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

\[ k_x \frac{\partial^2 h}{\partial x^2} + k_z \frac{\partial^2 h}{\partial z^2} = 0 \]
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Delay and queue lengths at stop/go control during half-width construction

L. Venter, C. J. Bester

It is common practice in South Africa that road reseal projects, road reconstruction projects or road upgrade projects on two-lane, two-way roads are constructed in half-widths, resulting in a situation where traffic on the remaining roadway is reduced to one-way operation.

There is currently no method for determining a suitable length of work zone for half-width construction based on traffic volumes, and also no method for determining waiting time for the vehicle in the front of the stationary queue at the STOP/GO control, and of the back-of-queue position at such a work zone.

An Excel-based calculation sheet for determining the back-of-queue position and the maximum waiting time, was developed. This was used to develop design tables, graphs and equations that can be used by designers and contractors to estimate the back-of-queue position and maximum waiting time.

Based on the conclusions of this paper and the fact that this project was largely based on literature studies of traffic conditions not related to work zones for half-width construction, the main recommendation of the study is that all the input parameters which were investigated need to be verified and calibrated specifically for work zones for half-width construction.

INTRODUCTION

The definition of a “work zone for half-width construction” (work zone), in the context of this paper, shall be a “workroad site for a rural two-way, two-lane road that is partially closed to traffic, where the remaining roadway is reduced to less than two lanes in width, for whatever reason, and where traffic is controlled manually at either end of such workroad site by means of a STOP/GO operation or temporary traffic signals to allow one-way traffic only, alternately in each direction, on the remaining roadway that is open to traffic”. The impacts of the work zone for half-width construction on traffic operations will, however, extend beyond the work zone itself, i.e. into the approaches of the work zone for half-width construction.

Background

Waiting in a stationary queue at a work zone for half-width construction could be a frustration to motorists. In addition, motorists at the back of the queue cannot see the “maximum waiting time” displayed on the information traffic sign at the STOP/GO position and have no idea of how long they would have to wait.

During the design phase of a roads project, the engineer should devise a traffic management plan or traffic accommodation plan that would, inter alia, optimise site efficiency, traffic flow and all aspects of safety. Although it is not possible to predetermine how all construction sites will be managed during construction, because there are too many variables, it is considered very important that the engineer will plan, and work, in a systematic manner and in standardised steps to achieve the goals of the traffic accommodation plan, namely to optimise site efficiency, traffic flow and all aspects of safety. At a more detailed level, during the design phase, the engineer should identify the components of the construction site that would influence the traffic accommodation plan.

It is common practice in South Africa that road reseal projects, road reconstruction projects or road upgrade projects are constructed in half-widths. Whether the traffic, for a construction project, can be controlled by means of a STOP/GO operation or temporary traffic signals will depend largely on the environment, volume and speed of traffic, and on the length of the section of roadway subject to one-way control.

The length of the work zone for half-width construction is dependent on the time that a motorist would have to wait in the front of a stationary queue at a work zone for half-width construction, which in turn, is largely dependent on the volume and composition of traffic and the speed at which the vehicles can travel through the said work zone. It is therefore clear that the length of the work zone for half-width construction will differ from project to project, since the topography of the land through which the
road traverses, and the volume and composition of traffic will determine the speed at which vehicles can travel through that work zone.

The COLTO Standard Specifications for Road and Bridge Works for State Road Authorities (COLTO 1998, pp 1500–5) Section 1513 (Accommodation of traffic where the road is constructed in half-widths), prescribe that:

a. The maximum length of half-width construction must be prescribed in the project specifications or on the drawings.
b. The maximum length of half-width construction shall not exceed 4 km, unless otherwise specified.
c. The minimum space between half-width construction sections shall be at least 2 km.

There is currently no method for determining a suitable length of work zone for half-width construction based on traffic volumes, and also no method for determining waiting time for the vehicle in the front of the stationary queue at the STOP/GO control, and of the back-of-queue position at such a work zone. In practice, practitioners easily use these prescribed values of 4 km and 2 km as a standard, irrespective of the site-specific or project conditions.

A safety issue related to work zones is that the first warning sign, in a series of signs for traffic accommodation for most traffic accommodation typical drawings, is the temporary “Roadworks ahead” warning sign (TW336), with a temporary information sign (TIN 11.3) that shows the distance to the work zone, i.e. the distance to the stop line at the STOP/GO position. These distances are normally shown as 600 m and 1 km. The combination of the temporary “Roadworks ahead” warning sign (TW336) and the temporary information sign (TIN 11.3) does therefore not make provision for the distance to the back-of-queue position at a work zone.

Back-of-queue positions can easily be longer than 600 m, or even 1 km, which means that arriving motorists are not alerted to the danger of possible stationary vehicles at the back-of-queue position, which could lead to back-of-queue collisions.

Previous publications

Previous international publications were mostly concerned with delay (total queue delay or average delay per vehicle) prediction and queue length (in vehicles) under one-way traffic control, but not specifically waiting time for the vehicle in the front of the stationary queue and back-of-queue position (in metres).

Cassidy and Son (1994) describe the adaptation and application of queuing models, originally derived for intersections controlled by vehicle-actuated traffic signals, to estimate delay at two-lane highway work zones. The models estimate expected delay as a function of directional traffic demand rates, work zone physical length and observed traffic measures.

Washburn et al (2008) provided a calculation procedure for estimating the capacity, delays and queue lengths of two-lane, two-way work zones with flagging control. This calculation procedure utilises a combination of standard signalled intersection analysis equations, as well as some custom models developed from simulation data.

Alternative methodologies

The Highway Capacity Manual (HCM) (TRB 2000) procedure determines delay per vehicle (in seconds per vehicle) and uses theoretical queue lengths to determine average back-of-queue (in vehicles), i.e. it does not use vehicle length and inter-vehicle space to determine back-of-queue position.

SIDRA®1 intersection analysis programme, which is based on the HCM, calculates delay (total queue delay or average delay per vehicle) and average back-of-queue (in vehicles or metres).

These methodologies require cycle lengths and effective green time as input parameters, and do not take into account that the cycle length could be dependent on, amongst others, the length of work zone, the speed through the work zone or the dissipation time of the queue on the opposite side of a work zone for half-width construction, i.e. it makes no provision for one-way traffic only, alternately in each direction.

Objectives

The objectives of this paper, to determine the waiting time for the vehicle in the front of the stationary queue and back-of-queue position (in metres) at the STOP/GO control for a work zone for half-width construction, are as follows:

The first objective of this paper was to determine and investigate the factors that influence the waiting time for the vehicle in the front of the stationary queue at the STOP/GO control for a work zone for half-width construction.

The second objective of this paper was to develop equations to calculate the waiting time for the vehicle in the front of the stationary queue and the back-of-queue position.

The third objective was to use the aforementioned equations to develop design tables and graphs that could be used by designers and contractors for estimating the maximum waiting time for the vehicle in the front of the stationary queue and back-of-queue position.

Methodology

The following methodology was followed to achieve the aforementioned objectives:

a. Determine the variables that influence the waiting time for the vehicle in the front of the stationary queue at the STOP/GO control for a work zone for half-width construction and back-of-queue position.

b. Investigate the variables that influence the waiting time for the vehicle in the front of the stationary queue at the STOP/GO control for a work zone for half-width construction and back-of-queue position by means of a literature study.

c. Develop an Excel-based calculation sheet for determining the back-of-queue position and the maximum waiting time for the vehicle in the front of the stationary queue.

d. Develop design tables and graphs that could be used by designers and contractors for estimating the back-of-queue position and maximum waiting time for the vehicle in the front of the stationary queue, at a work zone for half-width construction.

Determining the sequence of a typical cycle for vehicles through a half-width construction work zone will help to determine the factors that influence the waiting time for the vehicle in the front of the stationary queue at the STOP/GO control and the back-of-queue position.

Waiting time for the vehicle in the front of the stationary queue

Let us assume that a vehicle arrives from direction “d” (the direction of analysis) at the stop line of a STOP/GO control for half-width construction. The time that this vehicle will have to wait at the stop line will depend on the following factors:

a. The time (t1 in minutes) that it takes for the last vehicle that passed the stop line (i.e. the vehicle that was just in front of the vehicle that had to stop), in direction “d”, to drive through the work zone.

b. Once that vehicle has driven through the work zone and passes the point at the stop line at the far-side of the work zone, a few seconds may laps before the lane will be opened to the opposing traffic.

It can be written as:

\[
t_1 = \frac{L \times 60}{v_1}
\]

where:

- \(L\) is the length of the work zone (in kilometres)
- \(v_1\) is the average speed (in kilometres per hour) of the vehicle through the work zone.
This “operator lost time” ($t_{oHV}$ in seconds) is attributed to the operator confirming that the last vehicle has passed the point at the stop line. The operator will then switch the traffic signal to green (or turn the STOP/GO sign) to indicate that it is safe for the vehicles to proceed.

c. “Start-up lost time” ($t_{oHV}$ in seconds), which is the additional time consumed by the first few vehicles in the queue above and beyond the saturation headway, must also be considered. Start-up lost time accounts for the additional reaction time and vehicle acceleration time after the operator switched the traffic signal to green (or turned the STOP/GO sign).

d. Following vehicles will then move past the stop line at a steady speed until the last vehicle in the original queue has passed the stop line. The saturation headway ($t_{ov}$ in seconds) for these vehicles will be relatively constant and the total time ($t_{ov}$ in minutes) that it takes for the queued vehicles to pass the stop line is equal to the saturation headway multiplied by the number of queued vehicles ($n_o$), in direction “o” (the opposing direction of analysis) that passed the stop line.

It can be written as:

$$t_{ov} = \frac{n_o}{60}$$

(2)

Substituting Equations 1 to 3 into Equation 4, the total waiting time ($T_d$) can be rewritten as:

$$T_d = t_1 + \frac{t_{1HV}}{60} + \frac{t_{2HV}}{60} + \frac{t_{ov}}{60} + \frac{t_{12}}{60} + \frac{t_{oHV}}{60}$$

$$+ \frac{t_{oHV}}{v_1} + \frac{t_{oHV}}{v_2} + \frac{n_o}{60}$$

(5)

f. The time ($t_2$) in minutes that it takes for the last vehicle that passed the stop line, in direction “o” (the opposing direction of analysis), to drive through the work zone.

It can be written as:

$$t_2 = \frac{L}{v_2}$$

(3)

g. Once the vehicle has driven through the work zone and passes the point at the stop line at the near-side of the work zone, a few seconds may laps before the lane will be opened to the opposing traffic. This “operator lost time” ($t_{oHV}$ in seconds) is attributed to the operator confirming that the last vehicle has passed the point at the stop line and that it is safe to open the lane to the opposing traffic. The total waiting time ($T_{dHV}$ in minutes) for the vehicle, in direction “d” (the direction of analysis), in the front of the stationary queue, at the STOP/GO control for a work zone for half-width construction, can be written as:

$$T_{dHV} = t_1 + \frac{t_{1HV}}{60} + \frac{t_{2HV}}{60} + \frac{t_{oh}}{60} + \frac{t_2}{60} + \frac{t_{12HV}}{60}$$

(4)

By substituting Equations 1 to 3 into Equation 4, the total waiting time ($T_{dHV}$) can be rewritten as:

$$T_{dHV} = \left( \frac{L}{v_1} \right) + \frac{t_{1HV}}{60} + \frac{t_{2HV}}{60} + \frac{t_{oh}}{60} + \frac{t_2}{60} + \frac{t_{12HV}}{60} + \frac{t_{oHVHV}}{v_1} + \frac{t_{oHVHV}}{v_2} + \frac{n_o}{60}$$

(5)

Assuming that the speeds of the last vehicles through the work zone, in both directions, are the same ($v_1 = v_2$), and that the lost time, at both ends, for the operator to switch the traffic signal to green (or turn the STOP/GO sign), is the same ($t_{1HV} = t_{12HV}$), Equation 5 can be rewritten as:

$$T_{dHV} = 2 \times \left( \frac{L}{v_1} \right) + 2 \times \frac{t_{1HV}}{60} + \frac{t_2}{60} + \frac{t_{oHVHV}}{60} + \frac{t_{oHVHV}}{60} + \frac{n_o}{60}$$

(6)

The number of queued vehicles ($n_o$) in direction “o” (the opposing direction of analysis) that arrived to form a queue at the far-side of the work zone is dependent on the uniform arrival rate ($Q_o$ in vehicles per hour) of vehicles in direction “o” during the time $T_o$ (the total waiting time in minutes for the vehicle, in direction “o”, in the front of the stationary queue at the STOP/GO control for a work zone for half-width construction) and can be written as:

$$n_o = T_o \times \frac{Q_o}{60}$$

(7)

The uniform arrival rate ($Q_o$ in vehicles per hour) of vehicles in direction “o” is equal to the hourly two-way demand ($D_o$ expressed as a decimal) of the hourly two-way volume on the road, and can be written as:

$$Q_o = V \times D_o$$

(8)

The design hourly two-way volume should typically be chosen to incorporate seasonal or monthly fluctuations in traffic demand.

A conversion of the design hourly two-way volume in terms of passenger car equivalents (pce) can be computed using a heavy vehicle factor ($f_{HV}$). The heavy vehicle factor accounts for the additional space occupied by heavy vehicles in the traffic stream and for the difference in operating capabilities of heavy vehicles compared to passenger cars.

The design hourly two-way volume ($v$) can be written as:

$$v = \frac{V}{PHF}$$

(9)

The heavy vehicle factor ($f_{HV}$) adapted from the Highway Capacity Manual 2000 (HCM 2000) (TRB 2000, pp 22–7) is:

$$f_{HV} = \frac{1}{1 + PHF \times (E_{HV} - 1)}$$

(10)

where:

$PHF = \frac{P_{HV}}{P_{HV}} = \frac{\text{proportion of heavy vehicles in the traffic stream, expressed as a decimal}}{\text{the passenger car equivalent for heavy vehicles – the number of passenger cars that is displaced by a single heavy vehicle of particular type under specified roadway, traffic and control conditions}}$ (TRB 2000, pp 5–11).

By substituting Equations 7 and 8 into Equation 6, and by substituting the hourly two-way volume ($V$) with the design hourly two-way volume of Equation 10 ($V/(PHF \times f_{HV})$), the total waiting time ($T_{dHV}$ in minutes) can be rewritten as:

$$T_{dHV} = 2 \times \left( \frac{L}{v_1} \right) + 2 \times \frac{t_{1HV}}{60} + \frac{t_2}{60} + \frac{t_{oHVHV}}{60} + \frac{t_{oHVHV}}{60} + \frac{n_o}{60}$$

(12)

By including Equation 11, the heavy vehicle factor, in Equation 12, and expressing all the parameters in terms of the opposite direction “o”, the total waiting time ($T_{dHV}$) is rewritten as:

$$T_{dHV} = \frac{L}{v_1} \times \frac{t_{1HV}}{60} + \frac{t_2}{60} + \frac{t_{oHVHV}}{60} \times \frac{V \times D_o}{(60 \times PHF \times f_{HV})} + \frac{t_{oHVHV}}{60}$$

(13)

Similarly, it can be shown that the total waiting time ($T_{dHV}$) for the vehicle in direction
that can pass the stop line, for each direction
(S in veh/s) is less than the maximum flow rate
(Q in veh) that can be reached where the degree of
saturation is reached. This equilibrium can
be solved by means of an iterative
process, whereby green time (G) and
effective red time (E) to solve the total waiting time (T)
in Equation 13 and Equation 14, will
have to be repeated and checked against the
degree of saturation, until an equilibrium is
reached where the degree of saturation in
each directions is less than one.

**Back-of-queue position**
The maximum stationary queue length (Q_{SM}
in veh) is equal to the traffic demand (uniform
arrival rate, Q in veh/s) that arrives during the
effective red time for each direction, where
the effective red time for each approach is
equal to the total waiting time (T_d and T_o in
min) in Equation 13 and Equation 14.

The maximum stationary queue length
for each direction can be written as:

\[ Q_{dSM} = Q_d \times T_d \times 60 \]  \hspace{1cm} (22)

and

\[ Q_{oSM} = Q_o \times T_o \times 60 \]  \hspace{1cm} (23)

As vehicles in the front of the stationary
queue start to move past the stop line, the
vehicles at the back of the queue are still sta-
tionary, i.e. the back of the stationary queue
is at the same position, although there
are now fewer cars in the actual stationary
queue. At the same time, vehicles are arriving
(at a uniform arrival rate, Q in veh/s)
the stop line in Equations 20 and 21 can be
substituted with Equations 7 to 11, in both
directions “d” and “o”, for design volumes.

The iteration process, whereby green time
(t_g and t_{dg}) is added, in both directions “d” and
“o”, can be rewritten from Equation 13 and Equation 14.

The total cycle length (C in seconds) for each
approach is equal to the effective red time
(R in seconds) plus the effective green time
(G in seconds). The effective red time for each
approach is equal to the total waiting time (T_d and T_o in
Equation 13 and Equation 14; and the effective green time for each
approach is equal to the total time that it
takes for the queued vehicles to pass the stop
line (see Equation 2) plus the added green
time. Therefore:

\[ C_d = T_d + (h_{ds} \times n_d) + t_{dg} \]  \hspace{1cm} (18)

\[ C_o = T_o + (h_{os} \times n_o) + t_{og} \]  \hspace{1cm} (19)

Equation 16 can be rewritten as:

\[ X_d = \frac{Q_d \times (T_d + (h_{ds} \times n_d) + t_{dg})}{(h_{ds} \times n_d) + t_{dg}} \times S_d \times 1 \]  \hspace{1cm} (20)

Similarly, Equation 17 can be rewritten as:

\[ X_o = \frac{Q_o \times (T_o + (h_{os} \times n_o) + t_{og})}{(h_{os} \times n_o) + t_{og}} \times S_o \times 1 \]  \hspace{1cm} (21)

The speed of the shock wave can be written as:

\[ u_w = \frac{Q_2 - Q_1}{K_2 - K_1} \]  \hspace{1cm} (25)

where:

\[ Q = \text{flow (veh/h)} \]

\[ U_l = \text{macroscopic speed (km/h)} \]

\[ K = \text{density (veh/km)} \]
where: 

\[ u_{ws} = \text{shock wave speed (km/h)} \]

\[ Q_2 = \text{flow of dispersing vehicles (veh/h)}, \text{which equals the saturation flow rate (S in veh/h)} \]

\[ Q_1 = \text{flow of stationary vehicles (veh/h), which is equal to zero} \]

\[ K_2 = \text{density of the dispersing vehicles (veh/km)} \]

\[ K_1 = \text{density of stationary vehicles (veh/km).} \]

The density of stationary vehicles (veh/km) will always be more than the density of the dispersing vehicles (veh/km), the shock wave speed will be a negative value, i.e. the shock wave travels backwards.

The density of the dispersing vehicles (\( K_2 \) in veh/km) is equal to the saturation flow rate (S in veh/h) divided by the speed of the dispersing vehicles, which is taken as the average speed (\( v_1 \) in km/h) of the last vehicle through the work zone. 

\[ K_2 = \frac{S}{v_1} \] (26)

The density of stationary vehicles (\( K_1 \) in veh/km) is equal to the inverse of the average vehicle length factor (\( f_{VL} \) in m/veh) that takes the vehicle composition, the vehicle length (\( L_i \) in metres) and the inter-vehicle space (\( s_i \) in metres) into account.

