear force at the interface between the two members (kN)
- **f** = frequency (Hz)
- **h** = the height of the member in m
- **I** = the second moment of area of the board on flat
- **I_S** = the second moment of area of the steel about its own axis
- **I_T** = the second moment of area of the timber about its own axis
- **K** = the stiffness of the connector in kN/m
- **K_ser** = the stiffness of the connector in kN/m (using Eurocode)
- **K_m** = the stiffness of the connector in kN/m (determined experimentally)
- **L** = the span of the beam
- **y_S** = the distance from the combined neutral axis to the neutral axis of the steel
- **y_T** = the distance from the combined neutral axis to the neutral axis of the timber
- **NA_T** = the position of the neutral axis for the timber on its own
- **NA_S** = the position of the neutral axis for the steel on its own
- **NA_C** = the position of the neutral axis of the ideally combined section
- **s** = the spacing of the connectors in m
- **t** = thickness of the board (m)
- **w** = mass of the board (kg)
- **W** = point load
- **x** = distance between the centrelines of the connected members (m)
- **ΔW** = change in the point load
- **γ** = a connector stiffness parameter
- **μ_m** = the mass density in kg/m³
- **Δδ** = the change in the deflection that matches the change in the point load, measured in the middle of the span
- **δ** = the deflection at the middle of the span
- **Δ** = the displacement of the connector under the shear force in m

REFERENCES

Abdul, J 2014. The prediction of the true effective stiffness of composite timber and steel beams. BEng Project Report, University of Pretoria, Department of Civil Engineering.


Huang, J 2014. The prediction of the true effective stiffness of composite timber and steel beams. BEng Project Report, University of Pretoria, Department of Civil Engineering.

Human, W 2013. The prediction of the true effective stiffness of composite timber and steel beams. BEng Project Report, University of Pretoria, Department of Civil Engineering.


Meintjes, J 2014. The prediction of the true effective stiffness of composite timber and steel beams. BEng Project Report, University of Pretoria, Department of Civil Engineering.


A risk and cost management analysis for changes during the construction phase of a project

S Schoonwinkel, C J Fourie, P D F Conradie

In civil construction projects, changes are inevitable, impacting projects in terms of cost, time and quality. It is nevertheless expected of project managers to effectively manage the impacts of project changes, and to complete the project within the project constraints, despite such changes. This article explores the impact of changes on a project by comparing the findings from a South African case study to the impact of changes found in literature. The article further investigates how consulting engineers in the Western Cape deal with changes in projects, and how cost risk management is performed during changes. The findings are startling and expose the shortage of necessary skills and competencies within project management. A fresh approach is required for project managers to deal effectively with project change.

INTRODUCTION

One of the primary ways in which value is generated in modern societies, is through projects that create physical assets, such as factories, commercial buildings, hospitals, schools and highways, which can then be exploited to social and economic ends. Most of these assets are created through construction projects, and as the size and complexity of these projects increase, a more intensive level of project management is required to successfully meet the expectations of time, cost and quality (Winch 2010).

However, managing a construction project is difficult in that all the relevant information is not always available at the initial stage of the project to plan and design the project accurately and make the best possible decisions. As information becomes available during the construction phase of the project, it can lead to various changes, which can in turn affect productivity, planned schedules, deadlines, work methodology, resource procurement and budget, all of which could result in the project objectives not being achieved. Design errors or variations, unforeseen site conditions and vagueness in the original scope are merely some of the reasons for change.

It is expected of a project manager to effectively manage the cost, time and risk impacts of all project changes, and complete the project within the project constraints regardless of any challenges. To manage projects more effectively, a fresh approach in project management is required. A project manager must understand the implications of changes and must manage these changes in such a way that all the project objectives are obtained within the time, budget and quality constraints.

Over and above the cost, time and risk consequences, changes can also affect stakeholder relationships and team morale. The uncertainties associated with change are often the result of iterative cycles or further changes due to unanticipated side-effects of the current change during the construction process (Lee et al 2005). It is thus imperative to understand change, the types of changes, its impact on the project, and how to analyse, manage and control it.

The aim of this article is therefore to determine the following two aspects about changes made to the works during the construction phase of a civil engineering project:
- What is the potential impact of project change?
- How are changes to the works currently managed in practice?

METHODOLOGY

In order to determine the impact of project change, the relevant literature was reviewed and a case study was done on a civil engineering construction project. The case study comprised the construction phase of a recently completed civil and structural construction project in South Africa, and the data for the case study was obtained from the consulting engineers who designed and managed the project. For the sake of confidentiality, all names of the stakeholders involved in the project have been omitted from this article.
To be able to understand the project management environment and the state of change management in practice, the same case study was used and various interviews with project managers were done. The case study was analysed to understand how project finances were managed, the reasoning behind the particular management approach, its effectiveness, and its shortcomings.

To understand how the management of cost and risk, as a result of changes, are currently done in practice, 18 project managers were interviewed. These were mostly directors of consulting firms who are actively involved in the market place. The semi-structured interviews consisted of a questionnaire to determine project managers’ experience and modus operandi in managing the costs and risks of change. The results are reported in this article.

IMPACT OF PROJECT CHANGE

It is important that project managers understand the impact of change on a project. Project managers cannot make informed decisions regarding change without knowing what the effect of the change will be on the project. Changes that are mismanaged could prevent a project from achieving its objectives (Love et al 2002).

Figure 1 illustrates the influence that various factors can have on the project life cycle. It indicates the relationship of stakeholders’ influence, risk and uncertainty against project time, as well as the cost of changes in relation to project change. This graph demonstrates that the impact of change becomes greater as the project progresses in time. The influence of project stakeholders, uncertainty and risk, reduces with time as the unknowns become less and the objectives are more clearly defined. Changes that happen during the construction phase of a project therefore have a greater cost impact on a project than changes that happen during the initial phases.

A study on the impact of project change conducted by Ibbs (1997) concluded the following:

■ As the number of changes increase, costs will also increase.
■ As change increases on a project, productivity decreases.
■ Change that occurs during the construction phase of a project has a more disruptive impact on the project than change that occurs during the design phase of a project.
■ A project that has a large number of changes would have less efficient implementation of those changes.

Table 1

<table>
<thead>
<tr>
<th>Cause of Variation</th>
<th>No of Events</th>
<th>Non-productive time</th>
<th>Total Cost ($)</th>
<th>Mean cost per event ($)</th>
<th>Percentage of contract value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>#</td>
<td>% of total</td>
<td>Days</td>
<td>% of total</td>
<td>Cost ($)</td>
</tr>
<tr>
<td>Client changes</td>
<td>49</td>
<td>18</td>
<td>10</td>
<td>26</td>
<td>105 620</td>
</tr>
<tr>
<td>User changes</td>
<td>132</td>
<td>48</td>
<td>14</td>
<td>36</td>
<td>235 440</td>
</tr>
<tr>
<td>Design omissions</td>
<td>83</td>
<td>30</td>
<td>13</td>
<td>33</td>
<td>265 980</td>
</tr>
<tr>
<td>Local authorities</td>
<td>5</td>
<td>2</td>
<td>2</td>
<td>5</td>
<td>146 080</td>
</tr>
<tr>
<td>Extension of time</td>
<td>6</td>
<td>2</td>
<td>0</td>
<td>0</td>
<td>53 240</td>
</tr>
<tr>
<td>Total</td>
<td>275</td>
<td>(100)</td>
<td>39</td>
<td>(100)</td>
<td>806 360</td>
</tr>
</tbody>
</table>

Note 1: Includes rates, taxes and fees

Table 2

<table>
<thead>
<tr>
<th>Cause of Variation</th>
<th>No of events</th>
<th>Non-productive time</th>
<th>Total Cost ($)</th>
<th>Mean cost per event ($)</th>
<th>Percentage of contract value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>#</td>
<td>% of total</td>
<td>Days</td>
<td>% of total</td>
<td>Cost ($)</td>
</tr>
<tr>
<td>Design change</td>
<td>65</td>
<td>30</td>
<td>20</td>
<td>29</td>
<td>182 893</td>
</tr>
<tr>
<td>Design error</td>
<td>12</td>
<td>6</td>
<td>13</td>
<td>19</td>
<td>59 233</td>
</tr>
<tr>
<td>Design omission</td>
<td>2</td>
<td>1</td>
<td>7</td>
<td>10</td>
<td>6 837</td>
</tr>
<tr>
<td>Construction change</td>
<td>14</td>
<td>6</td>
<td>2</td>
<td>3</td>
<td>72 979</td>
</tr>
<tr>
<td>Construction error</td>
<td>120</td>
<td>55</td>
<td>14</td>
<td>20</td>
<td>19 514</td>
</tr>
<tr>
<td>Construction omission</td>
<td>2</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>760</td>
</tr>
<tr>
<td>Construction damage</td>
<td>3</td>
<td>1</td>
<td>14</td>
<td>20</td>
<td>3 288</td>
</tr>
<tr>
<td>Total</td>
<td>218</td>
<td>70</td>
<td>345 504</td>
<td>1 584</td>
<td>3.15</td>
</tr>
</tbody>
</table>

Case study by Love et al (2002)

Love et al (2002) did a case study on a residential construction project (two six-storey residential apartment blocks, containing a total of 43 units) in order to better understand change and rework in construction project management.

Table 1 indicates that there were 275 items of change which resulted in 39 non-productive days and a 7.35% increase in the cost of the works. Table 2 indicates that there were 218 items of rework which resulted in 70 non-productive days and a total additional cost of $345 504.00, which is 3.15% of the...
total contract value. The changes and rework of the project had a combined 10.5% cost impact on the project and resulted in 109 non-productive days.

Change could also result in rework. Construction is the physical manifestation of a design, and thus rework usually entails the demolition or modification of work already constructed. For this reason, rework is perceived to have a greater impact on construction performance than change. When project managers are under time or resource constraints, they would rather avoid rework by modifying the design and specifications. In the case study under review, the value spent on changes is more than double the amount spent on rework. It is thus clear that change may have a greater impact on the works than rework (Park & Peña-Mora 2003).

The case study therefore made the following findings:

- Change can have a significant cost and time impact on a construction project.
- Change has a greater impact than rework.

**Case study conducted by the authors**

The authors also did a case study to determine the impact of project change. The project reviewed for the case study was the construction of a multi-million rand integrated industrial facility for a metropolitan municipality. The project was designed and managed by civil and structural consulting engineers in a joint venture with mechanical and electrical consulting engineers. The contract covered the construction of the structures, civil works and infrastructure, as well as the building-related mechanical, electrical and electronic works. Separate contracts covered the provision of specialised mechanical and electrical infrastructure. This contract was based on the General Conditions of Contract (SAICE 2004), and the tendered civil and structural contract value was in the order of R160 million (including VAT and contingencies), with a construction period of 16 months.

For the purpose of the case study the researchers reviewed all the site memorandums (SMs), requests and quote rates (QRs) and variation orders (VOs) against all the new work and changes made to the project during the construction phase. Other available data (such as documentation, minutes of meetings, etc.) was also perused for a better understanding of the project. The data was used to determine how many changes occurred on the project, how these were managed, and what the cost impact was.

For this project the main schedule of quantities (SOQ) had 753 original pay items. Each pay item of the SOQ has a tender quantity, rate and amount value (amount value is defined as the tendered quantity multiplied by the tendered rate). All the item amounts add up to the tendered value of the contract.

During construction of this project, 445 new pay items were added to the SOQ, and 212 pay items of the SOQ were never claimed and were thus omitted from the works. All these items were captured in the VOs. This data is reflected in Table 3.

Table 3 also indicates that the number of tendered SOQ pay items increased with a significant 59%, and that 28.2% of the original pay items were never claimed during the construction period of the project. These values clearly indicate that there were a substantial number of changes made to the project.

In this specific case extensive design changes were made to the project, due to the impact of the design requirements from the mechanical and electrical plant contracts, which impacted the civil and structural works of the contract under revision. One main reason for such a large value of omissions on the project was the reduction in the clients’ available budget for the works. This only became known after receiving the tenders, and therefore necessitated various changes and omissions to the initial design.

To determine the impact of changes to the project, the changes had to be quantified. The value of all the works added to the project; any extension of time or cost to the project, all omissions, as well as cost of variation between the tendered and claimed values of the scheduled work had to be determined. These costs could then be expressed as a percentage of the tendered contract value in order to determine the impact of the changes on the project, as can be seen in Table 4.

As seen in Table 4, the value of additional costs to the project is 13.1% of the total contract value, which is significantly higher than the 10% contingency usually allowed for additional works. However, there was an equal reduction to costs of 13.1% due to work omissions. Of the R17 720 274.96 saved due to omissions, R10 120 000.00 (7.46% of the tendered project value) was forced omissions due to a budget reduction by the client. There was also a 2.6% saving on the project due to changes to the initial tendered quantities, resulting in an overall 1.8% decrease in the tendered value of the project.

The cost impact of changes can further be analysed by taking a look at the rate categories of new items added to the project, as given in Figure 2, and the total value of the items per category, as given in Figure 3.

The majority of items (89% of the 445 new items of work) have a value of less than 0.05% of the contract’s tendered value. They represent only 28% of the value of additional work added to the project, as shown in Figure 3. Whereas only 1% of the additional items to the project have a value greater than 0.5% of the contract’s tendered value, these changes amount to 30% of the total value.

### Table 3 Information of item changes to the project

<table>
<thead>
<tr>
<th>Item information</th>
<th>Description</th>
<th>Value</th>
<th>% of tendered project value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of original pay items in the tender SOQ</td>
<td>753</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of new pay items added to the SOQ</td>
<td>445</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of original pay items in the tender SOQ that were never claimed</td>
<td>212</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total number of pay items that represent all the changes to the project</td>
<td>657</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Increase in rates claimed due to new rates added to SOQ</td>
<td>59.1%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Decrease in rates claimed due to omission of tender rates from SOQ</td>
<td>28.2%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table 4 Value of changes to the project

<table>
<thead>
<tr>
<th>Description</th>
<th>Value (R)</th>
<th>% of tendered project value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tendered value of project (excl VAT and contingencies)</td>
<td>135 660 389</td>
<td>–</td>
</tr>
<tr>
<td>Cost of project at completion</td>
<td>133 150 703</td>
<td>98.2%</td>
</tr>
<tr>
<td>Value of new works added to project</td>
<td>17 741 498</td>
<td>13.1%</td>
</tr>
<tr>
<td>Extension of time cost</td>
<td>1 009 330</td>
<td>0.7%</td>
</tr>
<tr>
<td>Total of additional costs to project</td>
<td>18 750 828</td>
<td>13.8%</td>
</tr>
<tr>
<td>Value of pay items omitted from project</td>
<td>–17 720 274</td>
<td>–13.1%</td>
</tr>
<tr>
<td>Value of quantity changes to tendered items in the SOQ</td>
<td>–3 540 239</td>
<td>–2.6%</td>
</tr>
<tr>
<td>Total of omission costs to project</td>
<td>–21 260 514</td>
<td>–15.7%</td>
</tr>
<tr>
<td>Total cost of changes to the project</td>
<td>–2 509 685</td>
<td>–1.8%</td>
</tr>
</tbody>
</table>
of additional works. The ‘medium high’ and ‘high’ categories (as seen in Figure 2) amount to 5% of the number of additional items, but they represent more than 50% of the total value of the items added to the project. These two categories thus have the greatest impact on the finances of the project.

As stated in Table 3, there are 753 pay items in the original tender SOQ. Of these 753 items, 212 items (28.2%) were omitted from the project, thus leaving 541 claimed items from the tender. The quantity variance of these claimed items will be broken up into further sub-categories. As can be seen in Table 5, 146 (19.4%) of the claimed tender items show a 15% decrease in their tendered quantities, resulting in a saving to the project. The investigation shows that 138 claimed tender items (18.3%) experienced more than 15% increase in the original quantities. Thus only 23.5% of all tendered items were claimed without any variance in quantity or cost.

The contract allowed for all pay items to be re-measurable. The tender quantities listed are based on estimates done during the design phase of the project. However, when the design changes, the quantities might also change.

Table 6 indicates a significant cost implication for all tendered items with a decrease or increase to the quantities of more than 15%. These changes to the item quantities amount to more than 10% of the tendered project value. It is thus clear that quantity changes of more than 15% can have a significant impact on the project.

Based on the data given above, the findings of the case study can be summarised as follows:

- Projects can have a significant number of changes.
- Similar to Love et al (2002), the authors found that the impact of changes on cost can be significant.
- Changes with a value greater than 0.1% of the total contract value can have a huge impact on the finances of the project.
- Quantity changes of more than 15% have a significant cost impact on the project.

**Change Management in Current Practice**

During a construction project there may be quite a number of changes. Regardless of the size of the change, each alteration to the works has a cost, time, quality and risk implication. Due to tight time constraints on most projects, every change requires quick, robust decision-making, so as to not delay the project, which therefore results in changes not being comprehensively evaluated. Based on feedback from project managers, decisions are often made on intuition or experience, sometimes without an assessment of the risks involved or the influence on the cost of the project, and often without applying well-known project management techniques. Mainly because of

**Table 5 Quantity variance of items**

<table>
<thead>
<tr>
<th>Quantity variance of items between the tender SOQ and the final claimed SOQ (represented by Y)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of items in the tender SOQ</td>
<td>753</td>
</tr>
<tr>
<td>Number of original pay items in the tender SOQ that were omitted</td>
<td>212</td>
</tr>
<tr>
<td>Number of items in the tender SOQ that were used</td>
<td>541</td>
</tr>
<tr>
<td>Number of items for which Y ≤ -15%</td>
<td>146</td>
</tr>
<tr>
<td>Number of items for which -15% &lt; Y ≤ 15%</td>
<td>257</td>
</tr>
<tr>
<td>Number of items for which -15% &lt; Y &lt; 0</td>
<td>42</td>
</tr>
<tr>
<td>Number of items for which 0 &lt; Y ≤ 15%</td>
<td>38</td>
</tr>
<tr>
<td>Number of items for which Y = 0</td>
<td>177</td>
</tr>
<tr>
<td>Number of items for which Y &gt; 15%</td>
<td>138</td>
</tr>
</tbody>
</table>
time constraints (Akintoye & MacLeod 1997) project managers use contingencies and construction buffers as the only mechanism for dealing with unexpected change.

**Change management practices based on the case study**

To determine how the changes were evaluated and managed in the case study, the paperwork for all the changes was assessed. When a change is made to the project, it is recorded by the engineer in an SM, which gives a clear description of the change as well as the cost implication thereof (if available). The SM therefore serves as the record of the change and its implications, and is seen as an important document to the engineer. If the change involves work where no rate is specified in the tender SOQ, the engineer requests a quote for it in the SM. The contractor will then be required to supply the engineer with a contractor’s quote (CQ) for the works, and the CQ is then reviewed by the engineer to determine if it is fair and reasonable. These two references (SM and CQ) are important for managing new items.