The average vehicle length factor (\( f_{VL} \) in m/veh) can be written as:

\[ f_{VL} = \sum_i (P_i \times (L_i + s_i)) \] (27)

where:

\( P_i = \text{proportion of type } i \text{ vehicles in the traffic stream, expressed as a decimal} \)

\( L_i = \text{the average length of type } i \text{ vehicles in the stream (m)} \)

\( s_i = \text{average inter-vehicle spacing (m).} \)

The density of stationary vehicles (\( K_1 \) in veh/km) can be written as:

\[ K_1 = \frac{1000}{f_{VL}} \] (28)

By substituting Equations 26, 27 and 28 into Equation 25, and dividing by the average vehicle length factor (\( f_{VL} \) in m/veh), the speed of the shock wave (\( u_{ws} \) in veh/s) can be written as:

\[ u_{ws} = \frac{S}{\left( \frac{1000}{f_{VL}} - f_{VL} \right) \times f_{VL} \times 3.6} \] (29)

By further manipulation of Equation 29 and multiplying it by minus one (-1) to convert the speed of the shock wave to a positive value, the speed of the shock wave (\( u_{ws} \) in veh/s) can be written as:

\[ u_{ws} = \frac{1}{\left( \frac{1000}{S} - \frac{f_{VL}}{v_1} \right) \times 3.6} \] (30)

The maximum back-of-queue position (\( Q_M \) in veh) is equal to the maximum stationary queue length (\( Q_{SM} \) in veh), as shown in Equations 22 and 23, plus the traffic demand (uniform arrival rate \( Q \) in veh/s) that arrives during the additional time (\( dT \) in min) needed for the queue to disperse, i.e. the additional time for the shock wave to reach the back of the queue. This relationship can be written as:

\[ (Q \times T) + (Q \times dT) = u_{ws} \times dT \] (31)

The additional time (\( dT \) in min) for the queue to disperse can be found by solving Equation 31:

\[ dT = \frac{T}{u_{ws} - 1} \] (32)

The maximum back-of-queue position (\( Q_M \) in veh) for each direction can be written as:

\[ Q_{dM} = (Q_d \times T_d) + \frac{Q_d \times T_d}{u_{ws} - 1} \] (33)

and

\[ Q_{oM} = (Q_o \times T_o) + \frac{Q_o \times T_o}{u_{ws} - 1} \] (34)

The maximum back-of-queue position (\( Q_L \) in metres) can be calculated by multiplying the maximum back-of-queue position (\( Q_{M} \) in veh) by the average vehicle length factor (\( f_{VL} \) in m/veh), from Equation 27, which takes the vehicle composition, the vehicle length (\( L_i \) in metres) and the inter-vehicle space (\( s_i \) in metres) into account.

<table>
<thead>
<tr>
<th>Table 1 Ranges of variables</th>
</tr>
</thead>
<tbody>
<tr>
<td>Variable</td>
</tr>
<tr>
<td>Two-way volume (veh/h)</td>
</tr>
<tr>
<td>Heavy vehicles (%)</td>
</tr>
<tr>
<td>Directional split (%)</td>
</tr>
<tr>
<td>Work zone length (km)</td>
</tr>
<tr>
<td>Speed (km/h)</td>
</tr>
</tbody>
</table>

Since there would be one less inter-vehicle space than vehicles in the queue, one average inter-vehicle space must be subtracted from the equation when it is converted from maximum back-of-queue position (\( Q_{M} \) in veh) to maximum back-of-queue position (\( Q_{M} \) in metres).

The maximum back-of-queue position (\( Q_1 \) in metres) can be written as:

\[ Q_L = (f_{VL} \times Q_{dM}) - s_i \] (35)

By substituting Equations 33 and 34 into Equation 35, the maximum back-of-queue position, in metres, for each direction can be written as:

\[ Q_{dL} = f_{VL} \times \left( Q_d \times T_d + \frac{Q_d \times T_d}{u_{ws} - 1} \right) - s_i \] (36)

and

\[ Q_{oL} = f_{VL} \times \left( Q_o \times T_o + \frac{Q_o \times T_o}{u_{ws} - 1} \right) - s_i \] (37)

LITERATURE STUDY OF THE VARIABLES THAT INFLUENCE WAITING TIME AND BACK-OF-QUEUE POSITION

Two types of variables were identified in the preceding section. Firstly there are those that can be directly measured or derived from the specifications of the project, such as traffic volumes, composition and directional split, length of work zone, and speed of the last vehicle through the work zone. For the development of the design tables these will be the independent variables, and the ranges in which they will be used are shown in Table 1.

Secondly there are the variables that will be fixed for the development of the design tables. Their values are discussed below.

**Operator lost time**

The operator lost time (\( t_{L1} \) and \( t_{L2} \)) is attributed to the operator confirming that the last vehicle through the work zone has passed the point at the stop line and that it is safe to open the lane to the opposing traffic. A
nominal 12 seconds for operator lost time was assumed, based on observations at various work zones for half-width construction sites in the Western Cape.

**Start-up lost time**

Start-up lost time ($t_{L2}$) is the additional time consumed by the first few vehicles in the queue above and beyond the saturation headway. Bester and Varndell (2002) quoted from previous studies that start-up lost time ranged from 0.75 seconds to 3.04 seconds. It was assumed, for the calculation of waiting time and back-of-queue position, that start-up lost time at a work zone shall be a maximum of 3.0 seconds.

**Saturation headway**

Saturation flow rate, according to the HCM 2000 (TRB 2000, pp 7–10), is defined as "the flow rate per lane at which vehicles can pass through a signalized intersection". When the saturation flow rate is determined from time measurements taken in the field, it is computed by Equation 38:

$$ S = \frac{3 600}{h_s} $$  \hspace{1cm} (38)

where:

- $S$ = saturation flow or departure rate (veh/h)
- $h_s$ = saturation headway (s/veh).

The saturation headway is described in the HCM 2000 (TRB 2000, pp 5–14) as "the average headway between vehicles occurring after the fourth vehicle in the queue and continuing until the last vehicle in the initial queue clears the intersection".

To determine saturation headway, Equation 38 can be rewritten as:

$$ h_s = \frac{3 600}{S} $$  \hspace{1cm} (39)

Since very little information on saturation flow rates for South African conditions could be obtained, which was affirmed by Bester and Meyers (2007), the saturation flow rate can be determined based on the estimation procedure described in the HCM 2000 (TRB 2000, pp 16-9 to 16-13) for signalised intersections, whereby a base saturation flow rate ($s_b$ in pc/h), under ideal conditions, is used and adjusted for various factors that can influence traffic behaviour and in turn the saturation flow rate. Ideal conditions are described in the HCM 2000 (TRB 2000), but include some assumptions, namely 3.6 metre lane widths, no heavy vehicles in the traffic stream, flat gradient, etc.

The estimated saturation flow rate is calculated by adjusting Equation 16-4 of the HCM 2000 (TRB 2000, p 16-9) as follows:

$$ S = s_b \times f_w \times f_{HV} \times f_G $$  \hspace{1cm} (40)

where:

- $S$ = saturation flow or departure rate (pc/h)
- $s_b$ = base saturation flow rate (pc/h)
- $f_w$ = lane width factor
- $f_{HV}$ = heavy vehicle factor
- $f_G$ = gradient factor.

By substituting Equation 40 into Equation 39 the saturation headway ($h_s$ in s/veh) can be rewritten as:

$$ h_s = \frac{3 600}{s_b \times f_w \times f_{HV} \times f_G} $$  \hspace{1cm} (41)

**Base saturation flow rate**

The HCM 2000 (TRB 2000) suggests that a capacity of 1 600 pc/h/ln be used for short-term freeway work zones, irrespective of the lane closure configurations; and for long-term construction zones, capacity values range from 1 550 veh/h/ln (where traffic crosses over to lanes that are normally used by the opposing traffic), to 1 750 veh/h/ln (where no crossover is needed, but only a merge down to a single lane – the value is typically higher).

Bester and Meyers (2007) derived an equation for saturation flow rate based on their study of saturation flow rates at different intersections under different circumstances in Stellenbosch in the Western Cape. Their equation is as follows:

$$ S = 990 + (288 \times TL) + (8.5 \times SL) - (26.8 \times G) $$  \hspace{1cm} (42)

where:

- $S$ = saturation flow or departure rate (pc/h/ln)
- $TL$ = number of through lanes (1 or 2)
- $SL$ = speed limit (60 km/h or 80 km/h)
- $G$ = gradient (percentage), e.g. 3% would be 3.

By setting the number of through lanes to one, and the gradient to zero, Equation 42 can be rewritten as:

$$ s_b = 1 278 + (8.5 \times SL) $$  \hspace{1cm} (43)

Equation 43 was used to determine base saturation flow rates ($s_b$ in pc/h) at different speed limits, as shown in Table 2.

<table>
<thead>
<tr>
<th>Speed limit (km/h)</th>
<th>Base saturation flow rate $s_b$ (pc/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>1 448</td>
</tr>
<tr>
<td>30</td>
<td>1 533</td>
</tr>
<tr>
<td>40</td>
<td>1 618</td>
</tr>
<tr>
<td>50</td>
<td>1 703</td>
</tr>
<tr>
<td>60</td>
<td>1 788</td>
</tr>
<tr>
<td>70</td>
<td>1 873</td>
</tr>
<tr>
<td>80</td>
<td>1 958</td>
</tr>
</tbody>
</table>

for the calculation of waiting time and back-of-queue position, that the speed limit in Table 2 is equal to the average speed through the work zone.

**Lane width factor**

According to the HCM 2000 (TRB 2000, p 16-10) "the lane width adjustment factor $f_w$ accounts for the negative impact of narrow lanes on saturation flow rate and allows for an increased flow rate on wide lanes" and is given by:

$$ f_w = 1 + \frac{(W - 3.6)}{9} $$  \hspace{1cm} (44)

where:

- $W$ = lane width (m) for $W \geq 2.4$ m.

For the purpose of this paper a lane width of 3.1 m was used.

**Heavy vehicle factor**

Heavy vehicles reduce the saturation flow rate because of their lower acceleration rates and the fact that they occupy more road space, both in length and width (Van As & Joubert 2000), compared with passenger cars. The heavy vehicle factor ($f_{HV}$) was given in Equation 11 as:

$$ f_{HV} = \frac{1}{1 + P_{HV} \times (E_{HV} - 1)} $$  \hspace{1cm} (45)

The calculation of the proportion of heavy vehicles ($P_{HV}$) in the traffic stream should be based on vehicle classification sourced from project-specific or local data.

The passenger car equivalent for heavy vehicles ($E_{HV}$) was chosen as 4.3 for the calculation of waiting time and back-of-queue position; and was kept constant for all calculations. This value is consistent with the average passenger car equivalent value quoted by Molina et al (1987, p 32), of 4.3 for a 5-axe heavy vehicle in the front of the queue at signalised intersections.

**Table 2 Base saturation flow rates for different speed limits**

<table>
<thead>
<tr>
<th>Speed limit (km/h)</th>
<th>Base saturation flow rate $s_b$ (pc/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>1 448</td>
</tr>
<tr>
<td>30</td>
<td>1 533</td>
</tr>
<tr>
<td>40</td>
<td>1 618</td>
</tr>
<tr>
<td>50</td>
<td>1 703</td>
</tr>
<tr>
<td>60</td>
<td>1 788</td>
</tr>
<tr>
<td>70</td>
<td>1 873</td>
</tr>
<tr>
<td>80</td>
<td>1 958</td>
</tr>
</tbody>
</table>
Gradient factor

“The gradient factor ($f_G$) accounts for the effect of grades on the operation of all vehicles” (TRB 2000, p 16-10) at traffic signals, namely it is expected that negative gradients will increase saturation flow rates and positive grades will decrease saturation flow rates.

Bester and Meyers (2007) concluded that the departure gradients at intersections have a much greater effect on the saturation flow rate locally than in the USA. They derived an equation for the gradient factor ($f_G$) based on their study of saturation flow rates at different intersections under different circumstances in Stellenbosch in the Western Cape, as follows:

$$f_G = 1 - \frac{G}{71} \tag{46}$$

where:

$G$ = departure gradient (percentage), e.g. 3% would be 3.

Added green time

“Added green time” ($t_G$) is the additional time given by the operator to allow any additional oncoming vehicles, over and above the original stationary queued vehicles, to pass the stop line.

It is therefore assumed that the operator will not “cut off” the moving queue after a certain number of vehicles have passed the stop line, but that all vehicles will clear the stop line before the operator will switch the traffic signal to red (or turn the STOP/GO sign), i.e. the end overflow queue will always be equal to zero.

Average length of type of vehicles in the stream

The average length ($L_i$) of type of vehicles in the traffic stream should be based on information sourced from project-specific or local data.

The SIDRA intersection analysis programme uses an average vehicle length of 5.1 m for light vehicles and 11.0 m for heavy vehicles in its “USA model”, and an average vehicle length of 4.5 m for light vehicles and 10.0 m for heavy vehicles in its “Standard model” (e.g. as it applies to Australia).

The average vehicle length of 4.38 m and 12.55 m, for light and heavy vehicles respectively, were used to calculate the waiting time and back-of-queue position. These values for average vehicle lengths were based on the data of a seven-day electronic traffic count (approximately 48 400 vehicles for the seven days) on the R45 near Malmesbury, in the Western Cape, in 2010.
Average inter-vehicle spacing
The inter-vehicle spacing \( s_i \) is the distance or space, in metres, between vehicles (rear bumper of one vehicle to front bumper of the following vehicle).

Long (Long 2002, p 86) concluded that “from observations measured at a variety of sites (in the USA), inter-vehicle spacings were found to average 3.66 m (12 ft) and were not found to differ significantly at different sites.”

The average inter-vehicle spacing of 3.66 m between all vehicles was used to calculate the waiting time and back-of-queue position at a STOP/GO control for a work zone for half-width construction.

DEVELOPMENT OF DESIGN TABLES AND GRAPHS
By using the iterative procedure in the Excel-based calculation sheet it was possible to develop design tables and graphs to show the waiting time at the front of the queue and the back-of-queue position for different design volumes, work zone lengths, speeds through the work zone, directional splits and percentages of heavy vehicles. All other input variables were fixed at the values given in the previous section. Examples of such tables and graphs are given in Tables 3 and 4, and Figures 1 and 2. From these it can be seen that for a two-way design volume of 600 veh/h, 10% HVs, a 50/50 directional split and a speed of 50 km/h (the 15th percentile heavy vehicle speed), a 5 km work zone will cause a 19.23 minutes waiting time and a back-of-queue position of 1 070 m from the stop line.

SENSITIVITY ANALYSIS OF INPUT VARIABLES FOR THE CALCULATION OF WAITING TIMES AND BACK-OF-QUEUE POSITIONS
The sensitivity analysis was built around a typical work zone scenario with 600 veh/h two-way design volume, 10% heavy vehicles, 60/40 directional split, 3 km work zone length and 50 km/h speed through the work zone. The sensitivity analysis did not make allowance for changes to average vehicle spacing (average vehicle length and inter-vehicle space), lane width, passenger car equivalent value, departure gradient, start-up lost time and operator lost time. The results are shown in Tables 5 to 9.

From the sensitivity analysis it was found that:

■ Waiting time and back-of-queue position exponentially increase with an increase in the traffic volume and the percentage of heavy vehicles.
■ The effect of directional split is not significant.
Waiting time and back-of-queue position increase linearly with the length of work zone.

Waiting time and back-of-queue position decrease linearly with the speed through the work zone.

**Regression Analysis**

The data from Tables 5 to 9 were used in a regression analysis to derive equations for practitioners to approximate the total waiting time and back-of-queue position as follows:

**The total waiting time (min):**

\[ T_d = 14.84 + (2.59 \times 10^{-5}) \times v^2 + 0.00610 \times HV^2 + 3.613 \times L - 0.425 \times v_1 \]  
\((R^2 = 0.890)\)

**And the back-of-queue position (m):**

\[ Q_L = 1.380 - 4.51 \times v + 0.0072 \times v^2 + 0.607 \times HV^2 + 231.2 \times L - 23.73 \times v_1 \]  
\((R^2 = 0.941)\)

where:

- \(v\) = design hourly two-way volume (veh/h)
- \(HV\) = percentage heavy vehicles, e.g. 10% would be 10
- \(L\) = length of work zone (km)
- \(v_1\) = average speed through work zone (km/h).

It should be noted that the above are only valid for the ranges used in the sensitivity analysis.

**Comparison of Results**

A brief comparison of the results from Tables 3 and 4 (for two-way design volume of 600 veh/h and 50 km/h) against a SIDRA analysis (where the cycle length and effective green times were converted from the Excel-based calculation sheet to the SIDRA input parameters) showed reasonably comparable values within a range of ± 15%. The SIDRA analysis results were found to be lower than the tabled values for two-way design values above 600 veh/h, and higher for two-way design values below 600 veh/h.

Although the SIDRA analysis results seemed reasonably comparable with the Excel-based calculation sheet, it should not be used as a validation of the equations or the parameters that were used in this paper, since the parameters in the Excel-based calculation sheet had to be converted to the input parameters used by SIDRA.

**Conclusions and Recommendations**

This paper examined the factors that influence the waiting time for the vehicle in the front of the stationary queue at the STOP/GO control, and back-of-queue position, at a work zone for half-width construction. An Excel-based calculation sheet for determining the back-of-queue position and the maximum waiting time for the vehicle in the front of the stationary queue was developed. This was used to develop "design tables" and "quick design graphs" that could be used by designers and contractors to estimate the back-of-queue position and the maximum waiting time for the vehicle in the front of the stationary queue.

Based on this paper, the following can be concluded:

- Very little information is available or documented in South Africa to accurately determine the back-of-queue position and waiting time for the vehicle in the front of the stationary queue at the STOP/GO control for a work zone with half-width construction.
### Table 8 Variable work zone length – waiting time and back-of-queue position

<table>
<thead>
<tr>
<th>Design two-way volume (veh/h)</th>
<th>Percentage heavy vehicles</th>
<th>Directional split (%)</th>
<th>Work zone length (km)</th>
<th>Speed (km/h)</th>
<th>Waiting time (min.)</th>
<th>Back-of queue position (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>600</td>
<td>10%</td>
<td>60/40</td>
<td>1</td>
<td>50</td>
<td>4.77</td>
<td>293</td>
</tr>
<tr>
<td>600</td>
<td>10%</td>
<td>60/40</td>
<td>2</td>
<td>50</td>
<td>8.73</td>
<td>536</td>
</tr>
<tr>
<td>600</td>
<td>10%</td>
<td>60/40</td>
<td>3</td>
<td>50</td>
<td>12.68</td>
<td>778</td>
</tr>
<tr>
<td>600</td>
<td>10%</td>
<td>60/40</td>
<td>4</td>
<td>50</td>
<td>16.63</td>
<td>1021</td>
</tr>
<tr>
<td>600</td>
<td>10%</td>
<td>60/40</td>
<td>5</td>
<td>50</td>
<td>20.57</td>
<td>1261</td>
</tr>
<tr>
<td>600</td>
<td>10%</td>
<td>60/40</td>
<td>6</td>
<td>50</td>
<td>24.52</td>
<td>1504</td>
</tr>
<tr>
<td>600</td>
<td>10%</td>
<td>60/40</td>
<td>7</td>
<td>50</td>
<td>28.48</td>
<td>1747</td>
</tr>
<tr>
<td>600</td>
<td>10%</td>
<td>60/40</td>
<td>8</td>
<td>50</td>
<td>32.43</td>
<td>1989</td>
</tr>
</tbody>
</table>

### Table 9 Variable speed through the work zone – waiting time and back-of-queue position

<table>
<thead>
<tr>
<th>Design two-way volume (veh/h)</th>
<th>Percentage heavy vehicles</th>
<th>Directional split (%)</th>
<th>Work zone length (km)</th>
<th>Speed (km/h)</th>
<th>Waiting time (min.)</th>
<th>Back-of queue position (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>600</td>
<td>10%</td>
<td>60/40</td>
<td>3</td>
<td>20</td>
<td>35.82</td>
<td>2012</td>
</tr>
<tr>
<td>600</td>
<td>10%</td>
<td>60/40</td>
<td>3</td>
<td>30</td>
<td>22.69</td>
<td>1347</td>
</tr>
<tr>
<td>600</td>
<td>10%</td>
<td>60/40</td>
<td>3</td>
<td>40</td>
<td>16.35</td>
<td>993</td>
</tr>
<tr>
<td>600</td>
<td>10%</td>
<td>60/40</td>
<td>3</td>
<td>60</td>
<td>10.31</td>
<td>634</td>
</tr>
<tr>
<td>600</td>
<td>10%</td>
<td>60/40</td>
<td>3</td>
<td>70</td>
<td>8.67</td>
<td>534</td>
</tr>
<tr>
<td>600</td>
<td>10%</td>
<td>60/40</td>
<td>3</td>
<td>80</td>
<td>7.46</td>
<td>459</td>
</tr>
</tbody>
</table>

- Waiting time and back-of-queue position exponentially increase with an increase in the traffic volume and the percentage of heavy vehicles.
- The effect of directional split is not significant.
- Waiting time and back-of-queue position increase linearly with the length of work zone.
- Waiting time and back-of-queue position decrease linearly with the speed through the work zone.
- The waiting time and back-of-queue position could be estimated by using the Excel-based calculation sheet. This can be used to determine the position of advance warning signs at a work zone.
- The Excel-based calculation sheet could also be utilised to find the optimum work zone length, based on the project parameters, to comply with predetermined criteria for, for example, maximum waiting time of 20 min or maximum back-of-queue position of 2 km.
- Based on the conclusions of this paper, and the fact that it is largely based on literature studies of traffic conditions not related to work zones for half-width construction, the most important recommendation is that all the input parameters which were used need to be verified and calibrated against field data pertaining specifically to work zones for half-width construction.
- A second recommendation is that the “Temporary Congestion” warning sign “TW355”, either as a VMS or not, should be used approximately 150 m in advance of the back-of-queue position at work zones.