Table 7 summarises the project costs for new items. Of the 445 new items added to the SOQ, 174 items (39%) were documented either in an engineer’s SM or a CQ, 51 (12%) items had both references, and 220 items (49%) had no paper-based record at all (refer to Figure 4).

Table 7 indicates that for 68.5% of the new items added to the SOQ, there is no record of any quote or information received for the works from the contractor and no indication of any form of review. These items amount to 31% of the value of all new works added to the project. These rates are added to the SOQ without any formal evaluation of its fairness towards the client or the impact on the project budget. It also implies that no cost, time or quality analyses were made of the changes, and it can furthermore be assumed that no risk analysis was done for these changes.

Table 7 also indicates that SMs were issued for only 30.6% of the new works, and thus the remaining 69.4% of the items (representing 63.6% of the value of the works) must have been initiated by another means, which is in contradiction of the project plan. It can therefore safely be assumed that the changes that necessitated these rates were not properly reviewed before the contractor was instructed to do the works, nor were alternative options investigated. The contractor was also not given any written information, instructions or specifications on the changes.

However, it must be noted that the contractor’s certificate was evaluated each month, and no rate could be added without the knowledge of the engineer. All items of change were also recorded in the VOs submitted to the client, compiled once a month in line with the monthly payment certificate (MPC). Even though the rates were not assessed in a formal, paper-based manner, they would have been acknowledged and reviewed at each MPC review. Certificate reviews consisted of detailed comparison of the changes between the quantities claimed in the current and the previous months, in order to assess the accuracy of the claim.

However, the review only examines items of work claimed in the certificate which have been initiated by another means, which is a reactive method and not the ideal way in which to manage cost changes.

The following concerns are raised for this case study:
- The method of change management is questionable.
- Not all changes are recorded in an acceptable manner.

**Change management practices based on interviews**

To examine how change management is currently being applied in practice, the
researchers also interviewed 18 project managers, of whom more than two thirds are directors of active firms in the market place. They were asked questions regarding their experience in project management, and more specifically, the cost and risk management of changes. The demographics of the group are illustrated in Figure 5.

**How are changes managed in general by engineers?**

Interviewees were requested to answer various questions with regard to their experience of the management of changes based on the company procedure. The results are shown in Figure 6. It is evident that changes are predominantly managed in two ways – through the requirements of the ISO 9000 certified quality management system (QMS), and a paper-based exercise of issuing SMs, VOs, QRs and MPCs (both these methods were stated by 50% of the interviewees). This substantiates the finding of the case study, which also found that changes are managed through the documenting and reviewing of SMs, VOs, QRs and MPCs. The results also indicate that, though there is a methodology or procedure in place, 33% of those interviewed admitted to not always following the set procedure prescribed by company protocol.

**How is the cost of changes managed?**

From the research it was found that not all changes are recorded in the appropriate manner, and that the cost and risk management of changes in most cases were inadequate. The participants generally regarded the following as methods by which they manage the cost of changes of new works:

- By doing a cost estimate of the proposed works.
- By asking for rates from the contractor and then assessing if the QRs are fair and reasonable.
- By determining the effect of the work on the project budget and contingencies.
- By determining if there are any other cost implications of the works, such as life cycle or time-related costs.

The results are shown in Figure 7, where only 33% of all participants determined the cost impact of the additional works on the overall project. The general opinion was that an initial cost estimate of the works is done and then the required rates are requested from the contractor. If the rate is found to be acceptable and within the allowed contingencies, it is added to the project. Quoting one of the participants: “You seldom go out to tender with a complete design, so you know there is going to be additional work, but you don’t know how much of your contingencies you are going to use.”

The interviewees were then questioned on the process followed to determine how reasonable a rate was on a CQ for new works. The following main methods were identified:

- By comparing the given rate to known rates from similar projects.
- By comparing the given rate to relevant rates from the project’s SOQ.
- By requesting that the contractor provides a breakdown of the QR.
- By acquiring supplier quotes for comparison to the QR.
- By judging the reasonableness of the rate based on experience.

Refer to Figure 8 for the results of the study. A rate breakdown, or the comparison of rates against those from similar projects or the existing SOQ, are similar methods to those identified by the case study and were thus expected.
How are the risks of the changes managed?

Risk management is a theoretical five-step process of planning, identification, assessment, determining the response, and monitoring and controlling the risks. However, when the project managers were asked how they manage risks, only 39% of interviewees followed this process, with more than 50% following no specific procedure. Project managers manage risks using the following methods (refer to Figure 9):

- Experience
- Crisis management
- Identification, analysis and mitigation
- Preventing scope creep
- Professional insurance.

One of the respondents replied: “Approximately 50% of our time is taken to manage things that go wrong, because everything is such a rush and we don’t do our planning properly. That goes for us as engineers, architects and contractors. You have to keep correcting things and direct them in the right way.”

This indicates that the risks pertinent to civil projects are not properly identified and managed, and most project managers do not follow the theoretical risk management process, rather relying on their experience and crisis management abilities.

What are the current difficulties with cost and risk management of changes?

Most participants commented on the time constraints of cost management. When changes are requested, it often involves design changes which must be implemented as soon as possible, and thus the change management process delays the work. Therefore the cost management has to be fast and effective so that works are not delayed in the process.

A general finding was that for some participants, cost management merely revolved around ensuring that the budget is not overspent and that there is enough money in the contingency budget for the works. They also regard cost management as the assurance that the rate is reasonable according to the engineer’s judgement. The overall effect of the works on the project, its indirect costs (such as maintenance), and time-related preliminary and general costs are rarely considered.

Time constraints were found to be the biggest obstacle for not applying standard risk management principles in practice. The two other major obstacles were knowledge and the practicality of risk management. When asked about the biggest stumbling block for performing risk management, an interviewee replied: “Time, and the lack of understanding..."
in the project team that there is value in doing risk management. There is never time to do it, but always time to do it twice.”

SUMMARY OF FINDINGS
This article investigated the impact that changes can have on a project, as well as the current state of change management of construction projects in practice. This was done by conducting a case study and various interviews with project managers. The key findings of the case study and various interviews with project managers, regarding the impact of changes on the project, can be summarised as follows:

- Change can have a significant cost and time impact on a construction project.
- Changes are mainly recorded based on the ISO 9000 certified QMS and a paper-based exercise.
- Changes are mainly recorded based on the ISO 9000 certified QMS and a paper-based exercise.
- Project managers make extensive use of their experience and engineering judgement for managing the cost of changes.
- Other methods of cost management include:
  - Doing a cost estimate of the proposed works.
  - Requesting rates from the contractor and assessing their reasonableness.
  - Determining the effect of the work on the project budget and contingencies.
  - Determining the indirect cost implications, such as life cycle or time-related costs.
- More than 50% of participants do not follow a formal change management process.
- Time, knowledge and practicality are hindering the application of standard risk management principles.
- Most companies seem to have a QMS in place, but the practical application of that process is not clear. Changes to projects are captured through a paper-based exercise of SMs, QRs, VOs and MPCs. Neither the interviews, nor the findings of the case study indicated any current use of a formal change impact review methodology.
- Most project managers do not follow the theoretical risk management process of planning, identification, analysis and mitigation. Time constraints were found to be the biggest reason why project managers do not apply generally accepted cost and risk management practices. They therefore follow no specific procedure and rely on their crisis management abilities.

CONCLUSIONS
The aim of the article was to determine the impact of changes made to the works during the construction phase of a civil construction project, and three main conclusions were made:
1. Changes can have a significant cost and time impact on a construction project, and changes have a greater impact than rework.
2. The current method of change management is not adequate, as not all changes are recorded in the appropriate manner, and risk management is seldom done.
3. Time, knowledge and practicality are factors hindering the application of standard risk cost and risk management principles. The results are disturbing, given the magnitude of the projects discussed in this study, and the possible implications of changes. The results expose project managers’ and engineers’ lack of critical skills and competencies, aspects which are prerequisites for the successful execution of a project in terms of time, quality and cost. It is therefore recommended that project managers and engineers are carefully selected, not only based on experience, but also based on the necessary skills and competencies.

REFERENCES
Wind loading on catenary vault structures

R A Bradley, M Gohnert, A M Goliger, Y Mistry

Catenary vault structures are increasingly being utilised as an efficient alternative to the traditional circular vault. However, little is known regarding wind loading effects, in particular the pressure distribution, over these forms. Consequently, applicability of the data given for circular vaults used for the design of catenary vaults is also uncertain. In this paper the results of a wind-tunnel investigation and their implications on the design of catenary vaults are discussed. A series of tests were undertaken to assess the effects of variation in vault geometry and wind orientation on surface pressures measured over these structures. These parameters are evaluated to clarify their influence on the loading on catenary vaults. Only the results relating to mean and root-mean-square pressure coefficients are reported, in line with wind loading standardisation practice (if localised pressures are relevant, the designer is referred to expert advice or an adoption of the data from similar geometrical forms). The primary focus of the investigation is to examine the applicability of the design data given in SANS 10160-3:2011 (SANS 2011b) (referring to circular and duo-pitched roofs) to catenary structures. Subsequently, the merit of developing a set of exclusive design coefficients relating to catenary vaults is considered.

INTRODUCTION

The fundamental advantage of catenary shells is that they contain compressive stress only (i.e. no bending, shear or tensile forces) under the action of self-weight alone. This benefit has endorsed their use for medium-sized unreinforced masonry shell structures (i.e. housing and recreational buildings). Furthermore, these structures can be constructed reasonably cheaper than typical masonry and reinforced concrete structures of a similar nature (Bulovic 2014). However, whilst masonry is strong in compression, it is very weak in tension and flexure. The flexural strength is also influenced significantly by the level of adhesion that is achieved between mortar and bricks on site. Consequently these shells have an affinity to crack when exposed to relatively small tensile forces.

These aforementioned characteristics draw attention to the importance of non-uniform loading that may induce tensile stress in these structures. Thus, to ensure safety and economy, the designer must carefully consider all possible loading that may produce tensile stress, namely wind loading, thermally induced stresses, live loading, settlement and dynamic loading (in areas prone to seismic activity). Furthermore, lateral loads are severe, compared to vertical loads, particularly as the vaults become ‘steeper’. In South Africa, in the absence of severe seismic activity, wind loading is typically the most significant lateral load for low-rise buildings.

It is commonly acknowledged that shell structures offer significant benefits with regard to aerodynamic efficiency, as discovered by researchers investigating wind loads over cylinders and curved roof structures. Typically, the commonly utilised shapes (e.g. circular) are included in codes of practice for the design of silos, single-vault structures and domes. At low height-to-base radius ratios, the catenary and circular profiles become practically identical. However, the scale (and form) of the structures examined in this study necessitates the use of ‘steeper’ profiles (to allow for reasonable amounts of usable space).

The aims of the investigations described in this paper are as follows:
1. To establish the critical importance of wind loading for unreinforced masonry catenary vaults.
2. To establish the applicability of the design pressure coefficients for circular and duo-pitch roofs stipulated in SANS 10160-3:2011 (SANS 2011b) to catenary roofs. (Therefore, in principle, the pressure coefficients that are considered in the current paper correspond to Cpe10, i.e. the largest overall load areas of 10 m², as specified in SANS 10160-3:2011 (SANS 2011b)).
3. If necessary, to develop specific external pressure coefficient data for catenary vault structures.

In order to achieve these objectives, a series of wind-tunnel tests were undertaken at the boundary-layer wind-tunnel (BLWT) of the CSIR, Pretoria, South Africa. Five vault roof forms were tested in a turbulent...
boundary layer at multiple wind orientations. Only the mean and root-mean-square (rms) pressure coefficients are considered in this paper.

LITERATURE

Generally, the work published pertaining to wind loading on curved surfaces refers to circular vaults and domes, and no research was found in relation to catenary structures. There is, however, a wealth of information on aspects relating to similar bluff body structures and wind-tunnel testing procedures, which is applicable to the current investigation. Several papers, including Paluch et al. (2003), Blessmann (1996), Taylor (1992), and Cheung and Melbourne (1983), discuss experimental investigations on single vaults, domes and curved roof structures. Of particular interest are the contributions of Blackmore and Tsokri (2006), and Toy and Tahouri (1988) (with regard to circular vaults) from which selected data is evaluated against the current results.

Blackmore and Tsokri (2006) conducted a series of wind-tunnel tests on circular vaulted roofs with different length-to-diameter ratios (L/d), height-to-diameter ratios (H/d) and with different sidewall heights (h). From these tests the authors noted that the EN 1991-1-4 (EN 2004) values for Zone B (i.e. over the central zone) appear to be most appropriate for long, vaulted buildings (i.e. L/d > 10) with two-dimensional flow rather than shorter buildings that generate three-dimensional flow.

Toy and Tahouri (1988) conducted tests on two- and three-dimensional semi-cylindrical models with different cross-sectional geometries and length/height (L/H) ratios in a turbulent boundary layer. Of particular interest are the mean surface pressure distributions over a pointed circular vault, which is somewhat similar in shape to a steep catenary vault, tested in three-dimensional flow.

From a structural point of view, the catenary is a highly efficient one-way spanning shell structure, but apart from a study on catenary cross-vaults (Bradley et al. 2011), no other loading information is available.

WIND-TUNNEL MEASUREMENTS

Wind-tunnel facility

The boundary-layer wind-tunnel used in this study is an open-circuit type, with a centrifugal fan, powered by a 75 kW thyristor-controlled motor. The cross-section is 1 m high x 2 m wide, with an 18 m long boundary layer development length. An overview of this facility and its capabilities are presented in Milford et al. (1988).

Boundary-layer modelling

A standard ‘open terrain’ boundary-layer profile, as specified in the literature (Davenport et al. 2000) was achieved using a honeycomb mesh screen at the entrance of the tunnel (to introduce the initial small-scale turbulence), four 250 mm high and 100 mm wide triangular spires, followed by a 50 mm high tripping wall (to achieve an initial speed deficit in the lower section of the profile). The boundary-layer was developed by a 16 m length of corrugated cardboard with a uniform roughness height of approximately 2.5 mm. Figures 1(a) and 1(b) show the mean wind speed and turbulence intensity profiles together with the Davenport’s reference data. The longitudinal turbulence spectrum, corresponding to a 50 mm elevation above the wind-tunnel floor, is shown in Figure 1(c).

It can be seen that the mean wind speed profile achieved in the wind-tunnel compares well with the reference data, but the turbulence characteristics do not match exactly with those stipulated for open terrain. This mismatch constitutes a typical limitation of boundary-layer modelling, and larger differences are also reported in full-scale experimentation (Goliger & Milford 1985).

Pressure and velocity measurements

Point pressures were measured for wind directions between 0° and 360°, with an interval of 15°. These measurements were obtained using a Synchronous Multi-Pressure Scanning System (SM-PSS) (based on RAD3200 manufactured by the Scanivalve Corporation). The scanner type is the ZOC23B/8px, which contains eight individual piezo-electric pressure gauges. Two ZOC23B/8px units, capable of instantaneous measurements at 16 pressure taps, were used in this investigation. The pressure scanners, stainless steel tubing connectors (mounted to the circular model) and flexible plastic tubing are shown in Figure 2.
Pressure taps were connected to the system using tubing of 1.0 mm internal diameter. The reference static pressure was obtained from the wind-tunnel roof, above the model, and all pressure coefficients were normalised by the free-stream dynamic pressure at apex height of the relevant vault model. A sampling frequency of 625 Hz was used in this study. The mean and root-mean-square (rms) pressure coefficients were calculated directly from the recorded time series.

Testing was conducted at a wind speed of approximately 11.5 m/s, measured by a Pitot tube located at 750 mm above the wind-tunnel floor. Additional wind speed measurements were taken using a hot-wire anemometer system. The blockage introduced by the model was less than 1.0% of the tunnel cross-section, which is well below the generally accepted limit of 5% (beyond which blockage effects need be considered) noted in the literature (e.g. Stathopoulos & Baniotopoulos 2007). Therefore, blockage effects were considered insignificant in these investigations.

Models
Four catenary vault models (and a single semi-circular model), at a scale of 1 in 500, with different ratios of height-to-base width, were manufactured using a three-dimensional printer. The models were initially drawn using Autodesk's AutoCAD software and then printed at the CSIR. The printer utilises colour inkjet technology in which successive layers of material are printed to create a three-dimensional shape. This method offered high precision (e.g. surface finish) and significantly improved the speed of construction – all models were printed simultaneously (the CAD arrangement is shown in Figure 3(a)). A comparison of the catenary and semi-circular models (with attached sandpaper), corresponding to H/R = 1.0, are shown in Figure 3(b). The catenary model profiles are illustrated in Figure 3(c), and all relevant geometric parameters are given in Table 1.

In all models, only a quarter of the roof was instrumented with twelve pressure taps, as shown in plan and elevation in Figure 4. A single pressure tap was also located at the centre of a sidewall for each model. Due to the geometric symmetry of the models and the application of a range of wind directions from 0° to 360°, effectively 35 roof pressure locations, distributed over the entire roof, were investigated. In this paper, the wind direction is denoted by $\alpha$, whereas the

<table>
<thead>
<tr>
<th>Table 1 Model dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shape</td>
</tr>
<tr>
<td>Catenary</td>
</tr>
<tr>
<td>Catenary</td>
</tr>
<tr>
<td>Catenary</td>
</tr>
<tr>
<td>Catenary</td>
</tr>
<tr>
<td>Semi-circular</td>
</tr>
</tbody>
</table>

Note: All models have width (D) = 120 mm, length (L) = 160 mm, and footprint aspect ratio (L/D) = 1.33

Figure 2 ZOC23B/8px scanners attached to the circular model

Figure 3 Profiles for catenary and circular vaults

(a) CAD Setup  
(b) H/R = 1.0 models  
(c) Catenary profiles
geometric inclination angle is represented by \( \theta \), as illustrated in Figure 4.

**Reynolds number modelling**

For all structures, flow separation points determine the distribution of external pressures. In the case of elements with sharp edges, this separation takes place (and is controlled) by these edges. However, this is not the case for round surfaces, where the pressure distribution is dependent on the viscous forces in the boundary layer. These are characterised by Reynolds's number, which defines the relationship between the characteristic wind speed \( V \), size of the structure \( D \) and air viscosity \( \nu \), as follows: \( R_e = \frac{VD}{\nu} \). The Reynolds number corresponding to a typical vault, with a size comparable to a house, is about \( 10^7 \), and in the model scale about \( 10^5 \). These correspond

![Figure 4 Pressure tap locations](image)

**Figure 4 Pressure tap locations**

![Figure 5 Mean pressure coefficients at selected locations](image)

**Figure 5 Mean pressure coefficients at selected locations**
to the supercritical and subcritical Reynolds number regimes respectively, and the loading implications are reflected in SANS 10160-3:2011 (SANS 2011b). In order to achieve similitude between model and full-scale flow regime, roughening of the model surface was adopted. The required size of the grain \( k \) was determined on the basis of the Engineering Science Data Unit-80025 (ESDU 1980) procedure. Sandpaper (P100) was attached to the model surface, giving grain size to model height \( (k/H) \) ratios between \( 2.5 \times 10^{-3} \) to \( 5 \times 10^{-3} \).

### RESULTS AND DISCUSSION

#### Effect of wind orientation \( (\alpha) \)

Initially, the variation of surface pressure coefficient (at selected positions) with wind direction is discussed. In general, similar trends were observed for all models investigated. However, significant variation is evident at certain tap locations and wind orientations. The mean pressure coefficient plots for four selected tap locations, namely base edge (tap 1), base middle (tap 9), apex edge (tap 4) and apex middle (tap 12) are shown in Figures 5(a–d).

In Figures 5(a) and 5(b) it is evident that the pressure coefficients corresponding to the base taps (1 and 9) show a relationship in which they initially experience positive pressures until the presence of the sharp edge begins to control the location of flow separation. At wind directions \( (\alpha) \) between 0° and 60° pressures are positive, and the switch from positive to suction pressures roughly occurs between 60° and 70° for all models. It can also be seen in Figures 5(a) and 5(c) that the highest negative pressure coefficients for the ‘edge’ taps (1 and 4) occur between 75° to 90°. The reason for these larger suctions is the close proximity of these taps to the position of flow separation at these wind orientations.

Considering tap 12 (Figure 5(d)), a substantial shift in the magnitude of pressure coefficients is apparent at a wind orientation between 15° and 45° for all models. In fact, for the orientation of about 30°, the highest magnitude suctions in the entire study were measured. Furthermore, these figures indicate a trend in which shallower catenary vaults generate higher negative pressures. The importance of three-dimensional flow for structures with a low footprint aspect ratio (i.e. \( L/d \)) is also highlighted in Figure 5.

Figures 6(a–g) and Figures 7(a–g) present contour lines of mean and rms pressures respectively. (These plots were produced using MATLAB® subroutines, with the

---

**Figure 6** Mean pressure coefficient contour plots \( (H/R = 1.0) \)
adoption of cubic interpolation to approximate contour lines. The plots are presented in terms of length (x-axis) and inclination angle $\theta$ (y-axis). Note that these figures do not depict the actual footprint of the roof, but are distorted geometrically for the sake of brevity. These graphs, for H/R = 1.0 only, confirm the observation made above in which the most negative pressures (i.e. dark blue) correspond to $\alpha = 30^\circ$. Furthermore, the largest positive pressures (i.e. intense red) correspond to the 0° orientation for all models. Both negative and positive pressures are relevant, as they can lead to critical loading situations and are thus the focus of further analysis later in this paper.

The distribution of mean pressure coefficients measured at mid-height of the gable wall is shown in Figure 8. As expected, the maximum positive pressure, $C_{pe} > +0.8$, occurs for wind blowing onto the wall (90° orientation), and its magnitude is similar to that which could be derived for vertical walls of rectangular plan buildings (SANS 10160-3:2011 (SANS 2011b)). For a wind orientation of 0°/180° (flow parallel to the gable walls), the negative mean pressure coefficient is $C_{pe} \approx -0.5$. For a wind orientation of 270° (pressure tap located at the leeside of the structure), the negative mean pressure coefficient is $C_{pe} \approx -0.2$. Both of these are visibly smaller than the respective values that could be established from the loading code. Intuitively such relationships seem to be correct, due to the lack of a vertical edge (i.e. walls) along which separation of the flow takes place. In the case of the curved edge, part of the air volume is directed upwards, thus reducing the volume of the flow subjected to flow separation.

**Effect of vault geometry (H/R)**

**Wind parallel to curvature ($\alpha = 0^\circ$)**

Mean and root-mean-square pressures measured along the centreline, for the four category vault models, are compared in Figures 9(a) and 9(b) respectively. The results of these tests reveal that there is a substantial mean pressure coefficient difference between all vaults on the windward side. The contour plots in Figure 10 – in which warm colours indicate positive pressure and cooler colours indicate suction – further illustrate this point. It is apparent in Figures 10(a) through 10(d) that, by increasing the H/R ratio, and effectively the average slope of the roof, the point at which positive pressure transitions to negative pressure (i.e. suction) moves upward along the windward face. This shift (when H/R is increased) generates a larger area subjected to positive pressures on the
Figure 8 Mean pressure coefficients: mid-height side wall

Figure 9(a) Mean centreline pressure coefficient distribution ($\alpha = 0^\circ$)

Figure 9(b) Root-mean-square centreline pressure coefficient distribution ($\alpha = 0^\circ$)
Figure 10 Mean pressure contour plots (α = 0°)

Figure 11 Design pressure coefficients (α = 0°)
windward surface, which has important
design implications that are discussed later
in this paper.

The minimum (i.e. most negative) mean
pressures occur near the apex of all four
models. Furthermore, by comparing the con-
tour plots (see Figure 10(a–d)) it is evident
that the extent of these minimum pressures
(shown in dark blue) decreases with an
increase in height to radius ratio of the
vaults. It is also observed that the pressures
on the leeward side of the H/R = 1.6 catenary
vault are near constant, from the roof apex,
i.e. $\theta = 90^\circ$. However, the distribution for
the H/R = 0.8 model reveals that suctions
become fairly constant at an elevation angle
of roughly 120°, which is similar to that
noted for the semi-circular vault. For the
most part, only negligible variation between
all four models is evident over the leeward
half for this wind orientation.

The variation of pressure coefficients with
H/R ratio, for $\alpha = 0^\circ$, are given in Figure 11.
An attempt was made to interpret these pres-
sures in a similar manner as those given in
SANS 10160-3:2011 (SANS 2011b) for circular
vaults. However, due to the nature of the
wind pressure distributions over the ‘steeper’
catenary vaults, it was considered necessary
to subdivide the central portion of the roof
(Zone B – in SANS 10160). Consequently,
four loading zones (A–D) are stipulated. The
distribution/extent of these zones is illustrated
in Figure 11. The magnitude of the pressure
corresponding to each zone was determined
through numerical averaging of the pressure
distributions over the respective area. Each
zone was divided into six equal segments
and the ‘actual’ distribution (derived through
the ‘streeter’
cubic interpolation) was approximated and
replaced by uniform values. The total coef-
ficient for each zone was thus approximated
as follows:

$$C_{p,\text{Zone } i} = \frac{\sum_{j=1}^{6} C_{p,\text{Z}}}{6} \quad (1)$$

Oblique wind orientations ($\alpha = 30^\circ; 45^\circ$)
As discussed in the previous section, the
largest negative pressures develop for wind
orientations (a) between 30° and/or 45° for
all catenary vaults, and the respective pres-
sure coefficient design data is considered
in this section. The data corresponding to
most negative suctions at both orientations,
and for each catenary profile, is plotted in
Figure 12 (i.e. for aspect ratios H/R = 0.8,
1.0 and 1.2 graphs correspond to $\alpha = 30^\circ$, whereas for $H/R = 1.6$ and $\alpha = 45^\circ$). It also reconfirms the previous observations that the most negative mean pressures correspond to the shallowest catenary vault. It can also be observed that at lower elevations, over the leeside of all vault types (for $\theta > 140^\circ$), the magnitude of pressure coefficients is similar ($-0.5 > C_{pe} > -0.6$).

Figure 13 presents the design plot, derived from numerical averaging, in a similar manner to that discussed in reference to Figure 11.

Wind blowing onto a gable wall ($\alpha = 90^\circ$)

For wind normal to the eaves of a circular roof there is no information given in EN 1991-1-4 (EN 2004) (or in SANS 10160-3:2011 (SANS 2011b)). However, Blackmore and Tsokri (2006) conducted tests on curved roofs and compared their results with EN 1991-1-4 (EN 2004) regarding duo-pitch roof data. Their findings generally revealed satisfactory agreement for design purposes. In the current study, a similar approach was investigated and proposed (with regard to the extent of loading zones and magnitude of the design pressures).

The distribution of mean pressure coefficients measured along the apex of the catenary vaults is shown in Figure 14. Flow separation for this wind orientation is controlled by the wall roof interface; thus variation observed between the roofs is likely due to differences in edge geometry. The corresponding distribution of design coefficients for this orientation is given in Figure 15. Along the edge, the largest suctions correspond to the lowest vault. However, suctions over zones D, C and E (as shown in Figure 15) are largest over the ‘steepest’ vault, and gradually reduce with height. (A similar trend is observed in data due to Blackmore and Tsokri (2006) for circular vaults.)

**Comparisons**

Catenary vs circular (current data)
The distributions of mean and rms centreline pressure coefficients corresponding to the catenary and semi-circular vault models (with the same height and base width, i.e. $H/R = 1.0$) are compared in Figures 16(a) and 16(b). The mean pressure coefficients indicate a similar trend, with considerable difference in the magnitude (of about 40% over the apex). The overall form compares
Figure 16(a) Mean centreline pressure coefficient distribution ($\alpha = 0^\circ$)

Figure 16(b) Root-mean-square centreline pressure coefficient distribution ($\alpha = 0^\circ$)

Figure 17 Mean pressure contour plots ($\alpha = 0^\circ$)
well with the standard textbook distribution for a circular element. Furthermore, the rms pressure coefficients are largely similar.

The contour lines of the mean pressure coefficients (for $\alpha = 0^\circ$) corresponding to both geometries are also shown in Figures 17(a) and 17(b). It can be noted that the semi-circular vault generates a larger magnitude of negative pressures, and the extent of the negative pressure zone is also larger. This disparity is likely to be caused by a ‘steeper’ (over most of the roof) slope corresponding to the catenary shape. In Figure 18, a comparison of both geometries (in terms of an ‘inclination’ angle $\beta$) is made. The line corresponding to the semi-circle is straight (i.e. constant angle), whereas the line of the catenary is curved and either above or below the 45° line. It can be seen that the semi-circle is ‘steeper’ up to about $\theta = 30^\circ$, after which the catenary becomes ‘steeper’ until the apex (i.e. $\theta = 90^\circ$).

Blackmore and Tsokri (2006)

Figure 19 evaluates the current results (obtained from numerical averaging for $H/R = 1.0$) with the corresponding data from SANS 10160-3:2011 (SANS 2011b) (which follows EN 1991-1-4 (EN 2004)), as well as the experimental data due to Blackmore and Tsokri (2006). It is evident that the current experimental data, as well as that due to Blackmore and Tsokri (2006), for the semi-circular vault differs from that prescribed by SANS 10160-3:2011 (SANS 2011b), particularly over the central portion of the roof (i.e. zone B in Figure 19). The current data does, however, generally match that due to Blackmore and Tsokri (2006) for a vault with a length-to-diameter ratio of 1.0 (i.e. small footprint aspect ratio). The vaults

![Figure 18 Variation of gradient ($\beta$) for the catenary and circular geometries](image)

![Figure 19 Pressure coefficients ($\alpha = 0^\circ$)](image)

![Figure 20 Two semi-cylindrical vaults investigated by Toy and Tahouri (1988)](image)
considered in the current investigation are relatively short, with $L/D = 1.3$. Furthermore, the pressure distributions shown in Figure 17 emphasise the three-dimensional nature of the flow. The difference observed in Figure 19 (over zone B) can be attributed to the disparity between two- and three-dimensional flow regimes over circular vaults.

Toy and Tahouri (1988) investigated the influence of the model aspect ratio (in the horizontal plane) reflecting the effects of two- versus three-dimensionality of the flow. (Three situations were considered, namely $L/H = 2$, $L/H = 4$ and a two-dimensional situation, i.e. a model across the entire width of the test section.) Furthermore, differences in cross-sectional geometry of the vaults were also tested. Figure 20 presents only two of these semi-circular cross-sectional geometries.

Figure 21 compares three semi-circular distributions (at different aspect ratios) and the current data. Data corresponding to Toy and Tahouri (1988) (for ratio $L/H = 4$) compares well with the present data. A similar observation can be made to that noted by Blackmore and Tsokri (2006) regarding the significant effect of two- versus three-dimensionality of the flow.

Interestingly, the distributions show some similarity. However, the sharp apex controls flow separation for the pointed cylinder, which is not the case for the catenary vault. Also of interest to note is that Toy and Tahouri (1988) observed the largest negative pressures over the pointed cylinder at a wind orientation of $\alpha = 45^\circ$.

**APPLICATION / DESIGN CONSIDERATIONS**

This section describes a series of finite element (FE) models which were implemented using the data developed in the current study, as well as the corresponding information derived from SANS 10160-3:2011 (SANS 2011b), as loading inputs. This was done to establish the applicability of the standardised loading data to the design of catenary vaults. Furthermore, the importance of wind...
loading on catenary vaults is investigated and discussed.

**Finite element models**

For ease of calculation, the catenary vaults were modelled as a one-metre wide arch/strip. This simple analysis is considered adequate, due to the nature of the loading, support conditions and the need for a critical expansion joint between the vault and gable walls (Bradley & Gohnert 2016). Furthermore, translational restraint only is assumed for the FE models considered in the present paper.

<table>
<thead>
<tr>
<th>Table 2 Masonry properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brick type</td>
</tr>
<tr>
<td>Standard clay brick</td>
</tr>
</tbody>
</table>

$f_{sb}$ – flexural strength parallel to bed joint, $f_{sp}$ – flexural strength perpendicular to bed joint

<table>
<thead>
<tr>
<th>Table 3 Geometric properties of full-scale catenary vaults</th>
</tr>
</thead>
<tbody>
<tr>
<td>H/R</td>
</tr>
<tr>
<td>0.8</td>
</tr>
<tr>
<td>1.0</td>
</tr>
<tr>
<td>1.2</td>
</tr>
<tr>
<td>1.6</td>
</tr>
</tbody>
</table>

All structures span 6 m and have a length of 8 m

The material and geometric properties of the masonry vaults adopted in the present study are given in Tables 2 and 3 respectively. To be noted, although simple two-dimensional FE models are assumed, all stress and bending analyses reflect vaults having plan areas of 48 m² (i.e. 6 m wide and 8 m long). However, the height and vault thickness are adjusted to accommodate the various comparisons considered in this paper. Although masonry is anisotropic, with variable stiffness and strength, material isotropy was assumed due to the one-way span and load distribution. Consequently, a two-dimensional analysis using bar elements, to create the curved catenary shape, was employed in the current investigation. The surface pressures were applied to the models as a series of horizontal and vertical point loads.

Three load cases associated with the two critical wind orientations (0° and 30°/45°) are summarised in Table 4. For these load cases, the internal pressure coefficients ($C_{pi}$) of –0.3, 0 and +0.2 were adopted. In principle, it has to be considered that the full-scale gable walls will accommodate openings (i.e. doors and windows). This, in turn, implies a risk of generating the situation of dominant openings, i.e. higher magnitude of positive internal pressures ($+0.7$), corresponding to the wind blowing onto these walls ($\alpha = 90°$). However, this situation results in a symmetrical load distribution, which is not critical for these catenary vaults. The wind pressures acting over the structures were determined using the coefficients derived in Figures 11 and 13 in combination with the stipulations of SANS 10160-3:2011 (SANS 2011b). The structures were assumed to be located in Johannesburg. The design and site parameters are summarised in Table 5.

**Finite element results**

The bending moment diagrams for the 0° and 30° wind orientations (corresponding to load variation 2 in Table 4) are shown in Figures 23 and 24 respectively. As expected, the bending moments are largest for the 'steepest' vault (i.e. H/R = 1.6) and lowest for the shallowest vault (i.e. H/R = 0.8).
Table 4 Load cases

<table>
<thead>
<tr>
<th>Load case</th>
<th>Variation 1</th>
<th>Variation 2</th>
<th>Variation 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.9 DL + 1.3 WL (α = 0°)</td>
<td>$C_{pi} = 0.0$</td>
<td>$C_{pi} = -0.3$</td>
<td>$C_{pi} = 0.2$</td>
</tr>
<tr>
<td>0.9 DL + 1.3 WL (α = 30°/45°)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DL = dead load, WL = wind load</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 5 Design and site parameters – SANS 10160-3:2011 (SANS 2011b)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fundamental basic wind speed ($v_{0,b}$)</td>
<td>28 m/s</td>
</tr>
<tr>
<td>Probability factor ($C_{prob}$)</td>
<td>1</td>
</tr>
<tr>
<td>Topography factor ($c_{o}$)</td>
<td>1</td>
</tr>
<tr>
<td>Peak wind speed ($v_{b,peak}$)</td>
<td>39.2 m/s</td>
</tr>
<tr>
<td>Air density (at 1 700 m)</td>
<td>0.98 kg/m³</td>
</tr>
<tr>
<td>Terrain category</td>
<td>B</td>
</tr>
</tbody>
</table>

Table 6 Pressure coefficient data for circular roofs derived from SANS 10160-3:2011 (SANS 2011b)

<table>
<thead>
<tr>
<th>Zone</th>
<th>H/R = 0.8</th>
<th>H/R = 1.0</th>
<th>H/R = 1.2</th>
<th>H/R = 1.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>+0.65</td>
<td>+0.80</td>
<td>+0.80</td>
<td>+0.80</td>
</tr>
<tr>
<td>B</td>
<td>-1.10</td>
<td>-1.20</td>
<td>-1.20</td>
<td>-1.20</td>
</tr>
<tr>
<td>C</td>
<td>-1.10</td>
<td>-1.20</td>
<td>-1.20</td>
<td>-1.20</td>
</tr>
<tr>
<td>D</td>
<td>-0.40</td>
<td>-0.40</td>
<td>-0.40</td>
<td>-0.40</td>
</tr>
</tbody>
</table>

The load cases considered in this paper incorporate dead, live and wind loading in combinations illustrated in Figure 25. Imposed loads are derived from SANS 10160-2:2011 (SANS 2011a) for inaccessible roofs (i.e. Category H1 and H2). The surface loading ($q_k$), for a plan area greater than 15 m², of 0.25 kN/m² is adopted. A value of $q_k = 0.5$ kN/m² is also considered to shed light on the sensitivity of catenary vaults to this type of loading. The most severe case for this distributed vertical load corresponds to its action over one half of the structure only (specifically the windward half when wind and live loads are considered in combination).

Figure 25 shows the relationship between aspect ratio H/R and the maximum bending moments, for all load combinations, corresponding to the vault size adopted in the current study. (The pressure distribution corresponding to H/R = 2.0 used in this comparison was obtained through an extrapolation of the data given in Figure 11). It is evident that the bending moments obtained for the vertically imposed loads do not vary visibly with changes in aspect ratio. However, an exponential relationship is observed for the wind load combinations. In fact, considering the H/R = 1.6 vault, the bending moment derived for the wind load case is an order of magnitude greater than that for the imposed vertical live loads. This result clearly identifies the significance of lateral/wind loads over these catenary shells – particularly for the profiles with high aspect ratios (i.e. H/R).

This observation is attributed to the critical nature of the pressure distribution over vaults with high aspect ratios (H/R). In particular, this is due to the development of larger positive pressure (applied over a larger area) on the windward face, in combination with negative pressure over the leeward half. However, it is also partially attributed to the large difference in total surface area between the various vaults (the total surface area for the ‘steepest’ vault is roughly 1.5 times larger than the ‘shallower’).