The validation of the outcomes of this paper could be used in the economic analysis of roads projects. For example, economic cost of delays, based on daily volumes and typical daily volume distribution graphs, could be used to determine the cost-effectiveness of stop-go versus the construction cost of a bypass.

### REFERENCES


COLTO (Committee of Land Transport Officials) 1998. Standard Specifications for Road and Bridge Works for State Road Authorities. Midrand: SAICE.


Perceptions of professional practitioners and property developers relating to the costs of green buildings in South Africa

D A Coetzee, A C Brent

Sustainable design practices are a key component in ensuring that the building and civil infrastructure industry does not damage our natural environment. Green buildings – and allied initiatives in other sectors of the built environment – are a primary mover in promoting sustainable design practices. An important and dangerous inhibitor of sustainable design practices, however, is the perception among key decision-makers that the additional costs of green – or sustainably designed – buildings are too high, and that they are therefore not economically viable. This study tested what those perceptions are, and compared the results to (limited) actual cost data and found that the cost premium is less than half of what most decision-makers think it is. Recommendations are then made around the development of awareness programmes at both undergraduate and postgraduate levels, the need to highlight the necessity for developing further and more accurate data related to green building costs, and the need to establish incentives to drive the take-up of sustainable design practices.

**INTRODUCTION**

The research that is summarised in this paper is based on the notion of sustainable development and its prerequisite, sustainable design. The sustainability philosophy that underlies sustainable design manifests (partially) in the development of the “green buildings” and “green precincts” movement.

The built environment industry imposes burdens on resources, and thus green

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**Table 1 Practical implications of green building design**

<table>
<thead>
<tr>
<th>Impact area</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Operative indoor temperature</td>
<td>Operative temperature is the result of a combination of air temperature, radiant temperatures (emitted from surfaces inside the building), air velocity and air humidity levels. All these factors impact on the thermal body balance.</td>
</tr>
<tr>
<td>Indoor humidity</td>
<td>As temperature and humidity levels increase, the perceived discomfort increases rapidly, since humans rely on evaporative cooling via perspiration. Humidity levels below 30% lead to irritation of the eyes.</td>
</tr>
<tr>
<td>Air velocity</td>
<td>Drafts are particularly uncomfortable when temperatures are low, and are often negatively perceived when caused by mechanical means.</td>
</tr>
<tr>
<td>Visual comfort</td>
<td>Visual comfort is a function of both natural and artificial lighting. Degree of illuminance, evenness (low contrast), freedom from glare and light colour impact on comfort levels.</td>
</tr>
<tr>
<td>Acoustics</td>
<td>Acoustics are possibly the most overlooked aspect of building design. Acoustic discomfort is often subconscious and not well expressed by occupants.</td>
</tr>
<tr>
<td>Air quality</td>
<td>Air quality is an important factor for health and has been linked to “sick building syndrome” by many studies over the last 30 years.</td>
</tr>
<tr>
<td>Electromagnetic compatibility</td>
<td>Electromagnetic radiation, and its implications for building design, have not been well researched. However, it should be noted as something to consider where possible.</td>
</tr>
<tr>
<td>Energy demand</td>
<td>The primary drivers of energy consumption are: space heating, water heating, space cooling, ventilation and lighting. The biggest opportunity to improve the economic return for green buildings lies in the reduction of energy consumption. Given the rapidly increasing cost of electricity in South Africa, coupled with the ever-increasing price of oil, this aspect provides the most compelling argument for green buildings.</td>
</tr>
<tr>
<td>Water demand</td>
<td>South Africa is a water-stressed country. Although water availability is adequate for the foreseeable future, we cannot consider ourselves to be water secure.</td>
</tr>
</tbody>
</table>

---

Keywords: green buildings, cost premium, perceptions, economic viability, South Africa
Buildings and civil infrastructure are going to play an ever more critical role in world society as population levels increase and natural resources become more and more constrained, and therefore expensive. The CSIR (2002) has estimated that buildings and civil infrastructure consume up to 40% of all resources and produce 40% of all waste, including 30% of greenhouse gas emissions, over the life cycle of the assets – from construction, through the operational life, to final decommissioning. Consumption of water is estimated at about 17% and energy at between 30% and 40% of the entire economy (CSIR 2002: 12).

The Green Building Council of South Africa (GBCSA 2012) states that “…a green building is energy and resource efficient and environmentally responsible. Using design, construction and operational practices, green building is high-performance and reduces the negative impact of construction on people and the environment.”

Some of the practical implications are summarised in Table 1. Many studies have been conducted elsewhere into the cost premiums of implementation versus the savings gleaned over the life cycle of a green building. In general, the studies indicate premiums ranging from 1% to 8% (Tatri & Kucukvar 2011). It is generally accepted, however, that perceptions in the marketplace are that cost premiums are much higher than the research shows, sometimes as much as 30% (Morris 2007). However, these perceptions have not yet been tested in the South African context to better understand the barriers preventing the pursuing of green buildings and sustainable design as a country.

**The economics of green building**

According to Boston Mayor, Thomas Menino (in Kubba 2010b: 305), “High performance green building is good for your wallet. It is good for the environment. And it is good for people.” This statement defines one of the basic premises of this study, namely that green building and sustainable design make economic sense.

Brown *et al* (2011) speak of the 4th economic revolution, one defined by resource scarcity. The first three – Agrarian, Industrial and Digital – are part of accepted economic theory. The 4th will be defined by the following five new competencies:

- Collaborative growth and efficiency
- Zero waste
- Renewable resources

- Climate resilience (an ability to adapt to climate changes that we are already seeing now), and
- Eco-performance measurement (measuring resource usage, particularly energy and water).

Some of the economic benefits of green buildings are:

1. **Increased income:**
   - a. Higher rental rates
   - b. Higher rental growth, and

2. **Lower costs:**
   - a. Operating costs:
     - i. Lower energy costs, and
     - ii. Lower water costs.
   - b. Reduced life cycle and capital costs.

Comparisons of costs for green buildings versus standard buildings are difficult because of the wide range of costings and many third variables. However, in spite of this the United States Green Buildings Council (USGBC 2012) indicates that:

1. The cost per unit area for LEED-certified buildings falls into the range of standard buildings. In other words, there are very little discernible cost premiums for green buildings [LEED stands for Leadership in Energy and Environmental Design].

2. LEED-certified office rentals are on average 11% higher than the average for multi-tenant Class A office space.

3. Operating costs of green buildings are 13.6% lower for new buildings and 8.5% lower for existing buildings.

4. Building values are 10.9% higher on new construction and 6.8% on existing buildings.

5. Returns on investments are 9.9% higher on new and 19.2% higher on existing buildings.

6. Occupancy rates are 6.4% higher for new and 2.5% for existing buildings.

7. Rental premiums are 6.1% higher for new and 1% higher for existing.

8. LEED gold building in the general services category use 25% less energy and 11% less water.

9. Maintenance costs are 19% lower, and this value is supported by Fowler *et al* (2010).

10. Occupant satisfaction is 27% higher in LEED-certified office buildings.

11. Green buildings have 34% lower greenhouse gas emissions.

The 2009 *Greening of Corporate America* report (McGraw Hill Construction & Siemens 2009: 2) indicates that 71% of corporates in North America expect a cost benefit from sustainability adoption (see Figure 1). This represents a major move towards a positive perception of the cost benefit of sustainable design and construction in America. The same report shows that
the most important driver of green buildings is increased energy costs (McGraw Hill Construction & Siemens 2009: 7).

Generally research indicates that energy (25% lower) and water (11% lower) (Fowler et al 2010) provide the largest opportunities for cost savings, and this makes intuitive sense, especially in the South African context where the country is facing major constraints.

The Incisive Media 2008 Green Survey: Existing Buildings showed that 65% of building owners report a positive return on their investment in green building features (in Kubba 2010b: 306).

The Turner Construction Company reported that their 2010 survey showed that almost all building owners and developers intend to incorporate some level of green building features into their next projects, with reduced energy and operating costs being the most common drivers. However, the focus is now extending to other benefits, such as health and well-being, and corporate reputation (see Figure 2).

Morris (2007) reports anecdotal evidence of perceptions of the green building premium ranging as high as 30%. Kats (2009) summarised research from various sources, including the World Business Council for Sustainable Development, and showed (see Figure 3) that there is indeed a significant disparity between actual cost premiums and perceptions, with an actual cost premium median at below 2% and perceptions at 17%, as shown in Figure 4 (WBCSD 2008).

In 2006 the Green Building Council of Australia reported that negative perceptions regarding green buildings were rife, and there was a belief that cost penalties were significant and tenants did not care about “green”. By 2008 this had changed and the focus had moved away from cost towards benefit (GBCA 2008: 12).

The perception is linked to the cost premium perception, because payback is the final measure of benefit. North American opinions of acceptable payback periods are shown in the Turner Construction Company Green Building Market Barometer (2010), as shown in Figure 5.

Figure 5 shows a fairly short required payback period, with 40% of US organisations requiring a four- to five-year period in terms of the Turner Construction Company Green Building Market Barometer (2010) survey. This appears to correlate with experiences in the South African market, where developers are believed to sell their interest fairly early on and therefore require short payback periods. This is known as a split incentive and is cited by the GBCSA (2012: 102) as a barrier to green building development in South Africa. This occurs because the entity that pays for the design and building of the facility is often not the one that operates and maintains it. Therefore required payback periods were the most significant inhibitor of green building development. In its Green Jobs Study the United States Green Building Council (USGBC 2009) developed data that shows an average payback of just less than five years (see Table 2).

The LEED-related spending value of US$4.01 divided by the annual savings of 90c gives a payback of just under 4.5 years; this is therefore consistent with the Turner Barometer.

Davis Langdon has probably been the most active researcher into the cost issues of green buildings. In an initial report in 2004 they reported no statistically significant differences between average costs per square foot for LEED buildings versus non-LEED buildings (Davis Langdon 2004). In a later report a 3% to 5% cost premium for a 5 Star (LEED) building, and a further 5% for a Gold Star building (Davis Langdon 2007) were documented. This represents US$19/m² for a possible change from 4 to 5 Star and US$40/m² for a change from 4 to 6 Star.

A number of additional key points are likely to affect the economic balance of green building implementation in the near future (Davis Langdon 2007):

1. The impact of possible future taxes related to sustainability.

Table 2 LEED spending and savings data per square foot

<table>
<thead>
<tr>
<th>Category</th>
<th>Average value per sqft</th>
<th>Number of buildings</th>
</tr>
</thead>
<tbody>
<tr>
<td>LEED-related spending</td>
<td>$4.01</td>
<td>18</td>
</tr>
<tr>
<td>Energy savings</td>
<td>($0.51)</td>
<td>14</td>
</tr>
<tr>
<td>O&amp;M savings</td>
<td>($0.32)</td>
<td>5</td>
</tr>
<tr>
<td>Water savings</td>
<td>($0.05)</td>
<td>7</td>
</tr>
<tr>
<td>Trash savings</td>
<td>($0.02)</td>
<td>5</td>
</tr>
</tbody>
</table>
2. The diminishing of the “additional cost” perception as green buildings become the norm (mainstreaming).
3. Marketplace influence and hence marketability, as green buildings become more mainstream and therefore expected. This applies particularly to the premium end of the market.
4. A growing tendency towards a full life cycle analysis.
5. A growing appreciation for the productivity benefits of green buildings, which have previously been discounted as too “soft” to evaluate.
6. The possibility of carbon trading systems.

Current status in South Africa
South Africa has made some advances in the green building space. Specifically, the Green Star rating system was developed, and has been managed by the Green Building Council of South Africa as a voluntary tool that provides the property industry with “an objective measurement for green buildings, and recognises and rewards environmental leadership in the property industry” (GBCSA 2012). However, the country has not yet reached a maturity level comparable with countries such as the United States, the United Kingdom or Australia (GBCSA 2012). This implies that the country needs to move from its current state to an improved position with regard to how it designs its built environment.

In 2011 the Department of Trade and Industry (DTI) added sections to the National Building Regulations (SANS 10400) that relate to environmental sustainability of (Part X) and energy usage in (Part XA) buildings. Although mandatory, the interpretation of the amended regulations is not standardised and remains a challenge (CivilSure 2015). O’Rourke (2015), in his examination of the natural building movement in South Africa, highlights costs as a key driver of change – environmental benefits that surpass the mandatory regulations, and banking requirements are still perceived as expensive. A lack of knowledge in the market is highlighted as a central theme that inhibits the green building movement.

Objective of this paper
The objective of this paper is therefore to confirm whether there is a predisposition by built environment professionals and property developers to overstate the cost premium of green buildings in South Africa. The paper then attempts to confirm whether that belief is reasonable and, if not, gathers evidence to prove that the perception is wrong. The purpose is to assist in providing knowledge for the development of green buildings in South Africa.

RESEARCH METHODS
An action research approach was followed, using case information, together with a perception survey. Thus the research comprised an inductive, non-experimental field study with no planned intervention and no random assignment (Welman et al 2005:34). The sample was self-selecting, because the surveys were sent out to a stratified random sample of built environment professionals and property developers, and the researcher had no control over the response rate.

The survey, as the main research instrument, was structured to provide a view of the perception of built environment professionals and property developers towards the costs of green buildings in South Africa. A relevant “third” variable is the grouping into which the respondents fall. The following groups were identified and the data grouped accordingly:
- Property developer – Commercial
- Property developer – Industrial
- Property developer – Residential
- Professional – Architect
- Professional – Building services engineer
- Professional – Electrical engineer
- Professional – Quantity surveyor

The data was analysed according to these groupings to determine whether there were any discernible patterns.

A further “third” variable is geographical, i.e. in terms of the regions where the respondents reside. Accordingly the analysis also considered these groupings to determine any possible patterns.

The primary question, regarding cost premium, was analysed as univariate data to determine distribution, mean mode and median, and analysed using appropriate statistical formulae, such as the single population median and its confidence range.

■ The independent variable is:
  - Built Environment Professionals and Property Developers

■ The dependent variable is:
  - Cost Premium Perception

An online survey was utilised. The advantage of an online survey is that it is easy to administer and distribute. The disadvantage is that an online survey requires any potential respondent to have access to the internet in order to complete the survey.

RESULTS AND DISCUSSION

Data gathering process
The original intention was to issue the perception survey via the various professional councils in the built environment. The Engineering Council of South Africa (ECSA), the South African Council for the Architectural Profession (SACAP) and the South African Council for the Quantity Surveying Profession (SACQSP) were approached and asked to support the survey. The South African Property Owners Association (SAPOA) was also approached in order to obtain input from this important part of the built environment industry.

Of these ECSA agreed to issue a request to complete the questionnaire to its registered members, SACQSP advised that it would not issue unsolicited email requests to its members, and SACAP did not respond. SAPOA indicated interest, but did not issue its request in time for the data to be considered.

The Association of South African Quantity Surveyors (ASAQS) provided enthusiastic support and issued a request to its members. The South African Institute of Architects was also approached, but did not respond in time.

The Green Building Council of South Africa (GBCSA) was willing to request its members to respond to the survey. This offer was declined, however, as it was felt that GBCSA members were likely to be biased towards sustainably designed buildings, as well as having better knowledge of their benefits. It was felt that a more representative sampling procedure would be obtained by accessing GBCSA registered members via the other professional bodies. In fact, 7% of respondents were members of the Green Building Council of South Africa.

Data gathered
ECSA issued the survey to its registered members in July 2013. Due to confidentiality issues and the nature of the ECSA mailing list, the survey request was issued to all registered members and no sampling methodology could be employed. However, based on the number of registered members, more than 20 000 professionals were reached through the survey.

A total of 1 192 responses were received between 5 July and 8 August 2013. Over 51% of all responses were received on the first day, with almost 90% received in the first eight days. These responses where all the result of the requests from ECSA and ASASQ to complete the survey.

Data results
The main question asked in the survey was what the respondent’s perception of the cost premium of green or sustainably designed buildings over standard buildings is. The results are shown in Figures 7 and 8.

The weighted median perception was 20.36%. It should be noted, however, that the responses have highlighted a weakness in the
survey design (as can be seen in Figure 6). The options for cost premium in the survey were set in 5% intervals from 0%–5% up to 25%–30%, with a “catchall” of > 30% for anything above. The results show that 22.5% of respondents (nearly one in four) selected “Above 30%”. This shows that the survey design should have had at least a further two or more categories, namely 30%–35% and 35%–40%. Given this weakness, the mean is a less reliable measure of central tendency. The median is a better measure and has been used in the analysis.

In the analysis of the median percentage premium the calculation is based on assuming that all “Above 30%” respondents fall into a 30%–35% interval. This, whilst conservative, may be misleading and will result in an understatement of the median value. Due to the relatively large sample, the confidence range of the results is narrow (1%) (refer to Table 3). The mean, median and mode values were calculated using grouped data methods.

Unfortunately only 16 quantity surveyors responded to the survey, thus not a statistically representative number. However, the median value of 12.9% is much lower than that of the whole group value. This most likely reflects the better financial knowledge of quantity surveyors. It also correlates better with the limited actual cost premium data available in South Africa (see Tables 4 and 5), and in other countries.

Whilst there is a shortage of information about the actual cost premium of green or sustainably designed buildings in South Africa, the following data is available:

2. WSP South Africa (2011) – see Table 5. These case studies indicate that green buildings in South Africa could reasonably be expected to cost between 2% and 10% more than so-called “standard buildings”.

The median cost premium perception of the sample (n = 1192) gives a median cost premium of (approximately) 20% to 21%. This compares well with other research that indicates a value of 17% (WBCSD 2008). It is reasonable to expect South Africa to have a higher cost perception due to the later take-up of sustainable building design in this country, since South Africa is at a lower maturity level with regard to green buildings than leading overseas countries (GBCSA 2012). As a new technology becomes embedded it generally gets less costly as it becomes mainstream. The perception is that the South African premium is higher than First World countries, but that it will drop over time.