Interestingly, for H/R between 0.8 and 1.2 the maximum bending moments correspond to the load case with wind blowing at 30°. However, for the ‘steepest’ vault (i.e. H/R = 1.6), maximum bending is observed for α = 0°.

Several load scenarios were considered to emphasise the importance of the correct consideration of wind loading for the design of unreinforced masonry catenary shells. The design of these shells typically incorporates the development of a pure compression form under self-weight alone, after which asymmetric load combinations are applied and the shell thickness is adjusted until no tension exists within the structure.
Comparisons of experimental data and SANS 10160-3:2011

The above observations, in combination with the form of the pressure coefficient distributions shown in Figures 9(a) and 12, prompted a comparison of the experimental data with the design coefficients stipulated for traditional structural forms (i.e. circular and duo-pitch roofs) in SANS 10160-3:2011 (SANS 2011b).

Circular roofs

Pressure coefficients, from SANS 10160-3:2011 (SANS 2011b), corresponding to circular vaults are given in Table 6. Figure 26 shows a comparison between the experimental pressure coefficients for the catenary vault, with $H/R = 0.8$ and the above-mentioned data corresponding to a circular vault with $f/d = 0.4$. Both distributions are represented by four zones (i.e. A, B, C and D), as discussed previously. A good degree of similarity is observed between these pressure coefficients. Figure 27 illustrates the bending moment diagrams obtained by applying both distributions to the catenary vault with $H/R = 0.8$. This figure demonstrates the general applicability of the codified data for circular roofs to shallow catenary vaults.

Duo-pitch roofs

Divergence of the bending moment diagrams occurs when the semi-circular vault design coefficients (from the loading code) are evaluated against the experimental data for the ‘steepest’ vault. Consequently, the applicability of data for duo-pitch roofs is investigated as an alternative. The relevant pressure coefficients, derived from SANS 10160-3:2011 (SANS 2011b), and the distribution of related zones F, G, H, J and I, are given in Table 7 and Figure 28 respectively. It should be noted that the suctions in zones F, G and J over duo-pitch roofs reflect the risk of vortices generated over the corners (i.e. wing-tip vortices), and flow separation at the eaves and ridge (this, however, is not the case for the curved roofs investigated in this study, which do not contain raised corners, a sharp ridge or eaves). Consequently, the negative pressures stipulated for zones F and G in SANS 10160-3:2011 (SANS 2011b) are disregarded, and only positive pressures are considered over the windward areas. However, the suction over zone J is adopted.

The average ‘pitch’ angle of the curved roofs is determined by inscribing straight lines from its base to apex, i.e. $\psi = \tan^{-1}(H/R)$ – see illustration in Table 8. The parameter ‘e’, which defines the extent of zones F, G and J, is also incorporates in Table 8. Each catenary geometry gives rise to a different average roof slope (not necessarily the same as per loading code). Therefore, in order to accommodate a fair comparison of both data sets, linear interpolation

<table>
<thead>
<tr>
<th>Roof pitch (°)</th>
<th>Zone F</th>
<th>Zone G</th>
<th>Zone H</th>
<th>Zone J</th>
<th>Zone I</th>
</tr>
</thead>
<tbody>
<tr>
<td>30°</td>
<td>+0.7</td>
<td>+0.7</td>
<td>+0.4</td>
<td>−0.5</td>
<td>−0.4</td>
</tr>
<tr>
<td>45°</td>
<td>+0.7</td>
<td>+0.7</td>
<td>+0.6</td>
<td>−0.3</td>
<td>−0.2</td>
</tr>
<tr>
<td>60°</td>
<td>+0.7</td>
<td>+0.7</td>
<td>+0.7</td>
<td>−0.3</td>
<td>−0.2</td>
</tr>
</tbody>
</table>

Table 7 Selected duo-pitch design data taken from SANS 10160-3:2011 (SANS 2011b)
between the 30˚, 45˚ and 60˚ roof slopes was implemented. The adopted coefficients are included in Table 9.

In this section only the pressure distributions corresponding to the ‘steepest’ catenary vault are compared. The pressure coefficients for both data sets are grouped into six zones to clearly illustrate the differences (see Figure 29). Some of the zones corresponding to the duo-pitch data have been sub-divided in half (i.e. zones H and I) to accommodate this comparison. It can be seen in Figure 29 that there is good agreement between both distributions for the windward zones 1 and 2. However, significant discrepancy between the coefficients over zone 3 is apparent. Also, the coefficients corresponding to the experimental data are more negative over the leeward surface. Although these coefficients show somewhat large variations on some parts of the roof, they ultimately result in a similar bending moment diagram as shown in Figure 30. If Figures 29 and 30 are considered in combination, the effect of the lower pressure on the windward face (in the experimental data) is countered by larger suctions on the leeward face, leading to a similar net loading effect for both distributions.

### Design recommendations

In order to determine the point at which the circular design coefficients become inadequate, and the duo-pitch data should be adopted, a comparison of the minimum required design vault thicknesses was carried out. An elastic design, adopting ‘the middle third’ technique and pinned end connections, was selected as a simple methodology to facilitate the relevant comparisons. To be noted, graphic statics (and limit analyses) are preferred design methodologies, since elastic analyses require several assumptions about the material, which is anisotropic and imperfectly elastic. Furthermore, elastic methods may lead to overly conservative designs. However, a simplified elastic analysis is reasonable for the comparisons made in the present paper. The concept of ‘the middle third’ is derived directly from bending theory as follows:

\[
\sigma_{C/T} = \frac{-P}{A} \cdot \frac{My}{I} + \frac{P}{bt} \cdot \frac{6Pe}{bt^2} = 0
\]

(2)

\[
\sigma_T = \frac{-P}{bt} + \frac{6Pe}{bt^2} = 0
\]

(3)

Required thickness \[t = 6e\] (4)

The results of the stress design processes are shown in Figure 31. It can be seen that the use of the duo-pitch data typically leads to a

---

**Table 8 Roof-pitch and pressure zone geometry**

<table>
<thead>
<tr>
<th>H/R</th>
<th>h (m)</th>
<th>b (m)</th>
<th>e (m)</th>
<th>e/10 (m)</th>
<th>Roof-pitch (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.8</td>
<td>2.4</td>
<td>8.0</td>
<td>4.8</td>
<td>0.48</td>
<td>≈ 40</td>
</tr>
<tr>
<td>1.0</td>
<td>3.0</td>
<td>8.0</td>
<td>6.0</td>
<td>0.60</td>
<td>45</td>
</tr>
<tr>
<td>1.2</td>
<td>3.6</td>
<td>8.0</td>
<td>7.2</td>
<td>0.72</td>
<td>50</td>
</tr>
<tr>
<td>1.6</td>
<td>4.8</td>
<td>8.0</td>
<td>8.0</td>
<td>0.80</td>
<td>60</td>
</tr>
</tbody>
</table>

\(e = \text{minimum (2h or b)}\)

**Table 9 Relevant duo-pitch pressure coefficients for catenary vaults investigated**

<table>
<thead>
<tr>
<th>H/R</th>
<th>Roof-pitch (°)</th>
<th>Zone F</th>
<th>Zone G</th>
<th>Zone H</th>
<th>Zone J</th>
<th>Zone I</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.8</td>
<td>≈ 40°</td>
<td>+0.70</td>
<td>+0.70</td>
<td>+0.53</td>
<td>−0.37</td>
<td>−0.27</td>
</tr>
<tr>
<td>1.0</td>
<td>45°</td>
<td>+0.70</td>
<td>+0.70</td>
<td>+0.60</td>
<td>−0.30</td>
<td>−0.20</td>
</tr>
<tr>
<td>1.2</td>
<td>≈ 50°</td>
<td>+0.70</td>
<td>+0.70</td>
<td>+0.63</td>
<td>−0.30</td>
<td>−0.20</td>
</tr>
<tr>
<td>1.6</td>
<td>≈ 60°</td>
<td>+0.70</td>
<td>+0.70</td>
<td>+0.70</td>
<td>−0.30</td>
<td>−0.20</td>
</tr>
</tbody>
</table>

---

**Figure 29 Design pressure coefficients (H/R = 1.6)**

**Figure 30 Bending moment diagrams [(0.9Dn + 1.3Wn); H/R = 1.6]**
Conservative design. However, the minimum shell thicknesses, derived from ‘duo-pitch’ and experimental coefficients, converge at an H/R value of approximately 2, and the circular data is less conservative than the duo-pitch data up to an H/R value of roughly 1.2. The significance of these differences is clearly emphasised in the percentage error plot shown in Figure 32. Based on these comparisons it is proposed, for design purposes, that catenary vaults with an H/R ≤ 1.2 could be designed using the coefficients given in SANS 10160-3:2011 (SANS 2011b) for circular vaults, whereas duo-pitch data may be adopted for profiles greater than H/R = 1.2. This finding demonstrates that no specific design data on wind loading of catenary vaults needs to be derived.

**CONCLUSIONS**

A series of wind-tunnel tests were carried out at the CSIR to measure wind pressure coefficients on catenary vault structures. Four catenary vault roof geometries and a semi-circular shape were modelled and tested in a nominal boundary layer corresponding to an open-terrain category. The primary purpose of these measurements was to assess applicability of the design data included in SANS 10160-3:2011 (SANS 2011b), and to establish the need for development of particular data related to catenary vaults.

The following conclusions can be made:

- A set of design wind pressure coefficients, measured in three-dimensional flow conditions, was derived for catenary vaults with aspect ratio (H/R) between 0.8 and 1.6.
- The critical orientation for the shallow (i.e. H/R ≤ 1.2) catenary vaults corresponds to α = 30°, whereas for the highest vault (i.e. H/R = 1.6) this corresponds to α = 0°.
- The distribution of pressure coefficients over the catenary vault with lowest H/R ratio shows the greatest degree of similarity with the circular form, as per SANS 10160-3:2011 (SANS 2011b). For the ‘steepest’ vault, the experimental data exhibits the closest resemblance to the duo-pitch data from the above-mentioned standard.
- It was established that an H/R ratio of 1.2 constitutes ‘the border case’ for the application of both approximations (i.e. for H/R ≤ 1.2) where the ‘circular data’ may be adopted, whereas for H/R > 1.2 the ‘duo-pitch data’ is more adequate.
- When adopting the ‘duo-pitch data’ (for H/R > 1.2), the average roof pitch (ψ) for the catenary profiles can be approximated as ψ = tan⁻¹(H/R).
- Masonry vaults are commonly considered as being of substantial self-weight, of markedly larger magnitude than the wind loading. However, the wind effects on unreinforced catenary shells (particularly those with steep profiles) may govern design, as demonstrated in the current paper.
ACKNOWLEDGEMENTS
The authors gratefully acknowledge the support of the South African National Research Foundation (NRF). Mr Ters van Wyk of the CSIR is thanked for his assistance with the execution of experimental work, and Mr Edward Pretorius of the University of the Witwatersrand for his assistance with model instrumentation and preparation.

REFERENCES
INTRODUCTION

The determination of the particle size distribution of a soil, including the clay content, is one of the most problematic areas in geotechnical engineering testing. The particle size analysis is a method of separating soils into different fractions based on the sizes of particles present in the soil. The particle size analysis is divided into two categories (coarse and fine) with associated laboratory test methods. A sieve analysis is used to separate the coarse-grained fraction of soil, i.e. the fraction of soil with particle sizes greater than 425 microns. On the other hand, sedimentation analysis, which is based on the principles of dispersion and sedimentation, is used for the analysis of the fine-grained fraction of the soil, such as silt and clay, of which the particle size is less than 75 microns. Analysis of these fine-grained soils is done either by the pipette method or by the hydrometer method (Arora 2003).

Since the clay content of a soil is used to determine its activity, which in turn is used for design purposes, it is very important to accurately determine the clay content of soils. Inaccurate clay content determinations have resulted in inappropriate design solutions, which have even led to unacceptable damage to the structures. In South Africa there is a problem with the accurate determination of the clay content of soils. This problem, which was formally expressed by Jacobz and Day (2008), reinforces the need for research into the optimum concentration and volume. Calgon proved to be the most effective in the alluvial soil, increasing the clay content by 38%. The NaPP was most effective in the relatively active black soil, increasing the clay content by 25%.

The effect of type, concentration and volume of dispersing agent on the magnitude of the clay content determined by the hydrometer analysis

A Kaur, G C Fanourakis

Knowledge of the physical properties of soils, including the clay content, is of utmost importance in the field of geotechnical engineering. The hydrometer analysis is the most widely used technique for the analysis of the particle size distribution of the fine-grained fraction of a soil, calculated using sedimentation principles. The hydrometer analysis utilises a dispersing agent – Calgon 33:7 (comprising 33 grams of sodium hexametaphosphate and 7 grams of sodium carbonate when mixed in 1 litre of water) is universally considered as the most effective dispersing agent.

In this investigation, hydrometer analyses were conducted (according to the TMH1 1986 method) on two soils (alluvium and black clay), using five dispersing agents. The results show that the clay size fraction can vary significantly (from 1% to 32%) for the two soils, depending upon the dispersing agent used. From these initial results, the two most effective dispersing agents (Calgon and sodium pyrophosphate decahydrate – NaPP) were investigated further to establish the optimum concentration and volume. Calgon proved to be the most effective in the alluvial soil, increasing the clay content by 38%. The NaPP was most effective in the relatively active black soil, increasing the clay content by 25%.

Keywords: grain-size analysis, hydrometer tests, dispersing agents, concentration, volume.
by the Soils Testing Laboratory of the South African government’s Department of Water Affairs. The justification for the use of this latter dispersing agent (di-sodium di-hydrogen pyrophosphate) could not be established.

Calgon, which is a combination of sodium hexametaphosphate and sodium carbonate, is one of the popular dispersing agents adopted by various countries for the sedimentation test analysis. The BS 1377 Part 2 (BS 1990) and IS 2720 Part IV (IS 1985) methods recommend 33 grams of sodium hexametaphosphate with 7 grams of sodium carbonate, while the (South African) Council for Scientific and Industrial Research (CSIR) recommends 40 grams of sodium hexametaphosphate with 10 grams of sodium carbonate. All the methods mentioned above use 125 ml of solution of the prescribed concentrations, except for ISRIC which uses only 20 ml of solution of the prescribed concentration.

Sridharan et al (1991) described a study on the effect of different types and quantities of dispersing agents on the grain-size distribution, particularly the percentage of clay-sized material. They concluded that the clay-size fraction can vary from 4% – 45% for marine clays, depending on the dispersing agent used, strictly following the IS (1985) method. It was further seen that 100 ml – 125 ml of Calgon (33 grams of sodium hexametaphosphate and 7 grams of sodium carbonate in 1 litre of distilled water) was found to be the most effective dispersing agent.

Bindu and Ramabhadran (2010) conducted a study to evaluate the effect of the concentration of the dispersing agent on the hydrometer analysis, and attempted to optimise the concentration of the dispersing agent to be added to obtain maximum dispersion. It was observed that the addition of sodium carbonate improved the dispersing capacity of sodium hexametaphosphate. The optimum volume and concentration was found to be 100 ml of 6% mixture of sodium hexametaphosphate and sodium carbonate, and there was a significant decrease in dispersion with a further increase in the concentration as well as volume of the dispersing agent added.

The objective of the current study was to investigate the effect of different dispersing agents on two different soil samples with varying mineralogy. An effort was made to compare the results of the hydrometer test analyses by varying the concentration and volume of the best two dispersing agents on the two soil samples, following the THM1 (1986) test method.

### EXPERIMENTAL DETAILS

#### Materials

Two soil samples were collected from various parts of South Africa. The first sample comprised an alluvial soil from the Sebokeng Township in the Gauteng Province, and the other was a black soil from the town of Brits in the North West Province. The Atterberg limits and activity of these soils were determined at the laboratories of the Department of Civil Engineering Technology of the University of Johannesburg, as shown in Table 1.

The TMH1 (1986) method was used for the determination of the liquid limit and plastic limit. The clay content was determined by means of the hydrometer analysis (Method A6 of TMH1 1986), with a deviation in the prescribed dispersing agent type, quantity and adjustment in the readings by subtracting the hydrometer readings obtained on the ‘blank’ companion specimens to account for the effect of the dispersing agent. TMH1 (1986) Method A6 prescribes 5 ml of sodium silicate and 5 ml of sodium oxalate as the dispersing agent. The activities of the soils used for current study were computed by using the clay content obtained by the hydrometer analysis when 125 ml of Calgon 33/7 (a solution comprising 33 grams of sodium hexametaphosphate (NaHMP) and 7 grams of sodium carbonate (Na2CO3) in 1 litre of distilled water) was used.

#### Hydrometer tests

Hydrometer analyses were conducted on both samples according to the TMH1 (1986) method to determine their clay content. These analyses were carried out in three stages. Stage I comprised two tests, one on each soil type, which excluded dispersing agents. These served as the control test results. Stage II testing entailed a total of ten hydrometer tests, five on each of the two soil types, using the dispersing agents shown in Table 2. Stage III comprised further testing on each of the two soil types, which entailed varying the concentration and volume of two of the dispersing agents (Calgon and sodium pyrophosphate decahydrate). These two dispersing agents were selected for further investigation, as the results of the Stage II testing indicated these to be the most effective of the five dispersing agents. The objective of this Stage III testing was to compute the optimum concentration and volume of the dispersing agent.

Internationally there are at least four methods – BS 1377 Part 2 (BS 1990), IS 2720 Part IV (IS 1985), South African CSIR, International Soil Reference and Information Centre (ISRIC 2002) – which recommend the use of Calgon as a dispersing agent.