The actual data compiled by WSP (2011) and GBCSA (2012) clearly indicates values of 2% to 10%. If a worst case of 10% for actual costs is assumed, then the South African perception of the cost is double or 10% higher than what it actually is.

This has important ramifications for the take-up of sustainable design in South Africa, because decision-makers will not pursue green or sustainable design practices if they believe them to be too expensive, or if they do not believe that green practices will provide an economic benefit. The data clearly shows that the actual cost is significantly lower than perceptions. This means that the potential take-up of sustainable design practices in the built environment is being inhibited inappropriately.

This is particularly unfortunate, because the attitude of key decision-makers is not at all negative towards the idea of sustainable design. Less than 5% say they actively do not intend to pursue sustainable design practices. Despite the cost perception mismatch with actual cost, and the fact that nearly 29% of respondents feel that the cost recovery takes too long, nearly 63% still believe that green or sustainably designed buildings make economic sense.

Furthermore, there is a mismatch between perceptions of the payback period versus the required payback period. The perception of the payback period is seven years, while the required payback period is just under five years. This requirement of developers is consistent with international data – 40% of respondents to a Turner Construction Company Green Building Market Barometer
Table 3 Confidence range calculation – all respondents

<table>
<thead>
<tr>
<th>Group</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>Σ</th>
</tr>
</thead>
<tbody>
<tr>
<td>x</td>
<td>2.50%</td>
<td>7.50%</td>
<td>12.50%</td>
<td>17.50%</td>
<td>22.50%</td>
<td>27.50%</td>
<td>32.50%</td>
<td></td>
</tr>
<tr>
<td>$f_i$</td>
<td>35</td>
<td>114</td>
<td>205</td>
<td>227</td>
<td>206</td>
<td>137</td>
<td>268</td>
<td>1 192.00</td>
</tr>
<tr>
<td>$f_{sx}$</td>
<td>0.875</td>
<td>8.55</td>
<td>25.625</td>
<td>39.725</td>
<td>46.35</td>
<td>37.675</td>
<td>87.1</td>
<td>245.90</td>
</tr>
<tr>
<td>$x^2$</td>
<td>0.0625%</td>
<td>0.5625%</td>
<td>1.5625%</td>
<td>3.0625%</td>
<td>5.0625%</td>
<td>7.5625%</td>
<td>10.5625%</td>
<td></td>
</tr>
<tr>
<td>$f_{sx^2}$</td>
<td>0.02188</td>
<td>0.64125</td>
<td>3.20313</td>
<td>6.95188</td>
<td>10.4288</td>
<td>10.3606</td>
<td>28.3075</td>
<td>59.92</td>
</tr>
</tbody>
</table>

Confidence interval

$t$-Statistic for 95% confidence interval, two-tailed test ($n = 1 192$) = 1.962

\[
\bar{x} = \frac{\sum x}{n}
\]

\[
\text{Mode} = 17.56% \\
\text{Median} = 20.36% \\
S^2 = \frac{\sum x^2 - n\bar{x}^2}{(n-1)} = 0.77% \\
S = \sqrt{S^2} = 8.78% \\
S.E = \frac{S}{\sqrt{n}} = 0.25% \\
7 \text{ Above 30%}
\]

95% Confidence range

Lower limit = 20.13% \\
Upper limit = 21.13% \\
Range = 1.00% \\
Range as % of mean = 4.8%

Table 4 GBCSA cost premium data

<table>
<thead>
<tr>
<th>Building name</th>
<th>Rating</th>
<th>Capital premium</th>
<th>Submission costs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aurecon Century City</td>
<td>5 Star design</td>
<td>5%–8%</td>
<td>1.7%</td>
</tr>
<tr>
<td>Nedbank Phase II</td>
<td>4 Star design</td>
<td>3.3%</td>
<td>&lt; 0.5%</td>
</tr>
<tr>
<td>Mayfair-on-the-Lake</td>
<td>4 Star design</td>
<td>5%</td>
<td>1%</td>
</tr>
<tr>
<td>24 Richefond Circle</td>
<td>4 Star design</td>
<td>10%</td>
<td>R750k</td>
</tr>
<tr>
<td>Falcon Building Menlyn Maine</td>
<td>4 Star design</td>
<td>9.2%</td>
<td>0.55%</td>
</tr>
<tr>
<td>Aurecon Lynnwood Bridge</td>
<td>4 Star design</td>
<td>2.6%</td>
<td>0.26%</td>
</tr>
<tr>
<td>Forty on Oak</td>
<td>4 Star design</td>
<td>&lt; 1%</td>
<td>Not available</td>
</tr>
<tr>
<td>ABSA Towers West</td>
<td>5 Star as-built</td>
<td>&lt; 2%</td>
<td>Not available</td>
</tr>
</tbody>
</table>

Table 5 WSP cost premium data

<table>
<thead>
<tr>
<th>Building</th>
<th>Floor area</th>
<th>Capital premium</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>129 055</td>
<td>2.0%</td>
</tr>
<tr>
<td>2</td>
<td>56 000</td>
<td>3.5%</td>
</tr>
<tr>
<td>3</td>
<td>46 000</td>
<td>4.0%</td>
</tr>
<tr>
<td>4</td>
<td>46 000</td>
<td>3.5%</td>
</tr>
<tr>
<td>5</td>
<td>45 000</td>
<td>2.6%</td>
</tr>
<tr>
<td>6</td>
<td>38 000</td>
<td>3.5%</td>
</tr>
<tr>
<td>7</td>
<td>20 000</td>
<td>3.0%</td>
</tr>
<tr>
<td>8</td>
<td>16 027</td>
<td>5.0%</td>
</tr>
<tr>
<td>9</td>
<td>15 000</td>
<td>5.0%</td>
</tr>
<tr>
<td>10</td>
<td>6 479</td>
<td>7.5%</td>
</tr>
</tbody>
</table>

1. There is a lack of training in sustainable design principles at undergraduate level.
2. There is a perception that a lot of current development in the green building sector is based on it being "fashionable", rather than for responsible design or good practice reasons.
3. The implementation of sustainable design principles in the built environment should be part of all good design, and should not be considered as something extra or new. This is particularly true for energy consumption – responsible design must strive for maximum efficiency beyond compliance to the regulations.
4. A large number of those surveyed felt that sustainable design was both ethically correct and socially responsible. This means that there is a strong foundation to build a sustainable design movement upon.
5. Many expressed the feeling that the "compliance" aspect of the green building certification programme is too "rules-based" and does not adequately recognise innovative thinking. The additional design costs caused by certification were also challenged. This could lead to projects that do not pursue certification, but may nevertheless follow the principles of green buildings. This is not necessarily a bad thing.
6. The approach being followed in South Africa is too closely modelled on the First World and is not sufficiently aligned with the reality of Africa.
7. Part of the cost premium problem is due to various parties taking advantage and charging more than necessary because of the immaturity of the market. This seems to be mostly directed at solar heating. Also part of this issue is untrained and unscrupulous operators trying to make money by "peddling" products under the moniker of "green", but which may not in fact support sustainability principles.
8. Local government needs to become more active in creating an enabling environment for sustainable design (for example, through the use of such systems as smart metering). This is inhibited by the lack of capacity in that sector.

9. There is a disjunction between the initial development of the building, which may be based on sustainability principles, and the operations, which often do not follow the same principles.

10. The split incentive was raised. This is where a developer pays the cost, but the tenant reaps the reward. This problem is addressed by the concept of a green lease, which attempts to share the benefits of a sustainably designed building between landlords and tenants. It also attempts to ensure that a sustainably designed building is operated in a sustainable way. This addresses the problem raised in the preceding point.

11. Existing specifications and practices were highlighted as constraints against the adoption and implementation of sustainably designed buildings. The key issue is that existing ways of doing things often counter the objectives of sustainable design. This means that it is important to ensure that all existing legislation, standards, by-laws and other regulations, specifically SANS 10400 Parts X andXA, as well as "best practice" are aligned with the principles and objectives of green buildings and sustainable design, and that they are also coherent.

**CONCLUSIONS AND RECOMMENDATIONS**

**Contribution to practice**

The objective of this survey was to test the perceptions of local built environment decision-makers to determine whether they believe that the cost premium of implementing green or sustainably designed buildings is more than it actually is. The data shows that this is the case – the perception of the cost is more than double what the actual cost premium is. This perception is consistent with international research.

The results also show that, despite these beliefs, there is a very positive attitude towards green or sustainably designed buildings. This means that there is a sound platform for promoting and improving sustainable design principles in the built environment.

The study shows that a well-constructed, facts-based information disseminating programme is likely to result in an accelerated take-up of green buildings built on sustainable principles, because there is a clear mismatch between the perception and the actual data.

It also shows that there is a need to assemble further (and more accurate) data regarding the actual cost premium of green or sustainably designed buildings, as well as data about the potential savings in operating costs that result from implementing more efficiently designed buildings. These two areas of data and information will provide better support to any initiative to increase the implementation of green buildings.

**Conclusions**

This study shows that the cost of implementing sustainable design practices in the built environment is less than most key decision-makers believe. Further studies are required to confirm international data which shows that the payback of such investments is less than five to seven years. However, it is certain that the net present value of any such investment is positive, because the lifespan of a building is more than thirty years, i.e. the benefit is realised for the investor over the lifespan.

The primary recommendations of this study are for more extensive and more accurate data and information, and for further education of decision-makers. This is likely to accelerate the take-up of sustainable design and operational practices in the built environment.

The following extracts from responses to the survey highlight three important points with regard to green buildings and sustainable design:

- "Sustainable design should be pursued as a matter of course. It is often cheaper than conventional design, so there is no 'pay-back' period."
- "I measure everything against cost. If it makes financial sense I incorporate it. Green in my language is written gReen."
- The words "It’s the right thing to do" were mentioned in 13 separate narrative comments.

The study shows that there is a need to assemble further (and more accurate) data regarding the actual cost premium of green or sustainably designed buildings, as well as data about the potential savings in operating costs that result from implementing more efficiently designed buildings. These two areas of data and information will provide better support for any initiative to increase the implementation of green buildings.

This research highlights the lack of good data regarding the actual cost premium of green or sustainably designed buildings, as well as the economic payback that they produce. This is required in order to compare the results of a perception study (such as this one) to the real facts. The type of data needed relates to both the capital cost of a building designed for sustainability and the potential life cycle savings that can be generated by such a building during its operational life.

**Recommendations**

- **Recommendation 1**: Carry out on-going detailed studies of the additional capital cost of green or sustainably designed buildings using all the data available for the current Green Star certified buildings. Categorise this, based on the rating achieved (Four Star, Five Star or Six Star).

- **Recommendation 2**: Carry out on-going detailed studies of the potential operational cost savings of green or sustainably designed buildings, using all the data available for the current Green Star certified buildings. Categorise this based on the rating achieved (Four Star, Five Star or Six Star).

- **Recommendation 3**: Implement an on-going programme to track the actual operational cost of Green Star certified buildings so that a comparison can be made against the savings predicted (as per Recommendation 2 above).

- **Recommendation 4**: Derive average payback periods for green or sustainably designed buildings using the information calculated in Recommendations 1, 2 and 3 above. Notwithstanding the limitations described above, this survey has clearly shown that there is a mismatch between what built environment decision-makers believe the green building cost premium is and what it really is. This means that there is a need to provide information and training to the industry to clarify misperceptions. In order to do this, one requires the information generated from Recommendations 1 through 4.

- **Recommendation 5**: Develop information sheets and training programmes which provide data to built environment professionals and other decision-makers, to support the notion that green or sustainably designed buildings are cheaper than they might think, and that the payback period for the additional investment is not only less than they think, it is well within the lifespan of a building and therefore results in a positive net present value.

- **Recommendation 6**: Elevate the principles of green buildings and sustainable design to the highest possible significance levels in undergraduate education in built
1. Personal know-how
2. Business community acceptance
3. Supportive corporate environment
4. Personal commitment
5. Economic demand
6. Positive climate impact
7. Pragmatic involvement
8. Building attractiveness.

Of these factors the first four are all related to the perceptions and attitudes of those involved in the industry. This highlights the importance of accumulating and disseminating accurate information to role players, as described in the recommendations above.

REFERENCES


INTRODUCTION

Ultra-thin continuously reinforced concrete pavement (UTCRCP) is a developing pavement type and has the potential to fulfil many of South Africa’s pavement repair strategy requirements (Kannemeyer et al 2007). UTCRCP consists of an approximately 50 mm thin layer of 80 MPa high-strength concrete. It is reinforced continuously with small-diameter (5 mm), closely spaced, welded deformed steel bar mesh. Research by Kannemeyer et al (2007) showed that UTCRCP is sensitive to the development of cracks large enough to allow moisture ingress. Premature failures occur when moisture deteriorates the pavement substructure, resulting in loss of support in the overlay (SANRAL 2012).

Cracks are formed in concrete when restrained deformation induces tensile stress greater than the tensile strength at the time. Concrete shrinkage and thermal movement both influence the deformation of concrete (Giussani & Mola 2012). The materials and proportions of materials used in concrete control the deformation of concrete (Neville 1995).

It has been stated by SANRAL (2009) that the mixture proportioning of HSC is one of the main challenges in UTCRCP. The mixture proportioning of HSC is more complex than normal-strength concrete, because low water/cementitious ratios and a wider variety of materials are used (Zain et al 2005). Workability and strength requirements are the main factors governing the mixture proportions used in concrete. Paste content consists of water, cement and admixtures, and is expressed as a percentage of the total concrete mass or volume. In the mixture proportioning of HSC there is an inclination to increase the paste content to facilitate workability (Alves et al 2004). The high paste content and low water/cementitious ratio of HSC increase its cracking tendency. Crack formation in concrete can be controlled by taking cracking tendency into account during the mixture proportioning process (Khokhar et al 2010).

The objective of this paper is to determine the effect of paste content on the properties of HSC, where the observed trends serve as a starting point for further research in minimising the cracking tendency in UTCRCP by mixture proportioning. The paper presents a literature review, the experimental programme and the analysis of the experimental data. Lastly, conclusions are drawn and recommendations are made.
cumulative contraction and expansion that induces a resultant stress in the presence of restraint. Load-independent deformation includes plastic shrinkage, chemical/autogenous shrinkage, drying shrinkage, thermal deformation and carbonation shrinkage. Load-dependent deformation consists of elastic strain and creep. Load-independent deformation is a function of a combination of moisture and temperature effects. Load-independent strains induce stresses when the concrete is restrained, and lead to deformation that is affected by load-dependent deformation. Figure 1 is a schematic diagram which was compiled to clarify the load-dependent and load-independent deformation.

Cracks can start forming in concrete within the first 24 hours (Holt 2001). Figure 2 is a schematic illustration of crack development in concrete. It shows how a crack develops when the stress exceeds the tensile strength (Owens 2009). Figure 2 also shows how cracking tendency is affected by time. As concrete hardens the tensile strength increases, reducing the cracking tendency, while the modulus of elasticity increases and stress alleviation by creep decreases, increasing the cracking tendency.

HSC mix designs are often described as ‘rich’ because of their high cement content. The cement content is dependent on the water content, which has the tendency to be high to maintain the workability at low water/cementitious ratios. High cementitious content also ensures high early strength gain (Neville 1995). Conventional mixture proportioning methods have restrictions on the amounts of cement used to limit shrinkage cracking (Neville & Brooks 2010). These restrictions are not always implemented for HSC. Acceptable cement content ranges between 500 and 550 kg/m³, while HSC has been made with cement content up to 1 000 kg/m³ (Alves et al 2004).

In a study by Alves et al (2004) the mixture proportions obtained from four methods used to design HSC in Brazil were compared. Three of the four methods showed how the paste content increased as the water/cement ratio decreased. The only method that did not show this tendency was the mix design method based on the particle packing theory. The other mix design methods were a method used for the mix design of normal strength concrete and methods adjusted from normal strength concrete mix design methods for HSC. The research emphasised the tendency of replacing fine aggregate with binder material to maintain workability in HSC. The suitability of a mix was determined by its cost and strength, and the cracking tendency of the mix designs was not taken into account.

The liberation of heat, during the exothermic chemical reaction of cement hydration, causes a temperature rise in the concrete member. The temperature rise is dependent on various factors, including material properties, member geometry and environmental conditions that can be adiabatic or semi-adiabatic (RILEM 1997). Increasing the paste content of a concrete mix would increase the maximum temperature reached due to heat of hydration.

The combination of heat of hydration and thermal expansion can cause cracks (Neville 1995). The temperature of the concrete rises because of heat of hydration. As the heat dissipates, the magnitude of thermal contraction is dependent on the difference between the peak temperature and the ambient temperature. If the concrete is restrained the contraction causes tensile stress in the concrete, because the concrete hardens and gains strength at elevated temperatures. Tensile stresses sufficiently large to crack the concrete can develop (Domone & Illston 2010).

To obtain high early strength in HSC the cementitious content is often increased while the water/cementitious ratio remains constant (Bentz et al 2011). The effect of the increased paste content on the long-term properties of HSC is not clear, but it has been shown for normal strength concrete that leaner mixes yield higher strengths after only seven days (Singh 1958). Stock et al (1979) found that, at a water/cement ratio of 0.5, compressive strength decreases
as the aggregate content increases from 0% to 40% of total volume. With aggregate content exceeding 40% the compressive strength increases. This phenomenon can be explained by a combination of factors that include reduced total interfacial transition zone area for low aggregate content concrete and reduced porosity of high aggregate content concrete. The modulus of elasticity of concrete is significantly influenced by paste content. Autogenous shrinkage is important in HSC, because low water/cementitious ratios are used to obtain high strengths. The magnitude of autogenous shrinkage increases as the water/cementitious ratio decreases. Autogenous shrinkage is important in HSC, because low water/cementitious ratios are used to obtain high strengths. The magnitude of autogenous shrinkage of concrete is also proportional to paste content.

Drying shrinkage is the volumetric contraction of concrete by the removal of water. Concrete with high water/cement ratios contain more water that can be removed from the paste. Similarly concrete with high paste content will also shrink more, due to the overall higher water content. When the aggregate content is increased from approximately 71% to 74%, drying shrinkage can be reduced by as much as 20% (Neville 1995).

The relative proportion of paste and aggregate influences the modulus of elasticity of concrete, but it is not usual to find significant variation due to this factor (Gutierrez & Canovas 1995). Contrary to this, Leemann et al (2011) found that an increase in paste content decreased the modulus of elasticity by as much as 20% for concrete containing 320 kg/m³ and 520 kg/m³ cement at a constant water/cement ratio. Aggregate volume concentration also affects the creep of concrete by restraining the movement of paste. An increase of aggregate content by volume from 65% to 75% can decrease creep by 10% (Neville 1995). Leemann et al (2011) found that concrete with high paste content, such as self-consolidating concrete, exhibits higher creep.

**EXPERIMENTAL PROGRAMME**

**Response Surface Methodology**

Design of Experiments (DoE) is the strategic planning and execution of experiments to reduce the experimental work required to obtain statistically relevant results. Response Surface Methodology (RSM) is a set of statistical and mathematical techniques (used in DoE) that assist in the modelling and analysis of responses that are influenced by a number of variables. The end goal of RSM is the optimisation of the response (Montgomery 2001).

RSM usually consists of three phases, where phase one focuses on the experimental study, phase two develops the response surface models and phase three uses the statistical models for optimisation (Lotfy et al 2014).

Central Composite Design (CCD) is the RSM design used in this study. CCD is an augmented version of the factorial design with centre points (Stat-Ease Corporation 2014). CCD is useful in RSM, because it makes it possible to develop first- and second-order models.