### Table 1 Atterberg limits, linear shrinkage and activity of the soils sampled

<table>
<thead>
<tr>
<th>Properties</th>
<th>Alluvial soil</th>
<th>Black soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid limit (LL)</td>
<td>32</td>
<td>56</td>
</tr>
<tr>
<td>Plastic limit (PL)</td>
<td>16</td>
<td>22</td>
</tr>
<tr>
<td>Plasticity index (PI)</td>
<td>16</td>
<td>34</td>
</tr>
<tr>
<td>Clay content (%)</td>
<td>21.4</td>
<td>32</td>
</tr>
<tr>
<td>Activity (A)</td>
<td>0.75</td>
<td>1.07</td>
</tr>
</tbody>
</table>

### Table 2 Details of various dispersing agents used

<table>
<thead>
<tr>
<th>Preparing stock solutions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dispersing agent</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>5 ml of sodium silicate and 5 ml of sodium oxalate</td>
</tr>
<tr>
<td>Sodium silicate: Dissolve sodium silicate, preferably the glass solution (Na2SiO3) in distilled water until the solution yields a reading of 36 at a temperature of 20°C on the standard soil hydrometer.</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>125 ml of Calgon solution</td>
</tr>
<tr>
<td>Sodium oxalate: This consists of a filtered saturated solution of sodium oxalate (Na2C2O4).</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>20 ml of sodium pyrophosphate</td>
</tr>
<tr>
<td>Calgon: Mix 35 grams of sodium hexametaphosphate (Na3P2O7) with 7 grams of sodium carbonate (Na2CO3) and add a sufficient quantity of distilled water to bring the volume of the solution to one litre.</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>20 ml of sodium tetra pyrophosphate</td>
</tr>
<tr>
<td>Sodium pyrophosphate decahydrate: Mix 36 grams of sodium pyrophosphate decahydrate (Na2H2P2O7) with a sufficient quantity of distilled water to bring the volume of the solution to one litre.</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>40 ml of sodium silicate and 40 ml of di-sodium di-hydrogen pyrophosphate</td>
</tr>
<tr>
<td>Di-sodium di-hydrogen pyrophosphate: Mix 36 grams of di-sodium di-hydrogen pyrophosphate (Na3H2P2O7) with a sufficient quantity of distilled water to bring the volume of the solution to one litre.</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>5 ml of sodium silicate</td>
</tr>
<tr>
<td>Sodium silicate: Add sodium silicate syrup (Na2SiO3) to distilled water until the solution yields a reading of 36 at the temperature of 19.5°C on the standard soil hydrometer.</td>
</tr>
</tbody>
</table>
However, all four of them use Calgon in different concentrations. BS 1377 Part 2 1990 (BS 1990) and IS 2720 Part IV (IS 1985) both recommend 125 ml of Calgon 33:7 (33 grams of sodium hexametaphosphate and 7 grams of sodium carbonate mixed with 1 litre of distilled water), CSIR recommends 125 ml of Calgon 35:7, while ISRIC recommends 20 ml of Calgon 40:10.

Amounts of 4%, 4.2%, 5%, 7%, 8% and 9% solution of Calgon, and 3%, 3.6%, 5%, 6% and 7% solution of sodium pyrophosphate decahydrate were prepared by mixing the required quantity in 1 litre of distilled water. The quantities (in grams) of chemicals added for the preparation of these stock solutions are given in Table 3.

In these tests the minimum volumes of Calgon and sodium pyrophosphate decahydrate used were 100 ml and 20 ml respectively. These volumes were incrementally increased until the optimum volume for each concentration was established.

**Testing procedure and calculations**

The testing was in accordance with the procedure described in TMH1 (1986). For all the tests performed, 50 grams of soil sample passing through a 425 micron sieve were mixed with the desired quantity of dispersing agent and about 400 ml of distilled water in a canning jar. The soil-water mixture was allowed to stand overnight. After the mixture had been allowed to stand, it was dispersed for 15 minutes with a standard paddle. The paddle was washed clean with distilled water allowing the wash water to run into a container with the suspension. The suspension was then poured into the Bouyoucos cylinder, and the canning jar was rinsed with distilled water from the wash bottle. The cylinder was then filled with distilled water to the 1 130 ml mark with the Bouyoucos hydrometer (152H) inside. Then the hydrometer was removed and the cylinder was inverted a few times, using the palm of one hand as a stopper over the mouth of the cylinder to ensure that the temperature was uniform throughout. After bringing the cylinder to a vertical position, a stop watch was started. The hydrometer was inserted.

<table>
<thead>
<tr>
<th>Concentration of solution (%)</th>
<th>Quantity of NaHMP added (g)</th>
<th>Quantity of Na₃CO₃ added (g)</th>
<th>Quantity of NaPP added (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>–</td>
<td>–</td>
<td>30</td>
</tr>
<tr>
<td>3.6</td>
<td>–</td>
<td>–</td>
<td>36</td>
</tr>
<tr>
<td>4</td>
<td>33</td>
<td>7</td>
<td>–</td>
</tr>
<tr>
<td>4.2</td>
<td>35</td>
<td>7</td>
<td>–</td>
</tr>
<tr>
<td>5</td>
<td>40</td>
<td>10</td>
<td>50</td>
</tr>
<tr>
<td>6</td>
<td>–</td>
<td>–</td>
<td>60</td>
</tr>
<tr>
<td>7</td>
<td>60</td>
<td>10</td>
<td>70</td>
</tr>
<tr>
<td>8</td>
<td>70</td>
<td>10</td>
<td>–</td>
</tr>
<tr>
<td>9</td>
<td>80</td>
<td>10</td>
<td>–</td>
</tr>
</tbody>
</table>
and readings were taken at 18 and 40 seconds without removing the hydrometer from the cylinder. The hydrometer was then taken out and rinsed with water and it was again inserted into suspension when the elapsed time was two minutes. This reading was noted and the hydrometer was removed and placed in distilled water. This procedure was repeated for readings at 5 minutes, 15 minutes, 30 minutes, 1 hour, 4 hours and 24 hours. After taking each hydrometer reading, the temperature of the liquid was recorded. Temperature corrections were applied to the readings.

A blank solution, comprising distilled water and dispersing agent, was also prepared in a second Bouyoucos cylinder in the same proportions as solutions prepared with the soil. The dispersing agent and water mixture was also soaked overnight and identical hydrometer tests were performed for the blank solutions.

The hydrometer readings taken on the samples which contained soil were appropriately adjusted by subtracting the hydrometer readings obtained on the “blank” companion specimens, at the relevant times. This accounted for the effect of the dispersing agent on the readings. It should be noted that TMH1 does not make any provisions for this correction.

The percentages finer than 0.075 mm, 0.05 mm, 0.04 mm, 0.026 mm, 0.015 mm, 0.01 mm, 0.0074 mm, 0.0036 mm and 0.0015 mm were respectively calculated by

\[
P = \frac{C \times S_f}{S_m}
\]

Where

- \( P \) = percentage finer than relevant size
- \( S_m \) = mass of soil fines used in analysis (50 grams)
- \( S_f \) = percentage soil fines in total sample (< 0.425 mm)
- \( C \) = corrected hydrometer reading

The percentage clay content present in each sample (fraction finer than 0.002 mm) was obtained from the relevant particle size distribution curve. The tests which gave the best dispersing agent, optimum concentration and volume were repeated to check the consistency of the results.

**RESULTS AND DISCUSSION**

**Stage I results**

In this stage, two tests were performed, one on each soil. These were the control tests that excluded dispersing agents. Figure 1 shows the soil suspension after 24 hours of both the black soil and the alluvial soil. In both cases it can be seen that the supernatant water is almost clear of soil grains including colloids.

The particle size distribution curves for these two control tests are shown in Figure 2.

**Stage II results**

The particle size distribution curves obtained for the two soil types, with the use of each of the five dispersing agents used (detailed in Table 2), are shown in Figure 2. From Figure 2 it is evident that the control samples of both the alluvial and black soils yielded near-zero clay contents.

The analyses using the five dispersing agent types indicated that the clay content of the alluvial soil ranged from 7% to 21%, and that of the black soil ranged from 0.1% to 32%. Furthermore, it can be seen that in the case of both soil types, Calgon (combination of sodium hexametaphosphate plus sodium carbonate), which has been recommended by many methods and researchers internationally (BS 1990; IS 1985; ISRIC 2002; Bindu & Ramabhadran 2010; Sridharan et al 1991), yielded the maximum clay content. This was most effective in the case of the black soil.

The second best dispersing agent after Calgon was found to be sodium pyrophosphate decahydrate (NaPP). Calgon yielded 21% and 32.1% clay content, while sodium pyrophosphate decahydrate yielded 20% and 20.5% clay content in alluvial and black soil respectively.

In addition, sodium silicate and sodium oxalate, which is prescribed in TMH1 (1986), was the least effective dispersing agent in the case of the black soil, yielding a clay content of 0%. The least effective dispersing agent in the case of the alluvial soil was sodium silicate and di-sodium di-hydrogen pyrophosphate, which yielded a clay content of 6.2%.
Stage III results

**Calgon**

The effect of different concentrations and volumes of Calgon for the alluvial and black soils is shown in Figures 3 and 4 respectively. The clay contents determined by the different concentration and volume combinations are shown in Table 4. The best and worst results for each concentration are shown in green and red respectively.

It is evident that at the volume of 125 ml (the volume recommended by BS 1377 Part 2 (1990) and IS 2720 Part IV (1985), 5% Calgon yielded the maximum clay content of 25.5% for the alluvial soil (Figure 3 and Table 4), while for black soil the 4.2%, 5% and 7% Calgon all yielded the maximum clay content of 32% (Figure 4 and Table 4).

Figures 3 and 4, and Table 4 indicate the following:

- The 4.2% Calgon proved to be the best dispersing agent yielding maximum clay contents of 29.5% and 35% for the alluvial and black soils respectively. This is in close agreement with the concentration prescribed by the South African CSIR CA 17 method, but in disagreement with Bindu and Ramabhadran (2010) and Sridharan et al. (1991). According to Bindu and Ramabhadran (2010), the optimum concentration of Calgon was found to be 53.7, while Sridharan et al. (1991) found Calgon 33.7 to be the optimum concentration.
- The optimum volumes of Calgon for the alluvial and black soil were 225 ml and 200 ml respectively. This is in disagreement with Bindu and Ramabhadran (2010) and Sridharan et al. (1991). An optimum volume of 100 ml (53.7) was obtained by Bindu and Ramabhadran (2010), while Sridharan et al. (1991) indicated that 100–125 ml of Calgon 33.7 yielded the optimum clay content. The results obtained by Sridharan et al. (1991) were in close agreement with the concentrations and volumes prescribed by the British and Indian standards.
- Low (4%) and high (9%) concentrations of Calgon proved to be less effective in the case of both soils.
- In the case of the alluvial soil, 5% and 8% Calgon concentrations were more effective than 4.2% Calgon for volumes up to 175 ml.
- In the case of the black soil, the 5% Calgon was more effective than 4.2% Calgon for volumes ranging from 125 ml to 175 ml.
- In the case of the black soil, for all the concentrations of dispersing agent, any further increase in volume of chemical

<table>
<thead>
<tr>
<th>Volume (ml)</th>
<th>Alluvial soil</th>
<th>Black soil</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4%</td>
<td>4.2%</td>
</tr>
<tr>
<td>100</td>
<td>19.7</td>
<td>22.8</td>
</tr>
<tr>
<td>125</td>
<td>20.1</td>
<td>21.4</td>
</tr>
<tr>
<td>150</td>
<td>20.2</td>
<td>23</td>
</tr>
<tr>
<td>175</td>
<td>19.9</td>
<td>21.5</td>
</tr>
<tr>
<td>200</td>
<td>18</td>
<td>24</td>
</tr>
<tr>
<td>225</td>
<td>29.5</td>
<td>19.7</td>
</tr>
<tr>
<td>250</td>
<td>20.1</td>
<td>19.5</td>
</tr>
<tr>
<td>275</td>
<td>24.7</td>
<td>23.2</td>
</tr>
<tr>
<td>300</td>
<td>24.9</td>
<td>20.7</td>
</tr>
<tr>
<td>325</td>
<td>20.5</td>
<td>19.3</td>
</tr>
<tr>
<td>350</td>
<td>17</td>
<td></td>
</tr>
<tr>
<td>375</td>
<td>15.5</td>
<td></td>
</tr>
</tbody>
</table>

The bold black numerals indicate the optimum concentration and volume for each soil type.
after attaining optimum volume generally resulted in a decrease in clay content. Similar results were obtained by Bindu and Ramabhadran (2010) and Sridharan et al. (1991).

With regard to the concentrations of Calgon, no general trend was evident regarding the relative effect of the quantities (ratio of) sodium carbonate and sodium hexametaphosphate on the results obtained for both soils. However, in the black soils, with the exception of the 4% Calgon results, an increase in sodium hexametaphosphate resulted in a decrease in clay content. This decrease is due to saturation absorption of the dispersants onto the clay particles, after which aggregation of particles might occur (Bindu & Ramabhadran 2010).

**Sodium pyrophosphate decahydrate (NaPP)**

The effect of different concentrations and volumes of NaPP for the alluvial and black soils are shown in Figures 5 and 6 respectively. The clay content determined by the different concentration and volume combinations are shown in Table 5. The best and worst results for each concentration are shown in green and red respectively.

From Figures 5 and 6 and Table 5 the following is evident:

- In the case of the alluvial soil, 40 ml of NaPP 5% gave the maximum clay content of 26%, while 40 ml of NaPP 3.6% yielded a clay content of 25.5% (which is a little less than the former). Hence, NaPP 3.6% may be considered as the optimum concentration for the alluvial soil.
- When the test was conducted on the black clay, the clay fraction obtained was at a maximum (35.2%) when using 80 ml with a 3.6% concentration. Hence, 80 ml of NaPP 3.6% is considered as the optimum concentration and volume for the black soil.
- In general, the 3.6% NaPP appears to be the most effective concentration in the case of both soils.
- Low (3%) and high (7%) concentrations of NaPP generally proved to be less effective in the case of both soils.

![Table 5](image)

**Table 5** Variation of clay content depending on the concentration and volume of NaPP added to alluvial soil and black soil

<table>
<thead>
<tr>
<th>Volume (ml)</th>
<th>Alluvial soil</th>
<th>Black soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>18.4 20 24 24 24.4 10 20.4 20.5 22.3 25.4</td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>17.7 25.5 26 25.2 24.7 20 27 26.1 33.5 33.4</td>
<td></td>
</tr>
<tr>
<td>60</td>
<td>20.5 24 23.3 25 24.3 22.2 32 32.1 32.5 28</td>
<td></td>
</tr>
<tr>
<td>80</td>
<td>19.3 25.4 24.2 24 23.5 21.9 35.2 28.3 34.2 34</td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>20.3 24.9 24.9 23.5 23.1 26 32.5 34 33.5 33.4</td>
<td></td>
</tr>
<tr>
<td>120</td>
<td>18.7 25.2 25.3 24.7 21.2 26 34 34 33.9 22.7</td>
<td></td>
</tr>
<tr>
<td>140</td>
<td>25.1 24.7 21.2 27 32.1 34.8 32 26</td>
<td></td>
</tr>
<tr>
<td>160</td>
<td>21.9 24.7 25 33 32.3 34 22.5</td>
<td></td>
</tr>
<tr>
<td>180</td>
<td>21.2 25 25.7 31.8 26 21.3</td>
<td></td>
</tr>
<tr>
<td>200</td>
<td>21.1 23.2</td>
<td>20.8 20</td>
</tr>
<tr>
<td>220</td>
<td></td>
<td>21</td>
</tr>
<tr>
<td>240</td>
<td></td>
<td>17.9</td>
</tr>
</tbody>
</table>

The bold black numerals indicate the optimum concentration and volume for each soil type.

![Table 6](image)

**Table 6** Summary of results

<table>
<thead>
<tr>
<th>Property</th>
<th>Alluvial soil</th>
<th>Black soil</th>
<th>Calgon</th>
<th>NaPP</th>
<th>Calgon</th>
<th>NaPP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Optimum concentration (%)</td>
<td>4.2 3.6</td>
<td>4.2 3.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Optimum volume (ml)</td>
<td>225 40</td>
<td>200 80</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay content (%)</td>
<td>29.5 25.5</td>
<td>20 80</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>∆% in clay content (relative to Table 1)</td>
<td>38 19</td>
<td>9.4 10</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Activity</td>
<td>0.54 0.63</td>
<td>0.97 0.97</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>∆% in activity (relative to Table 1)</td>
<td>−28 −16</td>
<td>−9.4 −9.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Summary of results**

Table 6 shows the optimum concentrations and volumes of Calgon and NaPP for the alluvial and black soil, and the clay contents yielded by these dispersing agents.

With reference to Table 6, it is evident that the Calgon and NaPP yielded the highest clay content for the alluvial and black soils respectively.

Furthermore, Figures 3 to 6 and Tables 4 and 5 indicate that increase in volume beyond the optimum volume resulted in a decrease in the percentage clay content in most cases. However, a contradictory trend was noticed where, with an increase in volume, there was an increase in percentage of clay content. The reason for this is that, with an increase in the volume of dispersing agent in the companion ‘blank’ solutions, the hydrometer readings increased, but the increase was not constant.

It was also found that, in the case of both soils and both dispersing agents, there was no correlation between the mass of dispersing agent granules in solution and the clay content yielded.

Furthermore, when comparing the maximum clay content of the two soils determined as part of this investigation (Table 6) with those in Table 1, the maximum clay content for the alluvial soil increased by 38% (from 21.4% to 29.5%). The maximum clay content for the black soil increased by 10% (from 32% to 35.2%). These increases in clay content resulted in decreases in the activity of the alluvial and black soils of 28% (from 0.75 to 0.54) and 10% (from 1.07 to 0.97) respectively.

**CONCLUSIONS**

The following conclusions were drawn from the study conducted:

- Tests with different dispersing agents clearly indicated that the percentage of clay-sized material can vary significantly, depending on the type of dispersing agent. In this investigation, which included ten dispersing agent types, Calgon and sodium pyrophosphate decahydrate...
(NaPP) were the most effective dispersing agent types.

In the case of both soils, the optimum concentrations of Calgon and NaPP were found to be 4.2% and 3.6% respectively. Furthermore, 225 ml of Calgon yielded the highest clay content in the case of the alluvial soil, whereas 80 ml of NaPP yielded the highest clay content in the case of the black soil.

Relatively high and low concentrations of Calgon and NaPP yielded low clay content in the case of both soil types.

No correlation exists between the mass of the dispersing agent granules in solution and the clay content yielded.

The results of this investigation confirm the findings of Means and Parcher (1963) that different dispersing agent types are more effective with certain clay types.

Finally, the effect of the dispersing agent on the hydrometer readings, particularly in the case of relatively high volumes, was considerable and hence should be accounted for by accordingly correcting (reducing) the hydrometer readings. The current South African method, SANS 3001 (SANS 2014), which utilises sodium hexametaphosphate as a dispersing agent, makes provision for such a correction to the hydrometer readings.

REFERENCES


The possible rate of transition to lower-carbon housing

P Lloyd

Two of the challenges facing any transition to a lower-carbon economy in the building sector are the questions of how rapidly the existing low-efficiency stock of domestic housing can be replaced with more efficient housing, and how efficient the new housing stock can be made. This paper therefore develops a model for the replacement of the global housing stock as it ages, and considers what the demand for new housing stock is likely to be. One driver will clearly be the increasing population. Another will be economic growth, which has the counter-intuitive effect of reducing the average occupancy of homes as nations develop economically. This not only accelerates the underlying rate of increase in new housing, forced by the increasing global population, but also offers opportunities for higher-value, more energy-efficient homes. Moreover, economic development is usually associated with greater levels of urbanisation, which allows greater use of multi-dwelling buildings with associated improved efficiency potential. Nevertheless, the lifetime of most homes is inherently long in comparison with the apparent urgency of reducing energy demand, and thus lower carbon emissions. It is concluded that, per se, more efficient housing is unlikely to play a significant part in the transition to a lower-carbon world over the next 35 years until 2050.

INTRODUCTION

The residential part of the global building sector consumed about 16 EWh of energy in 2010 (IEA 2015), and that is expected to grow to about 30 EWh by 2050 (Lucon & Vorsatz 2014) under a business-as-usual scenario. This is equivalent to a growth rate of about 1.7% per annum. Some of the growth will undoubtedly be due to population growth (Table 1).

Globally the average home houses about 3.4 persons, so the growth from 7.3 billion to 9.7 billion people by 2050 implies about 700 million new homes. In addition, the existing stock of homes will age and will have to be replaced. The new homes create the opportunity to introduce energy-efficient structures, provided there is the political will to enforce building regulations that require energy efficiency. In addition, there are opportunities to improve the efficiency of the existing stock. This paper therefore aims to develop a model which will enable some quantification of the possibilities for moving the housing stock into a more energy-efficient state, and to develop some ideas about the rate at which this could occur under different levels of policy intervention.

THE PRESENT SITUATION

At present there are approximately 1.9 billion homes in the world. This is derived from a large sample reported in Wikipedia (2015), which includes the peer-reviewed sources of the data. A portion of this sample – nations

Table 1 Population of the world and major areas, 2015, 2030 and 2050, according to medium projection (UNDESA 2015)

<table>
<thead>
<tr>
<th>Major areas</th>
<th>Population (millions)</th>
<th>Annual growth 2015–2050</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2015</td>
<td>2030</td>
</tr>
<tr>
<td>World</td>
<td>7 349</td>
<td>8 501</td>
</tr>
<tr>
<td>Africa</td>
<td>1 186</td>
<td>1 679</td>
</tr>
<tr>
<td>Asia</td>
<td>4 393</td>
<td>4 923</td>
</tr>
<tr>
<td>Europe</td>
<td>738</td>
<td>734</td>
</tr>
<tr>
<td>Latin America and Caribbean</td>
<td>634</td>
<td>721</td>
</tr>
<tr>
<td>North America</td>
<td>358</td>
<td>396</td>
</tr>
<tr>
<td>Oceania</td>
<td>39</td>
<td>47</td>
</tr>
</tbody>
</table>

Keywords: building sector, housing, energy efficiency, demand, population growth, occupancy
with more than 10 million households in the year reported – is given in Table 2 in descending order of number of households. The population of all countries in the sample totalled 4.8 billion, which was 68% of the global population in 2011. The global population estimate in 2011 was taken from the World Development Indicators (World Bank 2015). The sample totalled 1.4 billion homes, or an average occupancy of 3.4 persons per dwelling.

It was observed that there is a tendency for less developed nations to have a higher occupancy than more developed nations. This was tested against all nations in the sample, using the gross domestic product (GDP) per capita (World Bank 2015) as a measure of the state of development. A reasonable correlation resulted, which is shown in Figure 1. The occupancy level in nations for which there was no data on the number of dwellings was therefore estimated from the relationship:

\[ y = 35.87x^{-0.248} \]  

where \( y \) is the occupancy and \( x \) the GDP per capita for purchasing power parity (PPP) in 2011 US$. When this relationship was used to estimate the number of homes in those countries that were not included in the sample, but for which there were data on the GDP per capita, the total number of homes was estimated to be 1.86 billion with an average occupancy of 3.6 persons per home. The average occupancy (3.6) was somewhat higher than the average occupancy in the original sample (3.4), because poorer nations were under-represented in the original sample. Additional nations with more than 10 million homes were identified, which are shown in Table 3.

Having derived a reasonably complete picture of the world, it was then possible to aggregate the various nations into regional groupings as shown in Table 4.

**FUTURE PROSPECTS**

The factors that will change the number of dwelling units over the next 35 years are:

1. An increase in the number of units required to house a growing population.
2. An increase in the number of units driven by improved economies and therefore higher GDP per capita, leading to lower occupancy rates (this is also a reflection in areas such as South America and Africa of growing urbanisation and lower family sizes as a result).
3. The replacement of existing housing due to age or lack of suitability (e.g. abandoned rural homes).

---

### Table 2 Households and population of nations with more than 10 million homes (Wikipedia 2015)

<table>
<thead>
<tr>
<th>Country</th>
<th>Household population</th>
<th>Households</th>
<th>Household size</th>
<th>Year</th>
</tr>
</thead>
<tbody>
<tr>
<td>China</td>
<td>1 367 820 000</td>
<td>455 940 000</td>
<td>3</td>
<td>2012</td>
</tr>
<tr>
<td>India</td>
<td>1 018 865 868</td>
<td>192 671 808</td>
<td>5.3</td>
<td>2001</td>
</tr>
<tr>
<td>United States</td>
<td>304 130 000</td>
<td>117 538 000</td>
<td>2.6</td>
<td>2009</td>
</tr>
<tr>
<td>Brazil</td>
<td>189 790 211</td>
<td>57 324 167</td>
<td>3.3</td>
<td>2010</td>
</tr>
<tr>
<td>Russia</td>
<td>142 754 098</td>
<td>52 711 375</td>
<td>2.7</td>
<td>2002</td>
</tr>
<tr>
<td>Japan</td>
<td>124 973 207</td>
<td>49 062 530</td>
<td>2.5</td>
<td>2005</td>
</tr>
<tr>
<td>Germany</td>
<td>80 645 605</td>
<td>40 076 000</td>
<td>2.0</td>
<td>2008</td>
</tr>
<tr>
<td>United Kingdom</td>
<td>64 106 779</td>
<td>26 473 000</td>
<td>2.4</td>
<td>2011</td>
</tr>
<tr>
<td>France</td>
<td>65 920 302</td>
<td>25 253 000</td>
<td>2.6</td>
<td>2005</td>
</tr>
<tr>
<td>Italy</td>
<td>60 233 948</td>
<td>23 848 000</td>
<td>2.5</td>
<td>2008</td>
</tr>
<tr>
<td>Vietnam</td>
<td>89 708 900</td>
<td>44 442 322</td>
<td>4.0</td>
<td>2009</td>
</tr>
<tr>
<td>Mexico</td>
<td>95 380 242</td>
<td>22 686 196</td>
<td>4.3</td>
<td>2000</td>
</tr>
<tr>
<td>Philippines</td>
<td>98 393 574</td>
<td>18 539 769</td>
<td>5.3</td>
<td>2007</td>
</tr>
<tr>
<td>Turkey</td>
<td>74 932 641</td>
<td>17 794 238</td>
<td>4.2</td>
<td>2008</td>
</tr>
<tr>
<td>Iran</td>
<td>77 447 168</td>
<td>17 352 686</td>
<td>4.5</td>
<td>2006</td>
</tr>
<tr>
<td>Ukraine</td>
<td>45 489 600</td>
<td>17 199 000</td>
<td>2.6</td>
<td>2008</td>
</tr>
<tr>
<td>Spain</td>
<td>46 620 045</td>
<td>16 741 379</td>
<td>2.8</td>
<td>2008</td>
</tr>
<tr>
<td>South Korea</td>
<td>45 452 526</td>
<td>15 887 128</td>
<td>2.9</td>
<td>2005</td>
</tr>
<tr>
<td>Ethiopia</td>
<td>73 302 305</td>
<td>15 634 304</td>
<td>4.7</td>
<td>2007</td>
</tr>
<tr>
<td>Poland</td>
<td>37 812 741</td>
<td>13 337 040</td>
<td>2.8</td>
<td>2002</td>
</tr>
<tr>
<td>Canada</td>
<td>35 158 304</td>
<td>12 437 470</td>
<td>2.8</td>
<td>2006</td>
</tr>
<tr>
<td>Argentina</td>
<td>39 672 520</td>
<td>12 171 675</td>
<td>3.3</td>
<td>2010</td>
</tr>
<tr>
<td>South Africa</td>
<td>53 157 490</td>
<td>11 205 705</td>
<td>4.7</td>
<td>2001</td>
</tr>
<tr>
<td>Colombia</td>
<td>41 174 853</td>
<td>10 570 899</td>
<td>3.9</td>
<td>2005</td>
</tr>
</tbody>
</table>

---

**Figure 1 Relation between occupancy of homes and GDP per capita**
The first of these is derived directly from the growth rates given in Table 1. The second is the subject of considerable guesswork. One set of pundits believes that “growth in the OECD and emerging G20 countries is likely to decelerate from 3.4% in 1996–2010 to 2.7% in 2010–2060.” (Braconier et al. 2014). Another study (PWC 2015) estimates the “average real GDP growth rates for the 32 economies covered in this study over the period to 2050. Newly emerging economies such as Nigeria and Vietnam could grow at 5% or more per annum on average over this period, whilst the growth of established emerging economies such as China may moderate to around 3–4%. Advanced economies are projected to grow at around 1.5–2.5% per annum in the long run, with variations reflecting different working age population growth to a significant degree.” Several other studies have reached similar conclusions, so we assume the economic growth rates shown in Table 5.

The third and final factor is the rate of replacement of the existing housing stock. Data from Europe and North America indicated that demolition rates were of the order of 25% of new construction rates, so that it was necessary to build at least 25% more homes than were needed for new entrants into the market (Carliner 1990). However, the demolition rates are precisely what they indicate – the rate at which homes are physically destroyed. Other homes are lost because they are abandoned, so that the actual loss rate is significantly higher than the demolition rate.

Eventually the lack of data was resolved by making what seemed to be reasonable assumptions about how much of the 2011 housing stock was likely still to be in use by 2050, and calculating how many extra dwelling units would have to be constructed each year, over and above the number of units needed to house the growing population. An underlying assumption is that all of the houses built between 2011 and 2050 will remain in use. The calculations for Sub-Saharan Africa for the first few years are illustrated in Table 6.

The first parameter is the annual rate of population growth (from Table 1), and the second, the assumed annual GDP growth in constant US dollars (from Table 5). The third is the factor giving the additional number of homes to be built each year to make up for demolition or abandonment, and the fourth is the result of the calculation, the percentage of the 2011 housing stock still in use in 2050. First the population and the GDP are calculated each year, from which the GDP per capita is calculated, which gives the occupancy, persons per dwelling.

### Table 3 Estimates of additional nations with more than 10 million dwellings

<table>
<thead>
<tr>
<th>Country</th>
<th>Household population</th>
<th>Households</th>
<th>Household size</th>
</tr>
</thead>
<tbody>
<tr>
<td>Indonesia</td>
<td>249 865 631</td>
<td>64 690 040</td>
<td>3.9</td>
</tr>
<tr>
<td>Pakistan</td>
<td>182 142 594</td>
<td>40 456 530</td>
<td>4.5</td>
</tr>
<tr>
<td>Nigeria</td>
<td>173 615 345</td>
<td>39 362 600</td>
<td>4.4</td>
</tr>
<tr>
<td>Bangladesh</td>
<td>156 594 962</td>
<td>30 078 625</td>
<td>5.2</td>
</tr>
<tr>
<td>Egypt</td>
<td>82 056 378</td>
<td>22 457 024</td>
<td>3.7</td>
</tr>
<tr>
<td>Vietnam</td>
<td>89 708 900</td>
<td>22 444 322</td>
<td>4.0</td>
</tr>
<tr>
<td>Thailand</td>
<td>67 010 502</td>
<td>19 291 457</td>
<td>3.5</td>
</tr>
<tr>
<td>Saudi Arabia</td>
<td>28 828 870</td>
<td>11 648 044</td>
<td>2.5</td>
</tr>
<tr>
<td>Algeria</td>
<td>39 208 194</td>
<td>11 231 953</td>
<td>3.5</td>
</tr>
</tbody>
</table>

### Table 4 Population, households and economies of regions of the world in 2011

<table>
<thead>
<tr>
<th>Region</th>
<th>Population*</th>
<th>Households</th>
<th>Occupancy</th>
<th>GDP/cap</th>
<th>GDP, 2011 $</th>
</tr>
</thead>
<tbody>
<tr>
<td>Africa</td>
<td>1 064 155 026</td>
<td>212 431 669</td>
<td>5.0</td>
<td>4 067</td>
<td>4.33E+12</td>
</tr>
<tr>
<td>Asia</td>
<td>3 927 049 240</td>
<td>1 036 500 083</td>
<td>3.8</td>
<td>9 229</td>
<td>3.62E+13</td>
</tr>
<tr>
<td>Europe</td>
<td>823 051 563</td>
<td>311 607 623</td>
<td>2.6</td>
<td>27 722</td>
<td>2.28E+13</td>
</tr>
<tr>
<td>North America</td>
<td>435 107 405</td>
<td>152 393 278</td>
<td>2.9</td>
<td>41 578</td>
<td>1.81E+13</td>
</tr>
<tr>
<td>Oceania</td>
<td>35 357 623</td>
<td>11 279 277</td>
<td>3.1</td>
<td>26 957</td>
<td>9.53E+11</td>
</tr>
<tr>
<td>South America</td>
<td>471 458 411</td>
<td>132 223 849</td>
<td>3.6</td>
<td>12 194</td>
<td>5.75E+12</td>
</tr>
</tbody>
</table>

* Note that, because of changes in the definition of regions, the population of the various regions is slightly different from that of Table 1.

### Table 5 Assumed economic growth rates; constant US$ PPP b

<table>
<thead>
<tr>
<th>Growth rate %/a</th>
<th>Region</th>
<th>Growth rate %/a</th>
<th>Region</th>
<th>Growth rate %/a</th>
<th>Region</th>
<th>Growth rate %/a</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPA 3.7</td>
<td>PAS 2.1</td>
<td>LAM 3.1</td>
<td>SSA 3.5</td>
<td>EEU 1.3</td>
<td>POE 1.9</td>
<td>MNA 2.5</td>
</tr>
<tr>
<td>EEU 1.3</td>
<td>SAS 3</td>
<td>NAM 2.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table 6 Calculation of new build for new entrants (net build) and total build allowing for demolition and abandonment (gross build)

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Region</th>
<th>Factor</th>
<th>2011</th>
<th>2012</th>
<th>2013</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.13%</td>
<td>SSA</td>
<td>Population</td>
<td>843 533 804</td>
<td>861 501 074</td>
<td>879 851 047</td>
</tr>
<tr>
<td>3.50%</td>
<td>GDP 2011 $</td>
<td>2.48E+12</td>
<td>2.567E+12</td>
<td>2.657E+12</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GDP/cap</td>
<td>2.94E+03</td>
<td>2.98E+03</td>
<td>3.02E+03</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Occupancy</td>
<td>4.95</td>
<td>4.93</td>
<td>4.92</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Households</td>
<td>1.70E+08</td>
<td>1.75E+08</td>
<td>1.79E+08</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Net build</td>
<td>4.21E+06</td>
<td>4.31E+06</td>
<td></td>
<td></td>
</tr>
<tr>
<td>25%</td>
<td>Gross build</td>
<td>5.27E+06</td>
<td>5.40E+06</td>
<td></td>
<td></td>
</tr>
<tr>
<td>60%</td>
<td>Demolitions</td>
<td>1.06E+06</td>
<td>1.09E+06</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2011 stock remaining</td>
<td>1.70E+08</td>
<td>1.69E+08</td>
<td>1.68E+08</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
by the equation derived from Figure 1. The population divided by the occupancy gives the number of households in total. The new builds (net build) are then the difference between the number of households from one year to the next. The total of new builds (gross build) is the net build times (1+parameter), where the parameter is 25% in this case. The number of demolitions and abandonments in each year is then the difference between the gross build and the net build, and each year the original stock is reduced by the number of demolitions and abandonments of that year.

Continuing this process through 2050, the remainder of the 2011 stock is calculated, and the ratio (2011 stock remaining in 2050)/2011 stock then gives the fourth parameter, 60% in this case. It is argued that a loss of 40% of the original housing stock over 39 years is quite reasonable in the case of Africa, where the quality of the original stock is low and rapid urbanisation is taking place, so there is likely to be a rapid abandonment of rural homes. The results of this exercise for all the considered regions are summarised in Table 7.

What this indicates is that, over the next 35 years, there will be new stock needed of nearly 750 million dwellings in Asia (CPA+SAS+PAS), 350 million in Africa, over 100 million in Latin America, and less than 100 million in the whole of Europe and in North America. Opportunities for introducing energy-efficient housing are clearly greatest in Asia and Africa. This is illustrated in Figure 2.

### Table 7 Summary of new homes required between 2015 and 2050

<table>
<thead>
<tr>
<th>Region</th>
<th>2011 housing</th>
<th>2050 housing</th>
<th>% 2011 stock surviving</th>
<th>Net new dwellings occupied</th>
<th>Gross new dwellings built</th>
<th>% extra housing built each year</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPA</td>
<td>3.83E+08</td>
<td>6.34E+08</td>
<td>75%</td>
<td>2.51E+08</td>
<td>3.47E+08</td>
<td>38%</td>
</tr>
<tr>
<td>EEU</td>
<td>4.31E+07</td>
<td>4.07E+07</td>
<td>90%</td>
<td>4.15E+06</td>
<td>8.36E+07</td>
<td>107%</td>
</tr>
<tr>
<td>FSU</td>
<td>8.37E+07</td>
<td>1.13E+08</td>
<td>80%</td>
<td>2.88E+07</td>
<td>4.56E+07</td>
<td>58%</td>
</tr>
<tr>
<td>LAM</td>
<td>1.65E+08</td>
<td>2.64E+08</td>
<td>80%</td>
<td>9.98E+07</td>
<td>1.33E+08</td>
<td>33%</td>
</tr>
<tr>
<td>MNA</td>
<td>1.36E+08</td>
<td>1.98E+08</td>
<td>80%</td>
<td>6.15E+07</td>
<td>8.88E+07</td>
<td>44%</td>
</tr>
<tr>
<td>NAM</td>
<td>1.38E+08</td>
<td>2.05E+08</td>
<td>90%</td>
<td>6.71E+07</td>
<td>8.09E+07</td>
<td>21%</td>
</tr>
<tr>
<td>PAS</td>
<td>1.60E+08</td>
<td>2.68E+08</td>
<td>70%</td>
<td>1.09E+08</td>
<td>1.56E+08</td>
<td>44%</td>
</tr>
<tr>
<td>POE</td>
<td>7.26E+07</td>
<td>1.01E+08</td>
<td>90%</td>
<td>2.88E+07</td>
<td>3.59E+07</td>
<td>25%</td>
</tr>
<tr>
<td>SAS</td>
<td>3.20E+08</td>
<td>4.96E+08</td>
<td>75%</td>
<td>1.76E+08</td>
<td>2.56E+08</td>
<td>45%</td>
</tr>
<tr>
<td>SSA</td>
<td>1.70E+08</td>
<td>4.41E+08</td>
<td>60%</td>
<td>2.71E+08</td>
<td>3.39E+08</td>
<td>25%</td>
</tr>
<tr>
<td>WEU</td>
<td>1.82E+08</td>
<td>2.02E+08</td>
<td>90%</td>
<td>2.09E+07</td>
<td>3.90E+07</td>
<td>87%</td>
</tr>
</tbody>
</table>

### Figure 2 Regional needs for new housing by 2050

### Figure 3 Annual domestic energy consumption in global regions, 2011
communication and computation dominate. This is followed by small appliances such as kettles and irons. It is typically five years before the first major appliance is acquired, usually a refrigerator (Lloyd 2012). After about ten years the recently electrified home will start to look like the average low-income home in a developed society.