True functional relationships can be approximated for a small range of variables by polynomials of higher degree. In a second-order model, curvature is represented by interaction and quadratic terms. A central composite matrix does not make it possible to plot a cubic model. Equation 1 is the characteristic form of a second-order model:

$$y = \beta_0 + \sum_{j=1}^{k} \beta_j x_j + \sum_{i<j} \beta_{ij} x_i x_j + \sum_{j=1}^{k} x_j^2 + \epsilon$$

where: $\sum_{j=1}^{k} \beta_j x_j^2$ = quadratics effects of a single variable $\sum_{i<j} \beta_{ij} x_i x_j$ = interaction effect between two variables $\beta_{ij}$ = regression coefficients $x_i, x_j$ = investigated factors $k$ = number of factors $\epsilon$ = observed noise or error

CCD consists of $2^k$ factorial points, 2k axial points and n centre runs. In this context $k$ represents the number of independent variables. Figure 3 shows the three different types of points that define the region of interest for a two-factor design. The factorial points are situated on the corners of the square, the axial points are situated at distance alpha ($\alpha$) away from the centre point on the negative and positive sides of each axis, and the centre points are situated at the intersection of the two axes.

The quality of prediction of a response surface design is improved when it is rotatable. Whether a response surface design is rotatable is dependent on alpha and the number of centre points. Alpha can be calculated as a function of the number of independent variables. When two independent variables are being tested, alpha will be equal to the radius of a circle fitted on the factorial points that form a square. (Note that the factorial points are also distance alpha away from the centre.)

Figure 3 Central composite design with two variables (Montgomery 2001)
General Linear Model Analysis of Variance (GLM-ANOVA) is used to determine the influence of a test parameter on the tested response. The higher-order terms that model curvature are excluded to develop a linear model of the response after which the ANOVA of the model is used to evaluate the effect of the test parameters. For a parameter to have a significant influence on a response the $F$-value has to be large and the probability associated with it, the $p$-value, has to be smaller than 0.05. Lotfy et al. (2014) describes the $p$-value as a descriptor of the significance of the parameter on the test results. Similarly, the percentage contribution is a measure of the effect of the independent variables on the response.

Materials
Table 1 shows the relative density and Figure 4 shows the particle size distribution of the fine materials used in this study. The materials used were selected according to quality and availability. The particle size distribution of silica fume could not be obtained because of the material’s tendency to agglomerate in suspension.

Mix design
The effect of paste content on the properties of HSC was determined using two sets of experiments. For the first set, multivariable analysis was used where the two parameters that control workability (paste content and superplasticiser (SP) dosage) were varied. CCD was used as the response surface design. The concrete set and its results are labelled as Set 1. When response surface design is applied, the range in which the parameters are varied must be such that the resulting concrete mixes must be usable. In using CCD the parameters could not be varied over a very wide range, because the concrete mixes were not allowed to be too dry to compact and the SP dosage was not allowed so high that it would cause segregation.

In the second set of experiments the effect of the paste content was of primary concern and the SP dosage was only varied to ensure good workability for each concrete mix. The base mix design was derived from a published mix design for UTCRCP (Mukandila et al. 2009). The concrete set and its results are labelled as Set 2. Figure 5 shows the range over which the parameters were varied for each set of concrete. The paste content is calculated as a percentage of the concrete mass and the SP dosage is calculated as a percentage of the cementitious content, which is all cement, fly ash and silica fume. The target compressive strength was 80 MPa.

Thirteen concrete mixes were designed for the CCD response surface modelling where alpha equals the square root of two. As stated, two parameters were varied, paste content (22.9% to 37.1%) and SP dosage (0.238% to 1.262%). Table 2 shows the coded and actual values of the variables. Design Expert (Stat-Ease Corporation 2014), commercial software, was used for the statistical evaluation of the results and development of the mathematical models.

The mix designs of Set 1 can be viewed in Table 3 and the mix designs of Set 2 concrete can be viewed in Table 4. Note that the sand content decreased as the paste content increased.
Table 3 Mix designs of Set 1

<table>
<thead>
<tr>
<th>Coded variable level</th>
<th>Mix 1</th>
<th>Mix 2</th>
<th>Mix 3</th>
<th>Mix 4</th>
<th>Mix 5</th>
<th>Mix 6</th>
<th>Mix 7</th>
<th>Mix 8</th>
<th>Mix 9–13</th>
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</thead>
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<tr>
<td>A</td>
<td>–1</td>
<td>1</td>
<td>–1</td>
<td>1</td>
<td>–α</td>
<td>+α</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>B</td>
<td>–1</td>
<td>–1</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>–α</td>
<td>+α</td>
<td>0</td>
</tr>
<tr>
<td>Mixture component</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Cement kg/m³</td>
<td>321</td>
<td>434</td>
<td>319</td>
<td>430</td>
<td>296</td>
<td>455</td>
<td>380</td>
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<td>Fly ash kg/m³</td>
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<td>106</td>
<td>143</td>
<td>99</td>
<td>152</td>
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<td>Silica fume kg/m³</td>
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<td>35</td>
<td>26</td>
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<td>24</td>
<td>37</td>
<td>31</td>
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<td>Water kg/m³</td>
<td>159</td>
<td>215</td>
<td>158</td>
<td>213</td>
<td>147</td>
<td>225</td>
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<td>186</td>
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<tr>
<td>Coarse aggregate kg/m³</td>
<td>784</td>
<td>784</td>
<td>784</td>
<td>784</td>
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<tr>
<td>Fine aggregate kg/m³</td>
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<td>1064</td>
<td>764</td>
<td>1130</td>
<td>764</td>
<td>702</td>
<td>910</td>
<td>910</td>
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<td>SP dosage %</td>
<td>0.39</td>
<td>0.39</td>
<td>1.11</td>
<td>1.11</td>
<td>0.75</td>
<td>0.75</td>
<td>0.28</td>
<td>1.26</td>
<td>0.75</td>
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</table>

Table 4 Mix designs of Set 2

<table>
<thead>
<tr>
<th>Mixture components</th>
<th>Sample label</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>PC25</td>
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<tr>
<td>Cement kg/m³</td>
<td>368</td>
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<tr>
<td>Fly ash kg/m³</td>
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<tr>
<td>Silica fume kg/m³</td>
<td>55</td>
</tr>
<tr>
<td>Water kg/m³</td>
<td>144</td>
</tr>
<tr>
<td>Coarse aggregate kg/m³</td>
<td>998</td>
</tr>
<tr>
<td>Fine aggregate kg/m³</td>
<td>936</td>
</tr>
<tr>
<td>Superplasticiser kg/m³</td>
<td>7.36</td>
</tr>
<tr>
<td>Paste content %</td>
<td>24.9</td>
</tr>
<tr>
<td>SP dosage %</td>
<td>1.50</td>
</tr>
</tbody>
</table>

Test methods to determine fresh and hydration properties

The flow table test for concrete was used to determine the workability of the concrete tested. The test was conducted according to BS 1881–105:1984 (BS 1881 1984). The flow was reported to the nearest 10 mm.

To obtain an indication of temperature rise during hydration, semi-adiabatic calorimetry was used. A fresh concrete sample was placed in a plastic container and a thermocouple was embedded in the sample. The plastic container was then placed inside a Dewar flask and sealed. The starting temperature of the samples was measured using a high-accuracy laser with a range of 2 mm. The concrete mould was manufactured from aluminium. Teflon was used to line the inside of the concrete mould, ensuring that friction did not hamper the measured deformation. To prevent drying shrinkage, the procedure was conducted at 99% relative humidity. The temperature of the samples was measured while the deformation was monitored. The temperature of the samples did not rise beyond room temperature, which confirmed that the early-age deformation was not influenced by temperature variation. The small sample size allows sufficient heat dissipation to prevent temperature rise.

The fresh concrete was placed in the mould in two layers. Each layer was vibrated to ensure adequate compaction. The thermocouple was embedded during the vibration process. After the concrete had been placed, a reflector was positioned in the centre of the sample. The initial length of the concrete sample was determined with a Vernier caliper. The cone mould with the concrete and reflector was then inserted in the rig on which the laser was mounted. The position of the laser was adjusted so that shrinkage and swelling could be measured. The deformation of the concrete was logged every two minutes for 24 hours.

The total long-term deformation of the CCD concrete was determined using 50x50x300 mm prisms. The effective length of the prisms was 255 mm. The first measurement was taken 24 hours after casting. Subsequent readings were taken after seven days and after 28 days. No initial wet curing was administered. The orientation at which the samples were stored at 55% relative humidity remained constant throughout the experimental programme.

The modulus of elasticity as represented by the secant modulus was determined according to the 5864:2006 (SANS 5864 2006). The beam dimensions were 100x100x500 mm. The supports were 300 mm apart and the loads were applied at two points, 50 mm from the middle of the beam. To determine the split cylinder strength, samples were tested according to ASTM C496/C496M-11:2004 (ASTM C496 2004). The method provides an indirect tensile strength value.

Test methods to determine deformation properties

The total early-age deformation (consisting of settlement and autogenous shrinkage) was determined using the shrinkage cone method proposed by Eppers (2010) and Kaufmann et al. (2004). The size of the cone was adjusted for the measurement of concrete containing 9.5 mm aggregate. The deformation was measured using a high-accuracy laser with a range of 2 mm. The concrete mould was manufactured from aluminium. Teflon was used to line the inside of the cone mould, ensuring that friction did not hamper the measured deformation. To prevent drying shrinkage, the procedure was conducted at 99% relative humidity. The temperature of the samples was measured while the deformation was monitored. The temperature of the samples did not rise beyond room temperature, which confirmed that the early-age deformation was not influenced by temperature variation. The small sample size allows sufficient heat dissipation to prevent temperature rise.

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The maximum temperature was extracted from the temperature rise curve.

Test methods to determine mechanical properties

The compressive strength of the concrete was determined by crushing 100x100x100 mm cubes according to SANS 5863:2006 (SANS 5863 2006). The cubes tested after 24 hours were tested directly after demoulding. The air-cured cubes were tested without submerging them in water before testing. This was done to determine the in-situ strength.

The modulus of rupture (MOR) or flexural strength was tested according to SANS 5864:2006 (SANS 5864 2006). The beam dimensions were 100x100x500 mm. The supports were 300 mm apart and the loads were applied at two points, 50 mm from the middle of the beam. To determine the split cylinder strength, samples were tested according to ASTM C496/C496M-11:2004 (ASTM C496 2004). The method provides an indirect tensile strength value.

The maximum temperature was extracted from the temperature rise curve.
failure load of the concrete. This test method is non-destructive and the same samples were also used to determine the splitting strength.

A variation of ASTM C512/C512M-10:2002 (ASTM C512 2002) was used to measure the load-induced time-dependent compressive strain (or creep) of concrete. Cylindrical samples with a diameter of 80 mm and a length of 300 mm were used. The samples were subjected to 3 MPa, 24 hours after casting. This load was selected to represent environmental loads. The load was maintained for the 90 days that the creep was measured. Two targets were attached 200 mm apart on each side of each sample. An extensometer was used to determine the length change. For each mixture two samples were loaded to measure load-induced deformation, and one sample was used to measure all other deformation (such as drying shrinkage) that the concrete might experience. This value was reported as the total long-term deformation. Creep was calculated by subtracting the long-term deformation of the unloaded samples from the deformation of the loaded samples.

**RESULTS AND DISCUSSION**

**GLM-ANOVA**

The GLM-ANOVA of the tested responses can be seen in Table 5. Only the fresh and some of the early-age properties were affected by the varied parameters. Both paste content and SP dosage have a significant effect on the flow of the HSC. The percentage contribution also shows that both parameters are equally effective in controlling the response. This result confirms that workability can be controlled by paste content and SP dosage.

According to the GLM-ANOVA, paste content controlled the maximum temperature reached under semi-adiabatic conditions. The effect of paste content on the 24-hour compressive strength was less significant than the effect of SP dosage. The paste content influenced the total early-age deformation dominantly, where Holt (2001) found that SP dosage should also influence it significantly. The other properties, i.e. MOR, long-term compressive strength and long-term total deformation in Table 5 were not affected by the paste content in the range that was tested. The GLM-ANOVA also showed that the SP dosage did not affect these properties either, as expected.

**Fresh properties**

Figure 6 shows the response surface of flow. The response surface confirms that both paste content and SP dosage influence the flow of the HSC. The percentage contribution also shows that both parameters are equally effective in controlling the response. This result confirms that workability can be controlled by paste content and SP dosage.

![Figure 6 Contour plot of flow of Set 1](image)

| Table 5 General linear ANOVA for selected measured responses |
|---|---|---|---|---|---|
| **Dependent variable** | **Source of variation** | **DOF** | **Statistical parameters** | **Significant p < 0.05** | **Contribution (%)** |
| **Fresh properties** | N/A | Flow table | Paste content | 1 | 215 500 | 29.24 | 0.0003 | Y | 57.44 |
| | | | SP dosage | 1 | 159 700 | 21.67 | 0.0009 | Y | 42.56 |
| **Hydration properties** | Early-age – 24 hours | Maximum temperature | Paste content | 1 | 44.18 | 31.91 | 0.0002 | Y | 92.27 |
| | | | SP dosage | 1 | 3.7 | 2.67 | 0.1332 | N | 7.73 |
| **Mechanical properties** | Early-age – 24 hours | Compressive strength | Paste content | 1 | 27.96 | 2.16 | 0.1726 | N | 16.09 |
| | | | SP dosage | 1 | 145.84 | 11.26 | 0.0073 | Y | 83.91 |
| | | Modulus of rupture | Paste content | 1 | 0.18 | 0.43 | 0.5266 | N | 12.95 |
| | | | SP dosage | 1 | 1.21 | 2.97 | 0.1156 | N | 87.05 |
| **Deformation properties** | Early-age – 24 hours | Total deformation | Paste content | 1 | 2 806 000 | 101.24 | 0.0001 | Y | 98.85 |
| | | | SP dosage | 1 | 32 644 | 1.18 | 0.3138 | N | 1.149 |
| | Long-term 28 days | Total deformation | Paste content | 1 | 1 294.37 | 0.2 | 0.6646 | N | 82.59 |
| | | | SP dosage | 1 | 272.91 | 0.042 | 0.8416 | N | 17.41 |
workability of HSC to the same extent. When looking at the spacing between the contour lines diagonally, it can be seen that the flow increases faster at lower percentages of the constituents. The improvement of flow flattens out towards the higher paste content and SP dosage.

**Hydration properties**

Figure 7 shows that the maximum temperature reached under semi-adiabatic conditions has a positive correlation to the paste content. The SP dosage seemed to decrease the maximum temperature reached at specific paste content. This could be due to a retardation effect, where the combination of the retardation of heat generation and heat dissipation impedes high maximum temperatures.

The maximum temperature tested for Set 2 reached similar values to that of Set 1 for paste content of 25% to 35% (between 35°C and 45°C). Surprisingly, the higher SF addition percentage (5.7% for Set 1 in comparison to 11.2% for Set 2) did not increase the maximum temperature. Figure 8 shows that temperatures up to 70°C can be reached with paste contents of 60%. The hazardous consequence, in terms of heat of hydration, of significantly increasing the paste content is evident.

**Mechanical properties**

In the range tested for paste content and SP dosage, the compressive strength was influenced more significantly by the SP dosage (as indicated by the GLM-ANOVA). The contour plot showed in Figure 9 shows that the highest early-age strength can be achieved when the lowest paste content and SP dosage is used. The workability of such a concrete mix would be impractically low.

Figure 10 shows the effect of paste content on the compressive strength as well as strength development of concrete. The paste content clearly affected the early-age strength by increasing it at paste content percentages higher than 35%. The range of Set 1 was too small to observe this effect. Stock *et al* (1979) also showed that compressive strength exhibits a minimum in the range between 30% and 50% paste content.

Approximately 40 MPa can be obtained at 60% paste content 24 hours after casting. The compressive strength difference between the respective concretes decreased with time. After three days the benefits of using high paste content for compressive strength are already negligible. At 56 days the compressive strength had a negative correlation to paste content increase, which leads to the conclusion that curing conditions affect the high paste content concretes more
significantly. More severe curing conditions than 99% relative humidity and 25°C would likely have a more detrimental effect.

The statistical analysis of the Set 1 results showed that the MOR is not influenced by SP dosage or paste content in the range that was tested. In Figure 11 the split cylinder strength tested for Set 2 was unaffected by paste content after 24 hours. Paste content did, however, decrease the indirect tensile strength at later ages, making it clear that high paste content can decrease the tensile strength of concrete, leading to crack formation at later ages at lower stresses.

**Deformation properties**

The total early-age deformations include all movements measured with the shrinkage cone method. Although the setup was devised to measure the autogenous shrinkage, it was impossible to determine the transition between plastic and hardened phase. The total early-age deformation includes the settlement and autogenous shrinkage. Note that the positive values signify contraction.

Figure 12 shows that, by increasing the paste content, the total early-age deformation is increased. For a given paste content, an increase in superplasticiser dosage results in a marginal decrease in total early-age deformation. This contradicts the findings by Holt (2005) where the addition of superplasticiser increased the autogenous shrinkage.

In Figure 13 the total early-age deformation also increases as the paste content increases. The magnitude of deformation of Set 1 and Set 2 in the 25% to 35% range was similar between 1 500 microstrain and 2 500 microstrain. At 60% paste content a total early-age deformation of approximately 3 800 microstrain was recorded.

Even though some of the deformation occurs before stress can be induced, these values give an indication of the effect of paste content on the volume stability and cracking potential of the concrete. By minimising the paste content of concrete used in UTCRCP, early-age cracking can be prevented or at least minimised by preventing excessive shrinkage during the first 24 hours. In addition SP dosage may decrease early-age deformation. If early-age deformation in the same order of magnitude were to occur in UTCRCP, significant stresses would be induced if the concrete body is restrained.

The total long-term deformation of Set 1 did not produce a statistically relevant model. After 120 days at 55% RH and 25°C Set 2 contracted up to approximately 350 microstrain for 60% paste content as seen in Figure 14. Note that the deformation reported excludes the early-age deformation. As expected, the paste content increased...
the deformation significantly from 125 microstrain at 25% to 350 microstrain at 60%.

The modulus of elasticity decreased as the paste content increased. Figure 15 shows that this trend remained unchanged, even as the paste gained strength. Paste content had a significant effect on the stiffness of concrete.

As with the shrinkage deformation, the creep strain of Set 2 increased as the paste content increased. Similarly to the modulus of elasticity, the paste in the concrete deforms more than the aggregate. Figure 16 shows that the creep increases as the paste content increases. The creep increased significantly from 63 microstrain for 25% paste content to 420 microstrain for 60% paste content.

The effect of the paste content on the modulus of elasticity and creep of HSC is not necessarily undesirable for concrete pavement applications. A lower modulus of elasticity allows the concrete to deform without generating high stresses and the high creep allows the concrete to relax under stress, thus alleviating stress. Further research is required to determine whether there is an optimum paste content that minimises the cracking tendency.

CONCLUSIONS AND RECOMMENDATIONS

No statistically valid long-term property models could be developed using RSM for the first set of concrete, while Set 2 clearly showed that paste content does affect compressive strength, tensile strength, long-term shrinkage, modulus of elasticity and creep. It was not possible to extend the range of Set 1 over the same range that was used for Set 2, because all concrete mixes cast for Set 1 had to be practical (they must not be too dry to compact or contain such a high SP dosage that the concrete segregates). Set 2 was designed to be practical, and the SP dosage and paste content were varied at the same time.

The volume stability of concrete is severely affected by paste content, and high paste content will contribute to high strains throughout the lifetime of the concrete pavement. By using high paste content, pavements are subjected to high maximum temperatures during hydration, causing the pavement to contract more when it returns to ambient conditions, inducing higher tensile stresses.

Paste content can be used to obtain high early strength if it is increased to more than 35% of the concrete mass. The heat of hydration will be increased by doing this and the higher compressive strength is almost negligible after only three days. High paste content has a detrimental effect on the tensile strength of concrete, which can lead to crack formation at lower strains. In terms of concrete pavement application, paste content does, however, have a beneficial effect where it decreases the modulus of elasticity and increases the creep. The internal stresses in restrained UTCRCP could be reduced in this manner. The overall effect of paste content on the tendency of HSC to crack should be investigated.

Awareness of the side-effects of high paste content is essential. To ensure that UTCRCP does not fail prematurely it would be advisable to minimise the paste content, while maintaining reasonable workability. Ultimately, concrete mixture proportions can be optimised to minimise cracking tendency by determining the effect of mixture constituents on the relevant properties, using RSM and incorporating a mechanistic model to evaluate the cracking tendency.

REFERENCES


Figure 15 Modulus of elasticity of Set 2

Figure 16 Creep of Set 2


Design aspects of concrete towers for wind turbines

C. von der Haar, S. Marx

All over the world an increase in the use of renewable energy sources is being sought, and here the utilisation of wind energy plays an important role. Germany currently represents one of the world’s largest markets for wind energy. At the end of 2013, nearly 24,000 onshore wind turbines with a total output of approximately 34,000 MW had been installed in Germany. Hub heights of up to 140 m and outputs of 3 to 4 MW are now no longer unusual features of new onshore wind turbines.

The focus of this paper is on concrete support structures for wind turbines. Different concrete tower concepts are presented, and the influence of the construction method on the design and verification processes is described. In particular, the text deals with the eigenfrequency analysis, as well as the bearing, shear and torsional resistances of concrete towers. The differences between cast-in-place and precast towers are listed.

INTRODUCTION

The utilisation of renewable energy technologies and the generation of clean energy are worldwide trends. Due to increasing prices and limited supplies of fossil fuels, as well as the rising desire of the population for a sustainable use of natural resources, renewable energy technologies are becoming more economical and more and more important for image-conscious energy companies. Sun, water, wind and biomass are promising sources for the generation of renewable energy. Wind energy in particular is considered an energy source with very high potential. Today wind turbines are being planned and realised at onshore and offshore locations worldwide. New prototypes have a rated power of 10 MW, hub heights of 140 m, and rotor diameters of up to 190 m.

In Germany, the generation of wind energy began 30 to 40 years ago. In the year 1990, the average rated power of a new onshore wind turbine was about 200 kW, and the average hub height was about 30 m. These towers were almost exclusively built as lattice or tubular steel structures. In subsequent years wind towers and rotor diameters increased in size, and thus the rated power of wind turbines also increased rapidly. The advantage of taller wind turbines is that they are exposed to a higher average wind speed and a more constant wind profile over the height of the rotor. The maximum power of a wind turbine can be calculated as per Equation 1, which is derived from the law of the transformation of energy:

\[ P = \frac{1}{2} \cdot \rho \cdot A \cdot v^3 \cdot c_p \]  

(1)

where \( \rho \) is the density of the air, \( A \) is the area of the rotor, \( c_p \) is a power coefficient and \( v \) is the wind speed. Because the wind speed is a third-term factor it is obvious that taller wind turbines have higher efficiency potential based on the cubed wind speed than shorter ones. In the year 2013 the average rated power of new onshore wind turbines was around 2.6 MW. The average hub height of these wind turbines was 118 m, and 33% of the new wind turbines had hub heights of 120 m to 140 m. Therefore, the average rated power of newly installed onshore wind turbines increased thirteen-fold between 1990 and 2013 (Agora 2013; WindGuard 2013; Fraunhofer IWES 2014).

However, consequences of this development are also higher loads and stresses, as well as bigger dimensions of the support structures. The sizes of the larger steel sections, which were originally transported by heavy-load vehicles, started to exceed the road transport limitations, especially the clearance height of bridges, so that concrete towers became a more attractive alternative for the latest wind turbine support structures.

The focus of this paper is on concrete support structures of wind turbines. Different concrete tower concepts are presented, and the influence of the construction method on the design and verification processes is described. The text deals predominantly with German developments and with guidelines and standards for wind turbines with capacities of 3 to 4 MW, which represent the current upper range of new onshore wind turbines.

CONSTRUCTION TYPES

The towers of the first wind turbines built in Germany were almost exclusively constructed as lattice or tubular steel structures, while concrete towers were rarely built. Due to the demand for higher and more powerful wind turbines, hybrid towers, consisting of a lower...
part built in concrete and an upper part
built as a tubular steel section, have in recent
years been shown to be very economical
solutions for multi-megawatt class wind tur-
bines. This is how concrete towers entered
the limelight for designers and manufacturers.
The length ratio of the concrete part and the
steel part mainly depends on the require-
ments of the natural oscillation behaviour of
the tower and on economic aspects, which in
turn depend predominantly on the manufac-
turing and material costs.

The concrete shafts of these hybrid tow-
ers consist of in-situ or precast concrete with
internal or external prestressing.

**Cast-in-place concrete towers**

Cast-in-place towers can be built economi-
cally by using climbing or sliding formwork.
Sliding formwork has the advantage of
allowing a continuous and fast construction
progress. However, the concrete mixture has
to be adjusted carefully to the sliding veloc-
ity and the weather conditions, otherwise the sliding process has a high potential for
causing horizontal cracks in the finished
shaft wall.

Construction with climbing formwork has
the advantage of being done in sections, and
expensive night and weekend shifts are not
required. In Germany climbing formwork by
crane is generally used. The formwork exists
of both an inner and outer shell, which are
adjustable to the required diameters. The
construction process is as follows: At the
construction site the inner and outer form-
work are adjusted to the required diameters
of the tower. Subsequently, the reinforcement
and additional required parts are attached
to the inner formwork. In this construction
step the reinforcing bars that extend to the
next segment protrude above the formwork.
Next, the inner formwork with its reinforce-
ment is lifted by crane to the top of the tower
(Figure 1 left), and is placed in its position
(Figure 1 middle). Then the outer formwork
is lifted to the top of the tower and placed
over the inner formwork and its reinforce-
ment bars, after which the concrete is poured
(Figure 1 right). The formwork should be con-
structed so that it can withstand the weight
of the fresh concrete without requiring any
additional anchors. Anchors are needed only
for attaching the formwork and the working
platforms. The following day the formwork is
removed and the next section is constructed.

Due to the nature of this construction pro-
cess the tower shaft is a monolithic construc-
tion with continuous reinforcement bars. It is
usually externally post-tensioned; therefore no
ducts for tendons have to be installed, which
simplifies the construction process.

**Precast concrete towers**

Prefabricated concrete units are assembled
by crane on top of one another and tied
together with post-tensioning tendons. The
concrete units are manufactured in precast
plants so that high quality and short process-
ing times can be achieved (Figure 2 left). The

The connection types of the horizontal joints of this construction method can be designed as reinforced joints, using horizontal polished joints. The Max Bögl HybridTowers are externally post-tensioned, and no further tensile material or joining element is placed between the concrete elements. This requires special verification methods, which will be presented in the section entitled “Shear and torsional resistance of horizontal joints”.

**Foundation**

The foundation of onshore wind towers are predominantly designed as circular or annular foundations. The geometry of circular foundations is very simple, whereas annular foundations require less material and exhibit higher geotechnical stability. According to DIBt (2012), the permissible gap between the foundation and the soil under different loading conditions is limited. For quasi-permanent load combinations no gap is permitted, and for unfactored extreme loads a maximum gap area of one half of the foundation area is acceptable (plan view).

The improved stability of annular foundations with respect to circular foundations, according to the above-mentioned requirements of DIBT (2012), can be expressed by the following permissible eccentricities: $e_1$, which is defined as the ratio of the maximum bending moment and the normal force without an opening, and $e_2$, which is defined as the ratio of the maximum bending moment and the normal force with an opening up to a maximum of one half of the foundation area.

To explain the eccentricities, a numerical example is presented. The outer radius of the foundation is $r_a = 10$ m, the inner radius varies between $r_i = 0$ m (circular foundation $r' = r/r_a = 0.0$) and $r_i = 6$ m (annular foundation $r' = r/r_a = 0.6$). For the annular foundation, the permissible eccentricity $e_1$ increases from 2.5 m to 3.4 m, and the eccentricity $e_2$ increases from 5.9 m to 6.5 m with respect to the circular foundation (Figure 5). It can be seen that the bending capacity without and with opening of one half of the annular foundation area is 36% and 10% larger, respectively, than that of the circular foundation. However, a more slender annular foundation ($r' = 1$) also results in bigger soil stresses, which limit the slenderness of the annular foundation. The permissible eccentricities $e_1$ and $e_2$ can be calculated with Equations 2 and 3:

$$e_1 = \frac{r_a}{4} \cdot (1 + r^2) \quad (2)$$
e_2 = 0.59 \cdot r_a \cdot \left(1 - r'^4 \right) \left(1 - r'^3 \right) \quad (3)

**EIGENFREQUENCIES**

The extreme and operating loads of a wind turbine are calculated with total dynamic load simulations. In these simulations wind gusts, sudden changes of wind direction, starting, operating and stopping procedures of the turbine, as well as the dynamic behaviour of the whole system consisting of foundation, tower and turbine, are considered. The stiffness of the system has a direct influence on the resulting internal forces. To avoid dynamic amplifications, the eigenfrequencies of the structure should not be within the range of the excitation frequencies of the turbine. The excitation frequencies are:

1. Periodic excitation at the frequency of the rotational speed of the rotor (1P excitation), for example due to imbalances
2. Periodic excitation at the frequency of three times the rotational speed of the rotor by blade passing (3P excitation)
3. Whole-number multiples of the rotor frequency.

The nominal rotor speed of a 3 MW class wind turbine usually lies between 6 and 13 rpm. This means that the rotational frequency of the rotor lies between 0.1 and 0.22 Hz, and the blade passing frequency is three times higher, between 0.3 and 0.65 Hz for three blades. According to DIBt (2012) the excitation frequencies and the eigenfrequencies of the structure should not be within ±5% of each other. Additionally, calculation uncertainties should be considered by adding a safety of ±5% (see DIBt 2012). By this a safety margin of 1.05 × 1.05 = 1.1025 = 10% is achieved. Considering this safety margin, the permissible range of natural frequencies can be shown with the Campbell diagram. According to this diagram, the permissible range of eigenfrequencies lies between 0.24 and 0.27 Hz for the assumed rotor speed of 6 to 13 rpm (Figure 6).

A wind tower design for which the first eigenfrequency lies below the blade passing frequency (3P) and above the rotor frequency (1P) is called "soft-stiff". A design where the eigenfrequency of the structure lies above the blade passing frequency (3P) is called “stiff-stiff”. Such stiff designs, however, are uneconomical and require large quantities of material. If the first eigenfrequency is lower than the rotor frequency (1P), the design is called “soft-soft”. With this type of design very slender support structures can be created, for which the fatigue resistance has to be checked very carefully (see Grünberg & Göhlmann 2013).

**Influence of the geometry and the material on the eigenfrequency**

The height of the tower and the weight of the turbine are set by the turbine manufacturer; therefore designers only have limited options for shifting the eigenfrequencies of the structure into the permissible range defined by the Campbell diagram. Today most simulation software can determine the eigenfrequencies of a wind turbine with high reliability. Alternative calculations based on
Figure 7 First eigenfrequency of the tower in dependence of the bottom diameter and the shaft thickness

energy methods also yield good results (see Grünberg & Göhlmann 2013).

The main attribute influencing the eigenfrequency of the tower is its diameter. The stiffness of the tower, and hence the tower frequency, increase with its diameter. Eigenfrequency analyses for different tower diameters were performed for a wind tower with a hub height of 140 m and a turbine mass of 300 tons. The outer diameter at the top of the tower was chosen to be 3.0 m; this dimension is usually defined by the geometry of the nacelle. The outer diameter at 90 m above the ground was assumed to be 4.5 m to guarantee blade passing. The diameter at the base of the tower was varied between D = 8.0 m and D = 14.0 m. The diameter was assumed to decrease linearly between the given points (Figure 7 right). The results are based on concrete of strength class C60/75 according to EN 1992-1-1 (2010) with an elastic modulus of $E_{cm} = 39,100 \text{ N/mm}^2$. The adopted specific weight of the concrete was $\gamma = 25 \text{ kN/m}^3$, and it was not increased to consider additional installation parts such as leaders and so on. The thickness of the shaft wall in these analyses was varied between $t = 0.20 \text{ m}$ and $t = 0.40 \text{ m}$. As depicted in Figure 7 and mentioned before, the diameter has a large influence on the eigenfrequency of the tower. The shaft thickness influences the eigenfrequency as well, but the effect is not as dominant, because the stiffness increase due to a larger shaft thickness is offset by the resulting higher mass of the tower.

The elastic modulus of $E_{cm} = 39,100 \text{ N/mm}^2$ is only a mean value for concrete of the chosen strength. The actual value can vary depending on the used mineral aggregate, the concrete composition, as well as how and how long the concrete is cured. It is recommended to test the elastic modulus of the used concrete in the preliminary design stages to get realistic values for the calculations. Alternatively the eigenfrequency analysis should be performed for lower and upper values of the elastic modulus of the concrete.

In this context the cracking of the structure and the resulting stiffness reduction of the tower also have to be considered. But the dynamic amplification due to wind actions and dynamic interactions with the rotor, and the blade excitation frequencies are particularly important for operating conditions, and therefore also for fatigue loads and frequent load cases. It follows that the prestressing of the tower should be designed so that no cracks can form even for those load cases.

The elastic modulus $E_{cm}$ according to EN 1992-1-1 (2010) is generally defined as the secant modulus of the concrete. Its intersection point with the stress-strain curve is at 40% of the concrete strength. The compressive stress of the concrete is limited to 60% of the concrete strength under rare load combinations according to DIBt (2012). Therefore, the stresses caused by frequent actions are in the defining range of the modulus of elasticity; hence it can be used for eigenfrequency analyses without any further adjustments.

Influence of the foundation

In the previous calculation the base of the tower was fixed against rotational, horizontal and vertical displacements. This assumption is incorrect; the stiffness of the foundation and the soil have to be considered in the eigenfrequency analyses. The guideline DIBt (2012) refers to DGGE (2002) for geotechnical issues. According to this publication, the rotational, horizontal and vertical stiffness can be calculated using Equations 4 to 7 for circular foundations:

vertical stiffness $k_z = \frac{4 \cdot G_d \cdot r}{1 - \nu}$ (4)

horizontal stiffness $k_y = k_y = \frac{8 \cdot G_d \cdot r}{2 - \nu}$ (5)

rotational stiffness $k_{\phi y} = k_{\phi y} = \frac{8 \cdot G_d \cdot r^3}{3 \cdot (1 - \nu)}$ (6)

torsional stiffness $k_{\phi z} = \frac{16 \cdot G_d \cdot r^3}{3}$ (7)

where $G_d$ is the dynamic shear modulus of the soil, $r$ is the radius of the foundation and $\nu$ is the Poisson’s ratio of the soil. In DGGE (2002) ranges of values for these parameters in dependence of the soil conditions are given.

BEARING CAPACITY AND SECOND ORDER THEORY

The design process for concrete towers generally follows relevant design codes such as EN 1992-1-1 (2010). The internal forces in the tower are usually calculated by the turbine manufacturer using total dynamic simulations. These calculations are typically performed as
geometric nonlinear calculations that assume the concrete to behave in a linearly elastic manner. This means that the nonlinear behaviour of the concrete and the stiffness reduction due to crack formation have to be considered separately, especially for the ultimate limit state. Considering these issues yields higher deformations and additional second order bending moments. The joints of the precast towers also influence the deformations and additional bending moments, as will be shown in the following paragraphs.

Crack formation and the physical nonlinearity of the concrete can be considered in bending moment-curvature relationships, or M-κ-curves. The M-κ-curves for annular cross sections can be calculated according to the approach presented by Grünberg and Göhlmann (2013).

Figure 8 shows the M-κ-curves for an annular cross section. The outer diameter is 12 m and the shaft thickness is 30 cm. A

![Figure 8 Bending moment-curvature relationship](image)

![Figure 9 Comparison of a monolithic and a precast wind tower](image)
concrete of strength class C60/75 according to EN 1992-1-1 (2010) is assumed. The calculations are performed for a normal force of $N = -30$ MN (dashed lines) and $N = -10$ MN (continuous lines). The reinforcement ratio $\rho$, which is defined as the ratio of the cross-sectional area of the reinforcement and the cross-sectional area of the concrete, varies between 0%, 0.7% and 1.3%. A ratio of $\rho = 1.3\%$ corresponds to a reinforcement ratio of 39 cm$^2$/m respectively, being 19.5 cm$^2$/m in the outer and inner sides of the shaft wall. A value of $\rho = 0\%$ represents the joint of a precast tower with external post-tensioning. Tension stiffening is neglected in these calculations. This assumes the design of concrete towers on the safe side. The $M-\kappa$-curves of the reinforced cross sections (blue and red lines) show two characteristic kinks in the curve, the first of which represents the transition from the uncracked to the cracked state. The second kink represents the point where first reinforcement bars start yielding. The kinks are less present than those of rectangular cross sections with only one layer of reinforcement. The reasons for this are the annular shape and the uniformly distributed reinforcement around the circumference. The diagram shows that the bending resistance of a reinforced cross section increases with the reinforcement ratio and the compressive force for a given curvature. The unreinforced cross sections (green lines), which represent the joints between two precast elements, show only one change of slope at the point where a gap between the elements opens. Beyond this point the bending capacity is nearly constant. The bending capacity is solely dependent on the compressive force.

With the $M-\kappa$-curves and the internal bending moments the curvature $\kappa$ over the tower height can be determined. By integrating the curvature $\kappa$ the rotation $\phi$ is obtained, and by integrating the rotation $\phi$ the deflection $w$ of the concrete tower under consideration of crack formation and stiffness reduction can be calculated. The deflection $w$ multiplied with the vertical loads over the tower height results in additional second order bending moments. Figure 9 shows the results for a 140 m high prestressed concrete tower as shown in Figure 7 (right). The calculations are performed for a continuously reinforced monolithic tower and a precast tower with a vertical joint distance of 5 m. The chosen concrete strength class is C60/75, and the tower is prestressed with 28 tendons with a prestressing force of approximately 3 MN each. At the beginning of the calculation the $M-\kappa$-curves have to be determined under consideration of varying dimensions, normal forces and reinforcement ratios over the tower height. Based on these curves and a given bending moment according to first order theory, the curvature over tower height is obtained. By double integration of the curvature the deflection of the tower is determined. The additional second order bending moment is determined by multiplying the vertical loads and the deflections. The following iteration step starts again with the determination of the curvature for the sum of bending moment according to first order theory and the additional second order bending moment. The iteration is done until the deformations converge.

The curvature of the monolithic tower shows a continuous progression along the entire hub height. In contrast the curvature of the precast tower shows a bump at each horizontal joint. The bumps are a result of the lower bending resistance at each joint with respect to the reinforced elements (see Figure 8). This leads to additional rotations and as a consequence to larger deflections and a bigger bending moment. The deflection at the top of the tower increases from 3.12 m for the monolithic tower to 3.89 m for the precast tower.

Precast towers with external post-tensioning exhibit a lower bending resistance in the horizontal joints than continuously reinforced monolithic towers or precast towers with internal post-tensioning. Therefore, the applied post-tensioning forces are usually higher for this type of tower.

For towers with external post-tensioning the eccentricity between the deformed shaft wall and the tendons introduces additional destabilising bending moments, which have to be considered in the calculations. These destabilising bending moments can be reduced by installing corbels on the inside of the tower, which can be positioned at the third points of the tower height as depicted in Figure 10.