The driver for development will be economic growth. There is a strong relationship between energy consumption and wealth, as indicated in Figure 4 (World Bank 2015). A very similar relationship can be traced back over the past 50 years; it is significant and persistent. High-energy-consuming nations tend to be those having lower ambient temperatures; conversely, lower consuming nations tend to have a tropical climate. It is possible to strengthen the relationship by correcting for the ambient temperature effect, which improves the regression coefficient $R^2$ to over 0.9, but this does not alter the relationship significantly.

We can therefore use this relationship to estimate the probable growth in energy demand until 2050. To allow for the slow uptake, the economic growth until 2045 will be assumed as the measure to the energy demand growth to be expected until 2050. Then the domestic energy consumption in each region in 2011 shown in Figure 3 is inflated according to the relation between GDP per capita in 2011 and 2045, using the equation

$$y = 0.00676 x^{0.8366}$$

shown in Figure 4. The results are given in Figure 5.

The results seem reasonable. North America will continue to dominate the per capita consumption. There will be strong growth in Eastern Europe, the Former Soviet Union and the Pacific OECD. The Centrally Planned Asian states will also see strong growth, but off a low base. Per capita consumption in the rest of the world will remain low, but growing.

However, the picture changes dramatically when we look at the total energy consumed, as shown in Figure 6. The total energy consumption in the sector will have grown from about 16 EWh in 2011 to nearly 33 EWh. The total consumption in Centrally Planned Asia will be approaching that of North America, and the southern Asian states will be approaching that of Western Europe. The greatest proportional growth will have taken place in Sub-Saharan Africa, with over three times its 2011 consumption.

There are interesting features in the way in which the various economies grow their energy consumption in this model. Figure 7 tracks the dynamics for each region. Western Europe (WEU), the Former Soviet Union (FSU), Eastern Europe (EEU) and the Pacific Asian States (PAS) all show essentially linear

![Figure 4 Relation between GDP and energy consumption, 2011](image1)

![Figure 5 Expected change in domestic energy consumption, 2011–2050](image2)

![Figure 6 Total domestic energy consumption, 2011 and 2050](image3)
growth. All have relatively low population growth and/or economic growth. The combination of high population growth and strong economic growth leads to an exponential increase in energy consumption, as so clearly shown by the Centrally Planned Asian (CPA) states. If there is to be any hope of achieving a lower-carbon world, it is clearly important to find means of curbing such exponential growth.

A LOWER-CARBON WORLD BY 2050
In previous sections we have seen how business-as-usual models and the anticipated growth of global economies, with some large-population nations developing rapidly, are likely to lead to large increases in the demand for energy. Today there are concerns that China and the other Centrally Planned Asian states will not be able to control their emissions before 2030. What we have shown is the following:

- Those regions that already have high per capita consumption are unlikely to reduce their per capita consumptions, and will probably increase their total consumption.

- Those regions that are developing rapidly are likely to increase their per capita consumption rapidly, and their population increase means that their total consumption is likely to rise rapidly.

In the light of this, we have to ask what assumptions might be necessary to reduce the growth in energy consumption. First, of course, is the possibility of improving the energy performance of new buildings. By 2050, about 1.5 billion new homes will have been built, of which 1.1 billion will be newly occupied and 0.4 billion occupied by people who previously lived in homes that were demolished or abandoned during the period. The total number of dwellings by 2050 will be about 3 billion, so that new dwellings will represent about half the stock. There are clearly opportunities for improving the energy performance of these new dwellings.

Quite wide experience has shown that it is probably possible to achieve about a 40% reduction in energy consumption in new homes without very significant changes in building practice. As a thought experiment, therefore, let us assume that by 2020 there is local legislation in some parts of the world requiring such a reduction in the design of every new home, and that by 2025 such legislation is widespread. Then the domestic demand pattern shown in Figure 7 will change to that shown in Figure 8.

Comparison between the two figures shows that there is now very low growth in seven of the regions, and that the exponential growth shown in some regions in Figure 7 is now close to linear. The reduction in energy from 33 to 26 EWh/a represents about a 25% reduction from the business-as-usual scenario, but it is still some 75% above the 2011 consumption, with growth driven by NAM, CPA, WEU and SAS in particular. NAM and WEU start from a high level of consumption (and high wealth), and both enjoy quite low population growth rates, so it is evident they should strive for additional curbs on growth in energy consumption. In contrast, CPA and SAS both start from a relatively low base and have to cope with significant development issues, so it seems there is little that they could do to reduce their energy demand further.

Clearly there are opportunities to enforce greater energy efficiency in existing dwellings, which could further reduce the demand, but the model foresees very aggressive action in respect of new homes, which will make up about 50% of the total homes by 2050, so improved efficiency in existing homes will not make a great difference to the outcome.

DISCUSSION AND CONCLUSIONS
It seems that the possibility of achieving a lower-carbon world through improvements in the energy efficiency of homes is limited. Of course, it is possible to improve the energy performance of existing homes, but Table 7 shows that, even though a significant fraction of today’s housing stock will survive until 2050, by 2050 the new stock will be as large as the existing stock. The focus should therefore be on ensuring that the new stock is energy efficient.

It should be possible to achieve a lower-energy demand than a business-as-usual scenario, but there is still likely to be a significant growth in energy demand by 2050. The growth in the demand for homes will drive the energy growth regardless of efficiency measures.

There are two drivers for more homes, first and obviously, the growth in the population, and secondly, the tendency for smaller families and thus less people per dwelling as economic development takes place. Even in developing nations, where population control measures have proved effective, there is a marked reduction in the number of people per dwelling and a rise in single-person occupancy (China Daily 2015).

A driver for increased energy consumption has been increasing domestic use of energy, driven by wider use of electricity in homes in developing nations. Clearly, energy-efficient appliances will play a part in helping to reduce the demand for energy, but even if all appliances are A-rated rather than D-rated, at best savings of the order of 50% are achievable (SEIA 2015), which
is insufficient to reduce 2050 consumption to less than 2011 consumption. There is already a reasonable population owning A-rated appliances, so the full benefit is no longer available.

It can only be concluded that, in the area of domestic homes, the opportunities for containing the growth of energy consumption over the next 35 years are very limited; the chances of reducing the energy consumption are virtually nil. This is the only conclusion responsible engineers can reach when they look at the likely growth of housing over the next generation. There is a drive to a lower-carbon world, but practical considerations indicate that, from the demand side, there is little that can be done to reduce demand significantly, particularly in the face of a strongly growing market. It is possible that low-carbon energy generation may succeed in reducing emissions on the supply side of the equation, but whether these will be able to do so at the same level of service delivery that fossil fuel-, nuclear- and hydro-powered generation currently afford, is doubtful, unless cost-effective energy storage systems can be developed, something which is widely sought, but tantalisingly remote at present.

NOTES
a. Note that there is some discrepancy in Lucon & Ürge-Vorsatz 2014. Figure 9.3 gives what appears to be incorrect data for per capita domestic energy consumption. This data led to an estimate of 24.3 PWh in 2010, as shown in Figure 9.4. The data of Figure 9.5, however, gives far more reasonable data for the domestic energy consumption. This data leads to an estimate of 15.8 PWh for the global consumption, which we use here.
b. CPA Centrally Planned Asia; EEU Eastern Europe; FSU Former Soviet Union; LAM Latin America; MNA Middle East North Africa; NAM North America; PAS Pacific Archipelago States; POE Pacific OECD; SAS Southern Asia; SSA Sub-Saharan Africa; WEU Western Europe.
c. The USEPA claims its Energy Star homes to be 15–30% more energy efficient (e.g. https://www.energystar.gov/index.cfm/t=products).

REFERENCES

Figure 8 Predicted growth in energy demand after implementing energy-efficient regulations for new homes from 2020
Seismic evaluation of the northbound N1/R300 bridge interchange

M N Solms, T N Haas

The design of the Stellenberg Interchange was finalised in 1982, with construction completed in 1986. The bridge was designed using a code of practice which did not include any requirements for seismic excitation. This code was superseded by the Code of Practice for the Design of Highway Bridges and Culverts, which provides detailed analysis guidelines for bridges located in seismic-prone areas. According to this code, the bridge is located in a seismic-prone area with an anticipated peak ground acceleration of 0.1 g. Current research suggests that this region could be exposed to a peak ground acceleration of approximately 0.2 g. Upon inspection of the bridge, it was noted that the bridge does not conform to modern-day best practice guidelines for bridges located in seismic-prone regions. These factors necessitated an exploratory investigation to determine whether the bridge can sustain earthquake magnitudes between 0.05 g and 0.2 g. The study was conducted by experimentally determining the natural frequencies with its corresponding mode shapes, which were used to calibrate a finite element model. The finite element model was subjected to different magnitude earthquakes to determine its structural integrity. The results show that, for an earthquake of 0.1 g, the bending moment at one of the column bases is exceeded, while two other column base moments are within 15% of its design capacity. For a 0.2 g magnitude earthquake, the design bending moments at five columns are significantly exceeded, while three other columns’ design moments are close to being exceeded. The exceedance of the design moments could lead to significant damage, with the possibility of collapse of the bridge.

INTRODUCTION

Background

The R300 regional road provides a link between two national highways, namely the N1 and the N2. The N1-R300, commonly referred to as the Stellenberg Interchange, incorporates two interchange bridges which cross the N1. The Stellenberg Interchange is located in the southwestern region of the Western Cape Province, approximately 27 km from Cape Town’s CBD. The N1 in this area is heavily congested during morning peak-hour traffic flowing into Cape Town’s CBD, while the same applies for the return leg in the afternoon. According to TomTom, Cape Town is the most traffic-congested city in South Africa. Therefore, any damage to or collapse of one of the Stellenberg Interchange bridges would cause serious disruption to traffic flow to and from Cape Town’s CBD, with associated economic impacts of traffic having to use other available routes.

The structural design of the Stellenberg Interchange was completed in 1982, while construction was completed early in 1986. The bridge was designed based on the Planning Manual Part 3: Bridge Design Manual of 1977 of the Cape Provincial Administration Department (CPAD) of Roads. Seismic excitation was, however, not considered in the design, as this design code did not stipulate provisions for the effects of seismicity on bridges. Although the bridges were not designed for seismicity, it adhered to the provisions of the code at the time.

The CPAD code was superseded in 1981 by the Technical Methods for Highways No 7, also known as TMH7 for the design of bridges and culverts in South Africa. TMH7 is based on Compléments au Code-Modèle CEB-FIP of 1978. TMH7 differs significantly from CPAD in that it has clear guidelines for seismicity and the regions to which it should be applied. TMH7 clearly indicates that the interchange is located in a seismic-hazard region which is susceptible to a peak ground acceleration (PGA) of 0.1 g with a 10% probability of exceedance in a 100-year period, equating in a return period of 950 years. Although the seismic intensity is classified as moderate, infrastructure not designed for this level of seismicity could experience serious damage or collapse when subjected to this load effect.

The southwestern region of the Western Cape Province where the interchange is located is susceptible to natural seismicity (TMH7 1981; SANS 2011). The magnitude of the seismicity in this region is however uncertain, with the current bridge loading...
code indicating a PGA of 0.1 g and a return period of 950 years (TMH7 1981). Current research estimates the region could be exposed to a PGA of up to 0.2 g with a return period of 475 years (Kijko et al 2003).

After close inspection it was noted that the interchange does not conform to modern-day recommended best practice for bridges located in seismic-prone areas. These factors include:

- Monolithically cast columns-to-deck connections
- In-plan curvature of the deck
- Single columns to wide deck connections
- Off-perpendicular abutment connections in terms of the longitudinal axis of the bridge.

In terms of general loading for which the bridge was designed, these design features and structural layouts are suitable and efficient solutions in non-seismic-prone regions. The bridge could, however, be susceptible to excessive displacements and secondary stress effects during an earthquake, which was not considered at design stage. Therefore, the uncertainty with regard to the seismic magnitude and the infringement of modern-day recommended best practice for bridges in seismic-prone areas necessitated an exploratory investigation to determine whether the bridge can sustain a moderate intensity earthquake.

**Bridge information**

An aerial photograph of the bridge leading from the northbound N1 highway onto the R300 is shown in Figure 1.

The bridge is composed of a post-tensioned continuous concrete box girder deck with a total length of 418 m, which is curved in-plan with an approximate radius of 245 m. The box girder has a width of 11.2 m with a total depth of 1.85 m from soffit to road surface. The deck is supported by nine single columns and two pairs of double columns at positions C5 and C9, as shown in Figure 2. The spacing between the columns ranges between 27.5 m and 38 m. Columns C5, C6 and C7 are supported on pile group foundations while the remaining columns are supported on pad footings. Columns C4 to C10 are monolithically cast into the bridge deck, while columns C2 and C12 support the deck via unidirectional plate bearings, allowing movement in the longitudinal direction of the bridge. Columns C3 and C11 support the deck via fixed plate bearings that allow only rotational freedom. Two side-by-side unidirectional plate bearings support the bridge deck at abutments C1 and C13, allowing additional movement in the longitudinal direction. The column lengths ranges from 11.5 m to 26.5 m.

**Seismic risk of the region**

The southwestern region of the Western Cape, hereafter referred to as the Cape Town region, is susceptible to the highest levels
of natural seismicity in South Africa (Visser & Kijko 2010). Several moderate intensity earthquakes occurred in this region. Visser and Kijko (2010) estimate an earthquake with a maximum magnitude between 6.0 and 6.87 with a return period of 475 years could be expected in the Cape Town region. Figure 3 graphically illustrates how the earthquake magnitude increases with a corresponding return period which was developed for a region within a 200 km radius from Cape Town. A worst-case scenario of a 6.87 magnitude earthquake would cause a level IX shaking intensity on the Mercalli Magnitude Intensity (MMI) scale. As the PGA value of an earthquake is site-specific, converting the magnitude of an earthquake to a PGA value is not very suitable without the known location of the epicentre, the recording location and the soil conditions. After evaluating the work of Kijko et al (2003), and Visser and Kijko (2010), a PGA of 0.2 g was established as appropriate for the worst-case earthquake with a 475 year return period in the Cape Town area.

Table 1 presents the estimated level of damage that could be experienced by building infrastructure in the Cape Town region as a result of a 6.87 magnitude earthquake (Visser & Kijko 2010). The three building categories represent approximately 70% of all urban structures in South Africa. The research shows that in the case of a seismic event of this magnitude, widespread damage could be experienced by infrastructure in this region.

**METHODOLOGY**

This section describes the importance of developing an initial finite element (FE) model, how the experimental tests were conducted to obtain the natural frequencies with its corresponding mode shapes, the detailed FE model, as well as the selection of earthquake acceleration time histories.

**Development of initial finite element model**

An initial FE model was developed using information from the ‘as built’ drawings to obtain the dynamic modal characteristics of the bridge. The modal behaviour of the initial FE model was reviewed to determine effective placement of accelerometers to capture the structure's dynamic response. Without this information, measurement devices could be placed at points along the bridge deck which would deliver no functional data.

**Ambient vibrational testing**

The total closure of the bridge was not permitted by the regulatory authority, the South African National Roads Agency SOC Limited (SANRAL), since it would cause severe disruptions to traffic on the N1, even during non-peak hour traffic. It was therefore important to find an experimental approach to determine the dynamic response of the bridge when one of the dual carriage lanes remain open. The physical ambient vibrational testing of the Stellenberg Interchange took into account the current state of the bridge, including all possible cracks and defects that had accumulated over time.
The bridge vibrations were recorded using accelerometers placed at a third spacing on each span. The accelerometer placement is presented by red indicators in Figure 2.

Seven tests were conducted over six of the twelve spans of the bridge. All tests conducted were performed at 20 minute time series recordings at a sampling rate of 1024 Hz. The time series data was resampled with a 66.67% overlap and passed through a low-pass filter to narrow the data to a band between 0 Hz and 13 Hz.

Enhanced frequency domain decomposition (EFFD) was used in the frequency domain, while stochastic subspace identification (SSI) was used in the time domain to produce the required modal parameters. Details regarding these methods can be obtained from publications by Bricker & Anderson (2006), National Instruments (2009) and Solms (2015). Figure 4 shows the spectral decomposition of the results obtained from the three recorded directions. The matching peaks of the data show the natural frequencies and are highlighted with vertical indication lines.

Table 2 shows the difference between the EFFD and SSI-UPC frequencies for the corresponding mode shapes. Although different methods are used to obtain the natural frequencies, the difference between the results of these methods is insignificant, indicating the accuracy of the experimental tests and the post-processing of the data. Figure 5 shows the experimental mode shapes with the natural frequencies using the SSI-UPC approach.

**Detailed finite element model development**

The initial FE model’s complexity was increased to provide an accurate representation of the actual structure, while maintaining computational efficiency. The initial FE model was modified using the experimental data to develop the final calibrated FE model. This was obtained by implementing appropriate refinements and adjusting appropriate parameters of the model. For a detailed description of the appropriate refinements on the parameters, the reader is referred to Solms (2015).

The experimental data enabled the calibration of the FE model, ensuring it produced accurate results. The final FE model was developed using SIMULA’s Abaqus version 6.13 FE analysis software. Since this was an exploratory investigation, the emphasis was to obtain a computationally efficient FE model which could accurately predict the structural response of the bridge. Therefore quadratic interpolated shear-flexible B32 beam elements (Timoshenko) incorporating full integration was used to model the structural elements of the bridge. These elements, with six degrees of freedom (DOF) per node, provided accurate force and displacement information while remaining computationally efficient. Although accurate stresses in the reinforcing steel and concrete could not be obtained from the beam elements, the use of three-dimensional solid elements to model the members in the global structure would be computationally expensive. The cross-sectional properties (area, moment of inertia and torsional moment of inertia) were calculated from the ‘as-built’ drawings and applied to the specific beam elements.

To obtain more accurate material behaviour, composite material properties were calculated for the beam elements. Since the mass and stiffness are key parameters when considering the modal behaviour of the structure, it is essential that the equivalent properties are accurately determined. The composite material properties were calculated by incorporating the area of steel and concrete in each section. The area of steel per metre for each cross-section was determined, whereafter the equivalent areas, densities and elasticity moduli were determined. The base material properties were obtained from the recommended values in SANS 10100-1 (SANS 2000).

Assumptions were made to effectively model the boundary conditions and element

---

**Table 2 Modal parameters from physical testing and finite element model**

<table>
<thead>
<tr>
<th>Mode</th>
<th>Physical modes (Hz)</th>
<th>FE frequencies (Hz)</th>
<th>Error (%)</th>
<th>MAC</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.855</td>
<td>2.824</td>
<td>-1.413</td>
<td>0.979</td>
</tr>
<tr>
<td>2</td>
<td>3.041</td>
<td>3.046</td>
<td>0.106</td>
<td>0.943</td>
</tr>
<tr>
<td>3</td>
<td>3.383</td>
<td>3.451</td>
<td>-1.438</td>
<td>0.905</td>
</tr>
<tr>
<td>4</td>
<td>3.368</td>
<td>3.871</td>
<td>-1.265</td>
<td>0.950</td>
</tr>
<tr>
<td>5</td>
<td>9.990</td>
<td>10.03</td>
<td>-3.258</td>
<td>0.870</td>
</tr>
</tbody>
</table>

**Figure 5 Modal comparison**

The bridge vibrations were recorded using accelerometers placed at a third spacing on each span. The accelerometer placement is presented by red indicators in Figure 2.