**Figure 10 Destabilising effect of external post-tensioning according to Grünberg and Göhlmann (2013)**

**Shear and Torsional Resistance of Horizontal Joints**

The different construction methods for the horizontal joints must be treated differently when verifying the designs. The joint of the Enercon tower, for example, can be treated like a horizontal crack according to the relevant design codes. The shear forces and torsional moments can be determined with the widely known truss models suggested in EN 1992-1-1 (2010).

The joints of the Max Bögl tower are unreinforced joints so that the shear forces and torsional moments have to be transferred between the elements by friction. For design verification it is important to distinguish between joints that are completely compressed and opening joints. If the joints are completely compressed a Bredt shear flow can be assumed. If they are opened...
the torsional capacity must be determined according to the approach by St Venant (Wriggers et al. 2005). As both approaches are only valid for thin-walled structures, this assumption is not correct for concrete towers. Additionally, the approach by St Venant assumes free warping of the cross section. Further investigations are required to solve these problems.

For completely compressed joints the action effects in the joints in the circumferential direction can be determined as per Equations 8 and 9:

$$
u_{Ed} = \frac{V_{Ed}}{\pi \cdot r_m}$$

(8)

$$\tau_{Ed} = -\frac{T_{Ed}}{2 \cdot \pi \cdot r_m^2}$$

(9)

The listed variables are defined in Figure 11. The shear resistance of a joint can be calculated according to EN 1992-1-1 (2010):

$$\nu_{Rd} = (\sigma_{ad} \cdot f_{adm} + \mu \cdot \sigma_{Nd}) \cdot \frac{t}{\tau_{Ed}}$$

(10)

The adhesive bond must be neglected for non-grouted joints and for dynamic actions. The frictional part depends on the frictional coefficient $\mu$ and the mean compressive stress $\sigma_{Nd}$.

The design check can be performed using Equation 11:

$$\nu_{Ed} + t_{Ed} \leq \nu_{Rd}$$

(11)

Neither the DIBt (2012) guideline nor the EN 1992-1-1 (2010) regulates the opening of joints. But, as mentioned before, crack formation of the reinforced elements should be prevented under fatigue loads and frequent load combinations. According to this the joints also should not open under these load combinations. Therefore, the opening of the joints and any associated reduction in stiffness must not be considered for the eigenfrequency analyses and load simulations.

Under higher load cases, and especially the ultimate limit state, an opening has to be considered. The resultant shear force that can be accommodated by the area of the flexural compression zone is:

$$V_{Rd,ct} = \mu \cdot F_{Nd}$$

(12)

The torsional moment must be resisted solely by the St Venant torsional resistance:

$$T_{Rd} = \mu \cdot F_{Nd} \cdot \frac{t}{3}$$

(13)

Referring to EN 1992-1-1 (2010) paragraph 6.3.2(5) the following linear interaction can be assumed for the combined analysis of the ultimate capacity:

$$\frac{V_{Ed}}{V_{Rd,ct}} + \frac{T_{Ed}}{T_{Rd}} \leq 1$$

(14)

CONCLUSION

The focus of this paper is on concrete support structures of wind turbines. Different concrete tower concepts were presented, and the impact of different construction methods on the design and verification process was explained. This article also deals with eigenfrequency analysis, and especially the bearing, shear and torsional resistances of the joints of precast concrete towers.

Precast concrete towers should be designed so that no cracks can form under fatigue loads and frequent load cases. Therefore, any associated reduction in stiffness must not be considered for the eigenfrequency analyses and load simulations. For higher load cases, and especially for the ultimate limit state, crack formation and the resulting stiffness reduction of the tower have to be considered in the design process. They lead to additional deformations and second order bending moments.

The advantage of precast concrete towers is a fast construction process on site. But the rotational stiffness at each horizontal joint is lower with respect to the reinforced concrete elements. This leads to additional rotations, larger deflections and bigger bending moments according to second order theory.

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Dynamic behaviour of normally reinforced concrete wind turbine support structures

W S van Zyl, G P A G van Zijl

Transportation logistics of large steel towers have led to concrete towers becoming a viable option. There are currently no design codes dealing exclusively with the design of concrete wind turbine towers. Wind turbine towers have strict constraints on the fundamental frequency of the tower to avoid resonance. This paper investigates the dynamic behaviour of wind turbine towers using nonlinear finite element modelling. Focus is placed on the effect of crack formation and soil stiffness on the fundamental frequency of the tower. An analytical model is then proposed that can be used in the primary design stage to determine the geometry of the tower that satisfies the fundamental frequency requirements.

INTRODUCTION

Modern civilisation is dependent on energy. The global energy demand is steadily rising each year and is accompanied by rising greenhouse gas emissions. Global warming is a real danger, and governments worldwide are implementing goals to generate large percentages of their countries’ energy needs with renewable energy. The Global Wind Energy Council (GWEC) predicts that by 2035 renewable energy will be generating more than 25% of the world’s electricity needs, with a quarter of this coming from wind energy (GWEC 2013). Wind energy is currently the second largest renewable energy source, after hydro power, and it has been growing exponentially over the last decade.

South Africa (SA) has the seventh largest coal reserve in the world. In 2012, 72% of the country’s electricity was being produced by coal, 24% by oil and natural gas, 3% by nuclear, and less than 1% by renewable energy. This dependence on hydrocarbons, particularly coal, has made SA the twelfth largest CO2 emitter in the world (GWEC 2013).

The Renewable Energy Independent Power Producer Procurement Programme (REIPP PPP) was created to encourage the exploitation of SA’s vast renewable energy resources. SA’s long-term energy blueprint, the Integrated Resource Plan (IRP), specifies that about 9 000 MW of electricity must be produced by wind energy by 2030. The first phase of 634 MW of wind energy is currently in operation and is connected to the national electricity grid. The second phase of 562 MW is currently under construction, and the third and fourth phases are still in the bidding phase and will have a generating capacity of 787 MW and 590 MW respectively.

Concrete support structures

The worldwide movement to generate large amounts of electricity with wind turbines has led to a significant increase in the generating capacity of wind turbines. The capacity of turbines has increased from a couple of kilowatts in the 1970s to anything between 2 000 and 7 500 kilowatt today.

Modern turbines require higher support structures, as higher wind speeds, combined with longer blades, are necessary to increase their generating capacity. The standard 80–90 m tower is thus not economically viable anymore. The base diameter of a steel tower is limited to approximately 4.2 m due to transportation logistics. The diameter limitations make steel towers uneconomical at hub heights greater than approximately 80–90 m. One solution to the height limitation is to use concrete towers. Concrete wind turbine towers can either be precast and assembled on site, or slip-formed. This has the advantage that the tower segments can either be made small enough to be transported by normal truck or the tower can be produced on site. This overcomes the limitation on the diameter of the tower, and therefore there is no limitation on the height that is achievable.

There are currently no structural design codes that specifically give design guidelines for the design of concrete wind turbine towers. This has resulted in a handful of companies worldwide, with the knowledge...
to design these towers, having a monopoly in the industry.

The overall objective of this paper is to propose a design procedure for a concrete wind turbine tower. A nonlinear finite element model (FEM) is used as a design tool to evaluate the tower. Focus is placed on determining the appropriate wind models and wind loads to accurately model the tower. The FEM is then used to study the behaviour of the tower under different loading conditions to determine the critical design load case. The effect that crack formation has on the stiffness and dynamic behaviour of the tower is studied, and a sensitivity analysis is done to determine the effect of soil stiffness on the fundamental frequency of the tower. Different structural design codes are then used to create an analytical design method that can be used in the preliminary design stage, and that in certain cases may even be appropriate to use in the final design stage. The analytical design method is compared to the FEM to determine its accuracy.

This paper only focuses on the dynamic behaviour of the structure and investigates the effect of crack formation and soil stiffness on the fundamental frequency of the tower.

**Dynamic behaviour of wind turbine towers**

The fundamental frequency of structures exposed to dynamic loading is of vital importance to avoid resonance. Resonance occurs when an external dynamic force is applied to a structure at the same frequency as the structure’s natural frequency. This causes the structure to undergo large displacements and can cause immediate failure or fatigue failure over time. There are mainly two methods to ensure that a structure is dynamically safe. The first is to ensure that the fundamental frequency of the structure does not coincide with any external vibration frequency that the structure may experience in its life. The second method uses damping to decrease the dynamic amplification of an external vibration force.

In the wind turbine tower industry, method one is preferred. Wind turbine structures are exposed to multiple excitation frequencies that can cause the structure to vibrate. The most important of these excitation frequencies is the blade rotational frequency, known as the 1P frequency, and the blade passing frequency, known as the 3P frequency. The 1P, or rotor revolution frequency, is caused by the unbalanced weight of the rotor, wind shear and tower shadow (DNV & Riso 2002). Modern wind turbines are variable speed turbines, thus the 1P and 3P frequencies are not single frequencies but a frequency interval. There are three different design options. The first is to design a structure with a fundamental frequency higher than the 3P frequency interval, called a stiff-stiff structure. The second option is to design a structure with fundamental frequency between the 1P and 3P frequency intervals, called a soft-stiff structure. The third option is to design a structure with fundamental frequency below the 1P frequency known as a soft-soft structure. This concept is illustrated in Figure 1. It has been shown that a soft-stiff structure is the most economical for wind turbine towers (Harte & Van Zijl 2007). It is difficult to calculate the exact fundamental frequency of a structure at the design stage, as there are various factors that may influence the frequency. Due to this uncertainty, the frequency of the tower is kept out of ±10% of the 1P and 3P frequency intervals (DNV & Riso 2002). The frequency between the 1.1P and 2.7P frequencies, is defined as the working frequency.

It is estimated that, by assuming the foundation to be fully fixed, the error in the fundamental frequency of the tower can be up to 20% (DNV & Riso 2002). Wind turbine guidelines generally propose that elastic springs be used to simulate the soil stiffness.

**FINITE ELEMENT MODEL (FEM)**

In this paper, a concrete wind turbine tower is modelled in the finite element package Diana (2012).

**Geometry**

A typical wind turbine is illustrated in Figure 2. The tower has a conical profile, with the diameter and wall thickness reducing with height. This is due to the fact that the bending moment and shear force are at a maximum at foundation level and then reduce to the top. Reducing the diameter and wall thickness saves unnecessary weight and costs.

**Material**

Internationally high-strength concrete (HSC) is commonly used in high-rise buildings because of its ability to reduce the size and weight of structural elements. In recent years, HSC has also been used in some buildings in SA. The high compression strength and high stiffness of HSC make it ideal for concrete wind turbine towers.

The physical properties of the concrete are calculated using the Model Code (FIB 2010). The Model Code forms the basis of many of today’s concrete design codes, including the Eurocode. The 2010 edition of the Model Code includes material properties for HSC. The properties from the code for a
class C80/95 concrete used here, are given in Table 1.

Diana offers various predefined finite element material models that describe the material behaviour under specific loading conditions. The material behaviour of the concrete is divided into concrete failing in compression (crushing) and concrete failing in tension (cracking).

A plasticity model is used to describe the concrete's behaviour in compression. When comparing elastic and plastic material behaviour, the main difference is that an elastic material will undergo no permanent deformation and a plastic material will undergo permanent or irreversible deformations.

A total strain-based smeared cracking model is used to model the crack behaviour of concrete in Diana (Feenstra et al 1991). Whilst alternative models in continuum plasticity or damage could be used, this model has been shown to be robust and reasonably accurate, and allows nonlinear compressive and tensile behaviour of concrete to be described separately from typical concrete characterisation test data. As the name suggests, the crack width is smeared out or averaged over an element. This is different to a discrete cracking model where two or more elements lose contact with each other and a physical gap occurs. The total strain of the smeared cracking model is decomposed into an elastic strain component and a crack strain component:

\[ \varepsilon_{\text{total}} = \varepsilon_{\text{elastic}} + \varepsilon_{\text{crack}} \]  

A crack is formed when the principal tensile stress violates the maximum tensile strength condition. The tensile strength of the material can then either be reduced to zero immediately, or it can gradually decrease to zero – the latter is known as tension softening and is governed by fracture energy. A rotating crack model that reorients the crack direction, so that it will always coincide with the principal stress direction, is used. If the stress in a crack is reversed, the crack will close and the material retains its full compressive strength. The tensile strength, however, is lost and the crack will reopen once tensile strains reoccur.

### Mesh and element type

Eight node curved shell elements are used to model the tower structure. A normal curved shell element has five degrees of freedom in every element node, three translations and two rotations. Thus, the basic variables of the curved shell elements are the translations \( u_x \) and \( u_y \) in the global XYZ directions, and the rotations \( \phi_x \) and \( \phi_y \) respectively around the local x and y axes in the tangent plane.

Reinforcing bar elements are used to model the reinforcing steel in the concrete. Reinforcing elements do not have their own degrees of freedom. When embedded, the displacements and strains of the reinforcing element are fully coupled to that of the element in which it is embedded. The reinforcing element adds stiffness to the elements in which it is embedded. A bar element only adds stiffness in the axial direction of the bar.

### Loads

The loading on wind turbine structures is unique when compared to normal concrete structures. Structural engineers are used to working with building structures of which the loading is dominated by the structure’s own weight and live loads, such as people and moveable loads in the building. Provision for wind loads is usually made by incorporating shear walls into the building frame. What makes wind turbine towers unique, is that the live loads on these types of structures are negligibly small. Instead, the tower’s loading is dominated by wind loading and the own weight of the structure is actually beneficial for resisting the wind loads. The wind loads acting on the tower are translated into direct wind pressure on the tower and turbine loads.

The direct wind pressure acting on the tower causes a positive pressure on the windward side and a negative pressure on the sides and back of the tower. This is illustrated in Figure 3. Wind pressure on the blades, hub and nacelle (generator housing) is transferred to the top of the tower. These forces are known as turbine loads and are the largest loads imposed on the tower.

### Foundation stiffness effect

The foundation is a vital part of a wind turbine tower design. The foundation cannot be assumed to be fixed when the fundamental frequency of the tower is computed. Studies have shown that the effect of stiffness of the foundation itself on the fundamental frequency of the tower is small when compared to the effect of the soil stiffness (DNV & Riso 2002).

The soil stiffness and soil-foundation interaction can be modelled in detail through the use of soil elements and interface elements respectively, but this is a time-consuming and computationally expensive procedure. Another method that is commonly used involves the use of linear springs to represent the soil stiffness. This is a simple and cost-effective method to simulate the effect of the soil stiffness on the dynamic behaviour of the tower. The soil stiffness is uncoupled into a vertical, horizontal, rotational and torsional stiffness component. The foundation itself is then assumed to be rigid and supported on the appropriate springs. One of the most commonly used models for representing the stiffness of the soil through linear springs, is the method described by George Gazetas in his paper on machine foundation vibrations in 1983 (Gazetas 1983). The equations for calculating the soil stiffness of a foundation on a homogeneous half space is given in Table 2.
The rocking motion is the dominant mode for the tower. The foundation is thus supported on vertical springs that give the same rocking stiffness as the stiffness calculated by Gazetas’s method. A sensitivity analysis is done to determine the effect of soil stiffness on the fundamental frequency of the tower. Typical soil types and their properties are given in Table 3. These generic soil types are used for the sensitivity analysis.

**FEM analysis**

The FEM is analysed for structural strength using a static non-linear analysis. The analysis is non-linear in terms of geometry and material properties, i.e. geometrical and physical nonlinearity is considered.

The natural frequency of the tower is computed using an Eigen value analysis. The Eigen value analysis is first computed using the uncracked tower. This step may not be required by all FE software. In Diana this initiates matrices for subsequent calculation of frequencies in the cracked state. It is also interesting to have knowledge of these frequencies in the uncracked state. A static non-linear analysis is then used to determine the tangent stiffness of the cracked tower, and finally the tangent stiffness of the tower is used to compute the natural frequency of the tower in the cracked state. The FEM model used, is schematised in Figure 3.

**ANALYTICAL DESIGN METHODS**

The finite element method is an excellent method for designing complex structures, but it can be a time-consuming and expensive method. The results of a finite element analysis can also often be difficult to analyse, as the method produces a vast amount of data. Analytical design methods can sometimes produce accurate results in a time- and cost-effective manner.

**Energy methods**

There are many analytical methods for calculating a structure’s fundamental frequency, but very few of these methods can incorporate a varying section area, varying stiffness and lumped mass all together. Energy methods are of the few methods that can incorporate all the varying properties of the tower to calculate the fundamental frequency. The principle of conservation of energy forms the basis of all energy methods. The principle states that the total energy of a closed system will stay constant in the absence of losses such as friction, damping, etc. In practice there will always be energy losses, but these losses are negligibly small in many cases and thus an accurate answer can still be achieved by neglecting them (LaNier 2004). The total...
energy of a closed vibrating system can be categorised into potential energy \( (P_i) \) and kinetic energy \( (K_e) \) and has the following property in time \( (t) \):
\[
d\left(K_e + P_i\right) = 0 \tag{2}
\]

**Rayleigh’s method for rigid foundations**

Rayleigh’s energy method uses the conservation of energy principle to calculate the fundamental frequency of a structure. The kinetic energy of the system is the energy of the vibrating motion and is calculated from the velocity. The potential energy is given by the strain energy of the tower. The first step in Rayleigh’s method is to assume a displacement function for the structure. The displacement function can then be used to calculate the kinetic and potential (strain) energy. The accuracy of the assumed deflection function depends on the accuracy of the assumed deflection function. A generic equation for the fundamental frequency is:
\[
\omega^2 = \frac{\int_0^h E(x) I(x) |\dot{Y}(x)|^2 \, dx}{\int_0^h E(x) I(x) |\ddot{Y}(x)|^2 \, dx} \tag{8}
\]

It is important to note that Rayleigh’s method yields an upper-bound solution for the fundamental frequency of the tower. The assumed displacement function introduces additional constraints, which increase the stiffness of the system and thus leads to a slightly higher fundamental frequency than that of the real structure. There are various deflection curves available for cantilever beams – two of the common curves are given below (Buchholdt & Nejad 2012):
\[
F_1(x) = a \left[ 1 - \cos \left( \frac{nx}{2h} \right) \right] \tag{9}
\]
\[
F_2(x) = a \left[ \frac{3x^2}{2h^2} - \frac{1}{2} \right] \tag{10}
\]

where \( a \) is a constant describing the maximum deflection of the tower; \( a \) is left as a constant throughout the frequency calculations.

**Rayleigh’s method for flexible foundations**

Rayleigh’s method discussed above does not make provision for the flexibility of the soil under the foundation. This effect can be accounted for by using a method proposed by Berger-Abam Engineers (LaNier 2004). The method involves separating the vibration frequencies into a rigid body base rotation frequency, rigid body base translation frequency and the tower flexure frequency. The different frequencies can then be combined with the following equation:
\[
\frac{1}{f^2} = \frac{1}{f_{rot}^2} + \frac{1}{f_{trans}^2} + \frac{1}{f_{flex}^2} \tag{11}
\]

where
\[
f = \sqrt{f_{rot}^2 f_{trans}^2 + f_{rot}^2 f_{flex}^2 + f_{trans}^2 f_{flex}^2} \tag{17}
\]

**RESULTS AND DISCUSSION**

**Rigid foundation**

The first ten mode shapes for the tower obtained from the finite element analysis of the tower with a rigid foundation, are shown in Figure 4. The soil stiffness is not taken into account for this model and the tower is fixed at its base. The tower is symmetrical around the XY and ZY planes and thus certain mode shapes will occur twice at almost the same frequency.
The accuracy of Rayleigh’s method and its sensitivity to the assumed deflection curve are calculated by comparing the fundamental frequency obtained by both approximate deflection curves (Equations 9 and 10) and the static deflection curve to the frequency obtained from the FEM. The static deflection curve refers to the normalised deflection obtained from the FEM under static wind loading. The results are given in Table 4.