Seven tests were conducted over six of the twelve spans of the bridge. All tests conducted were performed at 20 minute time series recordings at a sampling rate of 1024 Hz. The time series data was resampled with a 66.67% overlap and passed through a low-pass filter to narrow the data to a band between 0 Hz and 13 Hz.

Enhanced frequency domain decomposition (EFFD) was used in the frequency domain, while stochastic subspace identification (SSI) was used in the time domain to produce the required modal parameters. Details regarding these methods can be obtained from publications by Bricker & Anderson (2006), National Instruments (2009) and Solms (2015). Figure 4 shows the spectral decomposition of the results obtained from the three recorded directions. The matching peaks of the data show the natural frequencies and are highlighted with vertical indication lines.

Table 2 shows the difference between the EFFD and SSI-UPC frequencies for the corresponding mode shapes. Although different methods are used to obtain the natural frequencies, the difference between the results of these methods is insignificant, indicating the accuracy of the experimental tests and the post-processing of the data. Figure 5 shows the experimental mode shapes with the natural frequencies using the SSI-UPC approach.

**Detailed finite element model development**

The initial FE model’s complexity was increased to provide an accurate representation of the actual structure, while maintaining computational efficiency. The initial FE model was modified using the experimental data to develop the final calibrated FE model. This was obtained by implementing appropriate refinements and adjusting appropriate parameters of the model. For a detailed description of the appropriate refinements on the parameters, the reader is referred to Solms (2015).

The experimental data enabled the calibration of the FE model, ensuring it produced accurate results. The final FE model was developed using SIMULA’s Abaqus version 6.13 FE analysis software. Since this was an exploratory investigation, the emphasis was to obtain a computationally efficient FE model which could accurately predict the structural response of the bridge. Therefore quadratic interpolated shear-flexible B32 beam elements (Timoshenko) incorporating full integration was used to model the structural elements of the bridge. These elements, with six degrees of freedom (DOF) per node, provided accurate force and displacement information while remaining computationally efficient. Although accurate stresses in the reinforcing steel and concrete could not be obtained from the beam elements, the use of three-dimensional solid elements to model the members in the global structure would be computationally expensive. The cross-sectional properties (area, moment of inertia and torsional moment of inertia) were calculated from the ‘as-built’ drawings and applied to the specific beam elements.

To obtain more accurate material behaviour, composite material properties were calculated for the beam elements. Since the mass and stiffness are key parameters when considering the modal behaviour of the structure, it is essential that the equivalent properties are accurately determined. The composite material properties were calculated by incorporating the area of steel and concrete in each section. The area of steel per metre for each cross-section was determined, whereafter the equivalent areas, densities and elasticity moduli were determined. The base material properties were obtained from the recommended values in SANS 10100-1 (SANS 2000).

Assumptions were made to effectively model the boundary conditions and element...
interactions of the bridge. All columns supported by pile groups and pad footings were modelled as fully fixed connections, thus fully restraining movement in all six degrees of freedom. The abutments (C1 and C13) on both ends connect to the deck via two side-by-side unidirectional bearings. The optimal solution to model these connections was to fully restrain the connection and release the rotational degree of freedom about the longitudinal axis of the bridge. The column to deck connections for the C2, C3, C11 and C12 columns were modelled using pin connections, thereby transferring all the translational degrees of freedom from one node to the other while releasing all rotational degrees of freedom.

Various model refinements and adjustments were applied to the model to calibrate the FE model to the experimental data. The parameters on which a sensitivity analysis were performed together with its influence is presented in Table 3.

The model calibration was performed by comparing the experimental and FE mode shapes with the corresponding frequencies. The five most prominent modes from the experimental data were used for comparison. For comparable model calibration, Magalhães et al (2008) suggest that the difference between the experimental and FE model frequencies be limited to 5%. Table 2 shows a comparison between the experimental and FE mode shapes with the corresponding natural frequencies. The largest error observed occurred at mode shape 5, which resulted in an error of 3.3%, indicating the accuracy of the FE model to predict a globally efficient and accurate model. Figure 5 shows the comparison between the experimental and FE model’s natural frequency and mode shapes for the first five modes.

It was also possible to extract the damping ratio from the experimental data, which was incorporated into the FE model. The complex interaction between all the elements that contribute to the damping of a large structure makes it impossible to calculate a theoretical damping ratio.

Earthquake simulations

The standard procedure when performing a time-history-based seismic analysis is to utilise the recorded ground motions of seven PGAs to determine the response of the structure. The selection of appropriate ground motions are based on the expected PGA, magnitude, distance and location of the epicentre, source mechanism and site soil conditions in the region. The time-histories are converted to spectral accelerations which are checked against the design response

<table>
<thead>
<tr>
<th>Table 3 Summary of FE model refinements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Refinement feature</td>
</tr>
<tr>
<td>Modelling of double columns</td>
</tr>
<tr>
<td>Effective pile foundations</td>
</tr>
<tr>
<td>Modelling of pad footings</td>
</tr>
<tr>
<td>Bearing conditions</td>
</tr>
<tr>
<td>Composite material properties</td>
</tr>
</tbody>
</table>

![Figure 6(a) Acceleration profile for P1524 north](image)

![Figure 6(b) Displacement profile for P1524 north](image)

![Figure 6(c) Acceleration profile for P1155 north](image)
spectra. If unavailable, these spectra should be developed for a range of soil conditions to evaluate the effect of potential amplification of motion due to the soil. Due to the unknown soil parameters and the large discrepancies in the recommended appropriate PGA for this region, the number of simulations required to incorporate all of the unknown variables would be very large. To reduce the number of simulations, some deviations from the typical approach were made. This was achieved by applying two PGAs with similar intensities, but resulting in very different ground displacement profiles. The PGAs were selected to obtain a maximum and minimum ground displacement for each of the acceleration versus time-histories considered, i.e. 0.05 g, 0.1 g, 0.15 g and 0.2 g. This approach thus allows for a lower and upper force band for each magnitude PGA considered. Figures 6(a) and 6(b) show the acceleration versus time-history and the displacement versus time-history for the 0.05 g case resulting in a minimum displacement profile, while Figures 6(c) and 6(d) cover the same for the maximum displacement profile. This study employed the recorded data from the Chi-Chi earthquake, which occurred in central Taiwan in 1999. The available comprehensive PGA and intensity ranges of the Chi-Chi earthquake governed the basis of this selection.

From Figures 6(b) and 6(d) we observe that the displacement profiles obtained from similar magnitude acceleration profiles yield significantly different maximum peak ground displacements. This would in turn lead to significant differences in the structural response of the structure. It was for this reason that the lower and upper bound approach was used in this study. For ease of reference, the earthquakes which cause the smaller displacement profile will be referred to as the ‘minimum intensity earthquake’, while the earthquakes causing the larger displacement profile will be referred to as the ‘maximum intensity profile’ for each PGA.

Three PGA datasets from the Chi-Chi earthquake represented the three cases considered for this study. The north–south and east–west acceleration time-histories were simultaneously applied to the column bases in both directions orthogonally to the vertical plane. These three acceleration cases are the following:

- A low-magnitude earthquake with a PGA of approximately 0.05 g which would have a higher probability of occurrence – the PGAs resulting in maximum and minimum ground displacement profiles applied to the structure were obtained from stations P1155 and P1524.
- An earthquake representing the recommendation prescribed by the TMH7 equating to a PGA of 0.1 g with a 10% probability of exceedance in a 100-year period – the PGAs resulting in maximum and minimum ground displacement profiles applied to the structure were obtained from stations P1468 and P1159.
- An earthquake with a PGA of approximately 0.2 g which could occur in the region with a 10% probability of exceedance in a 50-year period – this selection is based on the findings by Kijko et al. (2003) and the SABS 0160 (SABS 1989). The PGAs resulting in maximum and minimum ground displacement profiles applied to the structure were obtained from stations P1453 and P1288.

The PGAs applied to the structure could be oriented in endless arrangements and configurations. To limit the number of simulations, a selection of certain orientations and configurations were investigated. Various sensitivity analyses were performed to establish appropriate orientations. The aim was to obtain a worst-case orientation and execute all simulations for this condition. The worst-case orientation was obtained with the north and east PGAs directed normally and tangentially to the curvature at mid-span of the bridge.

RESULTS AND DISCUSSIONS

On completion of the simulations, the structural elements of the bridge that had indicated high forces were identified for closer inspection. An estimation of the capacity of each of these elements was determined and compared to the response from the FE model. Using these comparisons, an evaluation on the possibility of damage to each element could thus be concluded.

Once all the potential hazardous sections had been evaluated, a conclusion on the structural robustness when exposed to seismic excitation was made. The potential of damage or possible failure was evaluated for each magnitude earthquake applied to the FE model.

Various failure modes of the bridge were inspected to evaluate the possibility of damage to the structural elements of the bridge. All column bases were evaluated in terms of their shear and moment capacities when subjected to the applied earthquakes. The bearing connections were also evaluated in terms of their horizontal force capacity to determine the possibility of unseating of the deck. These aspects were assessed for varying magnitude earthquakes with PGAs between 0.05 g and 0.2 g. Since beam elements cannot effectively simulate failure of the reinforced concrete sections, estimations on the section capacities were made.

As the design calculations were not available, the estimated design section capacities were calculated from the reinforcement drawings. The estimated column capacities were calculated using SANS 10100-1 (SANS 2000), while the bearing capacities of the plate-bearing connections were obtained from the supplier.

To produce sensible histogram plots and comparable data for all the columns, the time-histories of the base shears and moments were reduced to a single value. Due to the occurrence of isolated individual peaks, the maximum value could be considered as an overestimation of the forces experienced by the structural elements, while the average would be an underestimation of the forces due to the oscillating nature of the seismic loading. The solution therefore was to determine an effective peak average force for each earthquake simulation, which was achieved using a peak-picking algorithm. Figure 7 presents the base shear time-history response of column C7 during the 0.1 g simulation, with the peak profile response, the average of the peak profile response, and the maximum and average of the base shear force response. The peak average values were used for comparative purposes with the codified capacities.
results and observations for the 0.05 g earthquakes

The 0.05 g earthquake responses produced results which are no cause for concern, as the responses are significantly below the estimated design capacities of the various sections. For this reason these results and discussions are therefore omitted.

Results and observations for the 0.1 g earthquakes

A magnitude of 0.1 g was selected as it conforms to the requirements provided in TMH7 for the region in which the interchange is located. The results and observations made are with respect to the shear forces and bending moments in the columns.

Figure 8 presents the column base moments for the maximum, minimum and design capacities when subjected to an earthquake with an intensity of 0.1 g.

From Figure 8 it is observed that the base moment capacity of only one column base moment capacity, i.e. C8, is exceeded by 18%, and thus could
experience significant damage. The base moments of columns C4, C9 and C10 are, however, within approximately 90% of its design capacity. Therefore, if column C8 attains its maximum design capacity, it would transfer the remaining force to the other columns and could result in columns C4, C9 and C10 also attaining its design capacity. If this situation arises it could result in significant damage to these columns.

Figure 9 presents the column base shear forces for the maximum, minimum and design capacities when subjected to an earthquake with an intensity of 0.1 g. From Figure 9 it is observed that the estimated shear capacities of the columns were not exceeded during the 0.1 g earthquake simulations. The maximum peak base shear force only attains 42% of the worst-case column’s (C10) estimated shear capacity. The shear resistance capacity of the columns could, however, be reduced if the bending moment capacity is exceeded, thereby increasing the risk of damage to the column base. This would, however, have a very low probability of occurring for the 0.1 g case due to the high estimated shear capacities.

Results and observations for the 0.2 g earthquakes
Current research suggests that an earthquake with a PGA of up to 0.2 g, with
a return period of 475 years, could be expected in the region where the Stellenberg Interchange is located. This PGA of 0.2 g also correlates with the design PGA of the previous building infrastructure design code, SABS 0160 (SABS 1989). The results and observations made are with respect to the shear forces and bending moments in the columns. Figure 10 presents the column base moments for the maximum, minimum and design capacities when subjected to an earthquake with an intensity of 0.2 g.

The results shown indicate that the moment capacity of five columns (C4, C8, C9.1, C9.2 and C10) is significantly exceeded, while the moment capacity of two columns (C5.1 and C5.2) is marginally exceeded. The bending moment capacity of the worst-case column (C8) was exceeded by 62% of the estimated column capacity during a third of the 90 seconds maximum intensity simulation, indicating a significant likelihood of severe damage, which could result in failure of the column.

The two pairs of double columns at C5 and C9 provide a large portion of the transverse stiffness of the structure. Therefore, damage to either of the double column pairings could potentially lead to the collapse of the bridge if severe damage occurs in the early stages of such a seismic event. This risk could be increased if the damage to the column bases significantly reduces the shear capacity.

The shear response of the column bases, however, did not indicate a high risk of damage, even when subjected to the maximum intensity of the 0.2 g earthquake, as shown in Figure 11.

The study also investigated the horizontal force capacity of the bearing assemblies used throughout the structure. None of the bearings indicated cause for concern, as the maximum intensity of the 0.2 g earthquake only attained 75% of the bearings’ horizontal force capacity.

CONCLUSIONS

The following observations can be made for the various peak ground accelerations applied to the calibrated FE model:

0.05 g For an earthquake with a lower return period than the current suggested design loading with a PGA of 0.05 g, the bridge should not experience any significant damage or failure since the worst-case column (C10) only experiences a bending moment equating to 5.2% of the calculated capacity of the column.

0.1 g For the PGA of 0.1 g suggested by current seismic SANS 10160-4 (SANS 2011) and bridge design codes (TMH7 1981), the bridge would be exposed to a low to moderate probability of damage. This damage should be localised to singular worst-case sections of the bridge, barring complete failure of the column does not transfer excessive additional load to remaining columns.

0.2 g In terms of current research on seismicity in the area, suggesting an applicable PGA of 0.2 g, the bridge would be exposed to a high probability of damage and even a risk collapse. It should be noted that this study was an exploratory investigation into the seismic response of the Stellenberg Interchange. Further investigation is required to assess the specific level of damage the bridge would undergo during certain seismic events. To define this level of damage, a three-dimensional solid element FE model utilising nonlinear materials properties would be required. Such a model would be able to predict the progressive failure of the bridge in terms of the structure’s performance when some of the sections are damaged, as well as highlight the load combinations and progressive loss in section capacity during a seismic event. It is also important to highlight the relative recent implementation of seismic design criteria in South Africa, and the average age of infrastructure in the Cape Town region. This suggests that widespread damage to buildings and other infrastructure in the Cape Town region would be probable if an equivalent 0.2 g seismic event occurs in the region.

ACKNOWLEDGEMENTS

The financial assistance of the National Research Foundation (NRF) towards this research is hereby acknowledged. Opinions expressed and conclusions arrived at, are those of the authors and are not necessarily to be attributed to the NRF.

This paper is furthermore presented with the approval of the South African National Roads Agency SOC Limited. The content of

![Figure 11 Summary of base shear force for all columns during the Chi-Chi 0.2 g earthquake](image-url)
the paper reflects the views of the authors, who are responsible for the facts and accuracy of the data presented herein. The content does not necessarily reflect the official view or policies of the South African National Roads Agency SOC Limited.

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.

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

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

CLASSIFICATION OF ARTICLES CONSIDERED FOR PUBLICATION

- Technical papers are well-researched, in-depth, fully referenced technical articles not exceeding 6 000 words in length (excluding tables, illustrations and the list of references).
- 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.
- 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.
- 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.
- 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.
- 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.
- 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.
- 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.

POLICY REGARDING LANGUAGE AND ORIGINALITY OF SUBMITTED ARTICLES

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

SUBMISSION PROCEDURES AND REQUIRED FORMAT

- Electronic submission: Manuscripts should be e-mailed to the editor at veredene@saice.org.za. File sizes should not exceed 4 MB per e-mail – figures may for example be sent one by one or in groups not larger than 4 MB. Manuscripts should not be sent in PDF format as this precludes reviewing of papers per track changes.
- Format: Manuscripts should be prepared in MS Word and presented in double line spacing, single column layout with 25 mm wide margins. Line numbers must be applied to the whole document. All pages should bear the authors’ names and be numbered at the bottom of the page. With the exception of tables and figures (see below) the document should be typed in Times New Roman 12 pt font. Contributions should be accompanied by an abstract of not more than 200 words.
- First page: The first page of the manuscript should include the title of the paper, the number of words of the main text (i.e. excluding figures, tables and the list of references), the initials and surnames of the authors, professional status (if applicable), SAICE affiliation (Member, Fellow, Visitor, etc), telephone numbers (landline and mobile), 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 also given in the text, nor contain material that would be better presented graphically. Tabular matter should be as simple as possible, with brief column headings and a minimum number of columns.
- Mathematical expressions and presentation of symbols:
  - Equations should be presented in a clear form which can easily be read by non-mathematicians. Each equation should appear on a separate line and should be numbered consecutively.
  - Symbols should preferably reflect those used in Microsoft Word Equation Editor or Mathtype, or should be typed using the Times New Roman symbol set.
- Variables in equations (x, y, z, etc) as well as lower case Greek letters should be presented in italics. Numbers (digits), upper case Greek letters, symbols of metric measurement units (m for metres, s for seconds, etc) and mathematical/trigonometrical functions (such as sin, cos and tan) are not written in italics, but in upright type (Roman).
- Variables and symbols used in the body of the text should match the format used in the equations, i.e. upright or italics, whichever is applicable.
- Metric measurement abbreviations/units should conform to international usage – the SI system of units should be used.
- Decimal commas may be used, but decimal points are preferred.
- Symbols should preferably be defined in the text, but if this is not feasible, a list of notations may be provided for inclusion at the end of the paper.
- Headings: Sections and paragraphs should not be numbered. The following hierarchy of headings should be followed:
  - HEADING OF MAIN SECTION
  - Heading of subsection
  - Heading of sub-subsection
- References: References should follow the Harvard system. The format of text citations should be as follows: "Jones (1999) discovered that..." or "recent results (Brown & Carter 1985; Green et al 1999) indicated that...".
- References cited in the text should be listed in alphabetical order at the end of the paper. References by the same author should be in chronological order. The following are examples of a journal article, a book and a conference paper:
- Papers published previously in the journal of the South African Institution of Civil Engineering should be cited if applicable.
- Footnotes, trade names, acronyms, abbreviations: These should be avoided. If acronyms are used, they should be defined when they first appear in the text. Do not use full stops after abbreviations or acronyms.
- Return of amended papers: Papers requiring amendments will be accepted up to six months after the referee reports have been sent to authors, after which the paper will be withdrawn from the system.

FINAL ARTICLE

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