It is interesting that the approximate function \( F_1 \) gives a more accurate answer than the static deflection function. All the functions do, however, approximate the frequency computed with the FEM within 5%, which in most cases will be sufficiently accurate.

**Rigid foundation with concrete cracked**

The formation of cracks in concrete reduces the stiffness of the concrete in tension. This can be captured numerically by performing an eigenvalue analysis once the cracks have formed (Diana 9.4.4 2012). The results of the Eigen value analysis, done while the serviceability limit state (SLS) wind loads are acting on the tower, are shown in Table 5.

From the results it is clear that the fundamental frequency of the tower is significantly reduced by the formation of cracks in the concrete. The reduction causes the frequency to fall outside the working frequency of the tower and will thus cause the tower to resonate. It is interesting to note that the first mode frequency is reduced much more than the second mode frequency, although they have the same mode shape. The reason for this is that the first mode coincides with the deflection direction of the tower in the SLS. The second mode is perpendicular to the applied wind load. This causes less concrete to be cracked in the tension zone of mode two, thus increasing the stiffness in this direction.

**Flexible foundation using FEM**

The FEM with foundation that is schematised in Figure 4 is used to determine the mode frequencies of the tower supported on different soil types. The spring stiffness is changed according to the specific soil type being modelled. The results for various soil types are given in Table 6.

The percentage reduction in frequency given in Table 6 is quite severe. Wind turbine foundations are, however, not constructed on untreated soil. Extensive soil preparation is done before the foundation is constructed. The preparation can include various base layers, compacting techniques and even pile foundations if the soil stiffness is still undesirable. The analysis does, however, emphasise the importance of a detailed geotechnical survey to determine the soil stiffness, as an overestimation of the soil stiffness may cause severe vibrations and even resonance of the structure. As an alternative, if it is known that the foundation will influence the frequency, the tower’s frequency has to be adjusted so that the combined frequency falls within the allowable operating frequency range.

**Flexible foundation using Rayleigh’s method**

Rayleigh’s method for flexible foundations is compared to the uncracked FEM results to determine the accuracy of this method. The results obtained by the modified Rayleigh’s method for different soil types are given in Table 7.

![Figure 4 Mode shapes of tower with rigid foundation](image)

### Table 4 Comparison of Rayleigh’s method to FEM

<table>
<thead>
<tr>
<th></th>
<th>Fundamental frequency (Hz)</th>
<th>Percentage error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FEM</td>
<td>0.445</td>
<td>–</td>
</tr>
<tr>
<td>Static(^1)</td>
<td>0.424</td>
<td>–4.72</td>
</tr>
<tr>
<td>( F_1 ) (Eq 9)</td>
<td>0.454</td>
<td>2.02</td>
</tr>
<tr>
<td>( F_2 ) (Eq 10)</td>
<td>0.463</td>
<td>4.04</td>
</tr>
</tbody>
</table>

\(^1\) Deflection curve determined by finite element analysis under static wind load

### Table 5 SLS FEM tower mode frequencies with concrete cracked

<table>
<thead>
<tr>
<th>Mode shape</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uncracked</td>
<td>0.445</td>
<td>0.446</td>
<td>2.418</td>
<td>2.423</td>
<td>6.490</td>
</tr>
<tr>
<td>SLS (Hz)</td>
<td>0.241</td>
<td>0.325</td>
<td>1.460</td>
<td>1.882</td>
<td>4.246</td>
</tr>
<tr>
<td>Reduction</td>
<td>45.84</td>
<td>27.13</td>
<td>39.62</td>
<td>22.33</td>
<td>34.58</td>
</tr>
</tbody>
</table>

### Table 6 FEM results for generic soil types

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Fundamental frequency (Hz)</th>
<th>% Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft clay</td>
<td>0.445</td>
<td>0.27283</td>
</tr>
<tr>
<td>Clay</td>
<td>0.445</td>
<td>0.32035</td>
</tr>
<tr>
<td>Fine sand</td>
<td>0.445</td>
<td>0.34268</td>
</tr>
<tr>
<td>Sand</td>
<td>0.445</td>
<td>0.36129</td>
</tr>
<tr>
<td>Coarse sand</td>
<td>0.445</td>
<td>0.36694</td>
</tr>
<tr>
<td>Gravel</td>
<td>0.445</td>
<td>0.37835</td>
</tr>
</tbody>
</table>
Table 7 Fundamental frequency using Rayleigh’s method

<table>
<thead>
<tr>
<th>Soil type</th>
<th>FEM</th>
<th>Static</th>
<th>Percentage error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft clay</td>
<td>0.273</td>
<td>0.279</td>
<td>2.38</td>
</tr>
<tr>
<td>Clay</td>
<td>0.320</td>
<td>0.330</td>
<td>2.93</td>
</tr>
<tr>
<td>Fine sand</td>
<td>0.343</td>
<td>0.356</td>
<td>3.96</td>
</tr>
<tr>
<td>Sand</td>
<td>0.361</td>
<td>0.376</td>
<td>4.12</td>
</tr>
<tr>
<td>Coarse sand</td>
<td>0.367</td>
<td>0.382</td>
<td>4.18</td>
</tr>
<tr>
<td>Gravel</td>
<td>0.378</td>
<td>0.395</td>
<td>4.31</td>
</tr>
</tbody>
</table>

1 See section dealing with “Foundation stiffness effect”
2 Deflection curve determined by finite element analysis under static wind load

CONCLUSION AND FUTURE RESEARCH

Concrete wind turbine towers play a vital role in ensuring the continual development of large-scale wind turbines. The lack of knowledge on the design of concrete wind turbine towers in SA gave rise to this research project. The goal of this paper was to investigate the dynamic behaviour of large concrete wind turbine towers and to highlight important factors influencing the dynamic behaviour of the tower.

The sensitivity analysis done on the fundamental frequency of the tower, emphasised the importance of modelling the actual structure and boundary conditions as accurately as possible when carrying out an eigenvalue analysis.

The fundamental frequency of the example tower is reduced by 46% after the tower has cracked. This reduction leads to the tower’s frequency falling outside the working frequency of the turbine. It may be necessary to increase the percentage reinforcing to reduce cracking and ultimately reduce the stiffness reduction caused by cracking. It is important to note that the reduction in fundamental frequency is strongly dependent on the specific tower being modelled. Differences in height, reinforcing layout and level of loading may all affect the reduction in fundamental frequency.

The soil stiffness has a significant influence on the tower’s natural frequency. The preparation of the soil under the foundation will thus strongly influence the dynamic behaviour of the tower. The soil sensitivity analysis highlighted the importance of a comprehensive geotechnical survey, as both understimation and overestimation of the soil stiffness may lead to structural failure due to resonance.

Rayleigh’s analytical method for calculating the fundamental frequency of the tower gives accurate results. The method calculated the frequency for both rigid and flexible foundations within 5% of the frequency computed by the FEM. The method is thus an excellent method for determining the geometry of the tower in the preliminary design stage.

It became clear, while doing this project, that the formation of cracks in the concrete can have a significant effect on the behaviour of the tower, and even make it difficult to predict the dynamic behaviour. Post-tensioned tower construction is suitable for dynamically loaded structures and should eliminate the formation of cracks in the concrete if the post-tension force is sufficiently large. This has the benefit that it is possible to accurately compute the fundamental frequency of the tower. The durability of the structure will also be greatly increased. These advantages of post-tensioned concrete constructions may justify the cost increase of the method.

REFERENCES

A study on the design and material costs of tall wind turbine towers in South Africa

A C Way, G P A G van Zijl

The aim of this project was to study the structural design and material costing of various designs of tall wind turbine towers and the associated foundations in a South African context. Design guidelines are proposed for the design of tubular steel, concrete and concrete-steel hybrid towers and foundations for hub heights of 80, 100 and 120 m. The results indicate that concrete and hybrid towers become viable alternatives to the conventional steel towers at hub heights equal to and above 100 m.

Three heights – 80 m, 100 m and 120 m – of each type of tower (steel, concrete and hybrid) and their foundations were designed according to the relevant design standards. The designs were verified using the Abaqus CAE finite element software (SIMULIA 2010). The material costs of the designs were calculated for a South African environment, according to the increases in material cost with increasing hub height.

In this paper, the required foundation sizes for the concrete and hybrid towers were found to be smaller than for the steel towers. The material costs of the concrete and hybrid towers were shown to be lower than for the steel towers, especially at hub heights above 100 m. An increase in hub height caused an increase in energy generation of 3.52% and 6.28% for 80 m to 100 m, and for 80 m to 120 m hub heights, respectively. It is postulated that the concrete and hybrid towers become viable alternatives to the conventional steel towers at hub heights above 100 m.

INTRODUCTION

Background and literature review

The introduction of the Renewable Energy Independent Power Producer Procurement Programme (REIPPPP) in August 2011 has led to a fast-growing renewable energy industry in South Africa, particularly in the wind power sector. In three years, South Africa has procured more investment in independent power generation than had been achieved across the African continent for the past 20 years (Eberhard et al 2014).

The REIPPPP saw fast-tracked competition develop in the South African wind power sector. This resulted in a reduction of the price of wind energy across the three bidding rounds, as can be seen in Figure 1 (Eberhard et al 2014). The rapid decrease in the price of wind energy is a sign of a fast-maturing industry, although many criticise the sudden drop in prices (due to over-competitiveness in the industry), arguing that this has negatively affected the potential for increased local content and sustainability. The lowered price for wind energy has nevertheless made it one of the main renewable energy sources in South Africa.

The current trend in the global wind industry is to use taller wind turbine towers to access the stronger and less turbulent wind resources that occur at greater heights.

Figure 1 Reduction in the cost of wind energy over REIPPPP rounds

(IRENA 2012). In 2012, approximately 90% of installed wind turbine towers were of the tubular steel type due to their cost-effectiveness and ease of construction (World Steel Association 2012). For hub heights of up to 80 m, tubular steel towers have proven themselves to be the most cost-effective solution. As the tower hub heights increase to 100 and 120 m, however, steel towers start to lose their appeal (Harte & van Zijl 2007).

One of the main reasons for the increasing hub height trend stems from the current situation of wind power in Europe. Many of
the most favourable wind sites have already been exploited by currently operating wind farms, leaving only low to medium wind resource sites. In addition to this, recent advances in low to medium wind resource technology allow for the exploitation of sites that were previously considered to have unprofitable wind resources.

As the hub height increases, the towers need to be able to resist increased ultimate loads, bending moments and increased fatigue loads and moments. This is as a result of the greater wind loads acting on the tower, nacelle and blade assembly. Also to be taken into consideration is the stiffness requirement of the tower with regard to the interaction between the natural frequency of the tower and the rotational frequency of the turbine (Nicholson 2011). In order to satisfy these requirements, either the tower shell thickness needs to be increased, or the tower base diameter needs to be increased. The main problem with the steel tower occurs when tower base diameters are required to be larger than the allowable road transportation height limit of 4.5 m.

The development of turbines with nameplate capacity (i.e. maximum power generating capacity at optimal wind speed) in excess of 3 MW has also created the need for access to stronger wind resources. The added weight of these larger turbines and blades requires an increase in the structural strength of the towers. It is generally due to these reasons that different tower material and designs are currently being employed as an alternative to the conventional steel tower for taller hub heights.

**Tower types**
The three most common designs for tall wind turbine towers are:

- the conventional tubular steel tower
- the precast, post-tensioned, segmented concrete tower
- the concrete-steel hybrid tower.

**Tubular steel tower**
The conventional steel tower is manufactured in 20–30 m sections that taper in diameter and shell thickness from top to bottom. This is the most common type of tower in the world, and as such there are established manufacturers around the world who have optimised the tower for hub heights of up to 80 m. This tower type has the advantage of rapid construction, as there are only three or four sections that need to be lifted into place. As previously mentioned, the base diameter is limited due to transport constraints, which causes a notable increase in tower shell thickness with increasing hub height (Cotrell et al. 2014). There are currently no commercial scale steel towers that further split the sections into segments, due to the fatigue loads that would act on the connections between segments. In addition, even the smallest of manufacturing imperfections in connection components cause high stress concentrations that lead to fatigue failure.

**Precast concrete tower**
The precast concrete tower is generally manufactured off site in sections and further into segments, which alleviates the transportation problems associated with the large base-diameter sections prevalent in the taller steel towers. The segments are then transported to site where they are placed, grouted and post-tensioned. These towers have distinct advantages, other than the ease of transportation, in that the thicker concrete sections are stiffer than a typical steel tower section. This allows for reduced lateral deflections, longer fatigue life and higher tower natural frequencies. The disadvantages include the obvious lack of tensile strength of the concrete, creating the necessity for post-tensioning and the need for increased crane hire time.

**Concrete-steel hybrid tower**
The hybrid tower generally consists of a lower post-tensioned concrete section, ranging from 40–80 m, and an upper steel section (Nordex 2007). The lower sections can be cast in-situ, but are generally manufactured in segments like the precast concrete tower and transported to and assembled on site. This type of tower combines the benefits of both the steel and concrete towers, and only has the disadvantages of the concrete tower in the form of the post-tensioning requirement.

**AIMS AND METHODOLOGY**
This paper aims to:

a. Acquire and analyse South African wind data, ranging from 80–120 m above ground.

b. Study the design of wind turbine support structures and foundations for steel, post-tensioned concrete and concrete-steel hybrid type towers.

c. Determine whether an increase in tower height is viable for South Africa or not, by calculating the increase in the tower and foundation material costs as a function of tower height.

d. Develop guidelines for the South African wind industry with regard to the material costs and structural design of tall wind turbine towers.

Initially a literature study was conducted on the global and South African wind industries to gain a better understanding of the current status of, and trends in, the wind industry. An investigation into the wind resource and the available wind resource information in South Africa was performed to justify the use of taller wind turbine towers. A total of nine towers (concrete, steel and hybrid with heights of 80, 100 and 120 m) and their foundations were then designed according to the current international design methods.

The respective tower designs were then used to determine the material costs associated with increased hub height. Thereafter the increase in revenue generated as a result of increases in hub height was determined. The last two processes were carried out to develop an indication of whether increasing the hub height is a viable option in a South African context.

Note that the cost analysis in this paper is limited to the material cost. Clearly total cost should be considered when alternatives are
The wind resource analysis was based on data obtained from the Wind Atlas for South Africa (WASA 2014) project. This project aimed to set up a numerical wind atlas database for South Africa, a sample of which can be seen in Figure 2, which uses colour coding to differentiate between areas of high and low mean wind speeds.

The project erected ten wind masts around South Africa (Western, Northern and Eastern Cape) in order to verify the database. This data was made freely available to the public and currently has three full years’ worth of data (2011–2013). The masts have anemometers at altitudes of 10, 20, 40, 60 and 62 m in order to get an accurate representation of the wind profile at the given sites. The data was condensed and logarithmic extrapolation techniques were employed by the authors to extend the data to an altitude of 120 m. The data for eight of the ten masts (two contained large gaps in data) were used to calculate the increase in wind speed as a function of hub height, as can be seen in Figure 3. The average increases in mean wind speed values from 62 m hub height to 80, 100 and 120 m are 4.1%, 7.2% and 9.8% respectively.

**WIND RESOURCE ANALYSIS**

The wind resource analysis was based on data obtained from the Wind Atlas for South Africa (WASA 2014) project. This project aimed to set up a numerical wind atlas database for South Africa, a sample of which can be seen in Figure 2, which uses colour coding to differentiate between areas of high and low mean wind speeds.

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**DESIGN**

The design philosophy of the International Electrotechnical Commission IEC 6400-1:2005 Wind turbines—part 1: Design requirements (IEC 2005) was followed. Each of the nine tower-and-foundation combinations was subjected to the Extreme Wind Speed Model (EWM), as set out in IEC 6400-1:2005.

**Wind loads**

In the EWM, the wind turbine is subjected to an extreme three-second wind gust (one-in-fifty year return period) of 52.5 m/s at hub height, as for IEC class IIIA. This value was compared to the equivalent in SANS 10160-3:2009 (SANS 2009) and was found to be more conservative than the wind speeds in the presence of even the worst terrain category (Category A). In this circumstance, the wind turbine is in a non-operational, parked state, with the blades feathered out of the wind, assuming a yaw-misalignment of 15 degrees. The design wind speed, $V_{z,\text{ref}}(z)$, is distributed along the tower according to IEC 6400-1:2005, where $V_{\text{ref}}$ is the ten-minute mean wind speed and $z$ denotes height:

$$V_{z,\text{ref}}(z) = 1.4 \cdot V_{\text{ref}} \left( \frac{z}{z_{\text{hub}}} \right)^{0.11}$$

(1)

Note that in Equation 1, $V_{\text{ref}}$ is the wind speed at the hub height $z_{\text{hub}}$. The EWM is a case of an ultimate limit state (ULS), and the design reflects this state. Load factors as prescribed in IEC 6400-1:2005 were used, in conjunction with factors from EN 1997-1:2004 (Eurocode 7) (EN 1997 2004) that are not contained in IEC 6400-1:2005. The pressure distribution around the circumference of the tower is done in accordance with section 8.10 in SANS 10160-3:2009, with particular use of Figure 29. In addition to the wind loading on the tower, there are also wind loads that act on the blades and nacelle. The loads are summarised in Figure 4.

**Loads on foundations**

The loads from the tower transfer to the square foundation directly through an anchor cage. The loads that act on the foundation are thus the sum of the loads that act on the wind turbine. Two cases are assumed. The first is for the case of wind acting in the x-direction, as shown in Figure 4. The second case is for wind acting at 45 degrees to the x-direction, as seen in plan, which creates the necessity for the design of the foundations to consider these two different wind orientation cases.

---

**Figure 3** Increase in mean annual wind speed vs height (2011)

**Figure 4** Illustration of loads acting on wind turbine structure
An approximate equation for the calculation of a tower’s natural frequency, \( f_n \), is reproduced from Manwell et al (2010) in Equation 2, and is accurate to within 15%, but will be verified using the Abaqus FEM software. The natural frequency of the tower is affected by the tower material Young’s Modulus, \( E \), the second moment of inertia \( I \), the tower length, \( L \), and the mass of the rotor and tower.

\[
f_n = \frac{1}{2\pi N} \sqrt{\frac{3EI}{0.23 \cdot m_{\text{tower}} + m_{\text{rotor}}}} \]

(2)

For the turbine used in this study, a Vestas V112 3MW, the feasible allowable tower natural frequency zone, is shown as being between 0.245 Hz and 0.544 Hz in Figure 5.

**Foundation design**

The foundations were all designed to be square-shallow-gravity foundations. Water depths, varying from well below the foundation to ground level, were used in the design of the foundations. The soil conditions for this project were chosen to represent a typical wind turbine site along the coast of South Africa (\( c = 60 \text{ kN/m}^2, \phi = 30^\circ \), defined below).

The foundations were designed with particular attention to the resistance against overturn, resistance against sliding of the base, soil and foundation stiffness, tensile reinforcing and resistance against punching shear. The calculation of the soil’s bearing capacity, \( q_c \), is carried out according to Craig (2004), where \( I_i \) refers to inclination factors, \( s_i \) refers to foundation shape factors, \( N_i \) refers to bearing capacity factors and \( c, \gamma, D_e \) and \( b \) denote the soil cohesion intercept, the bulk density of the soil, the embedded depth of the foundation and the effective breadth of the foundation respectively:

\[
q_c = c_d N_c s_c \gamma + \gamma D_e N_q s_q + \frac{1}{2} \cdot \gamma b N_b s_b
\]

(3)

The sliding resistance of the foundation is not usually a governing factor, but should be checked nevertheless, according to DNV/Risø (2002), for drained and undrained soil conditions respectively, where \( \phi \) denotes the design soil angle of shear resistance, \( A_{\text{eff}} \) denotes the effective area of the foundation, \( V_d \) is the vertical design load and \( H_d \) the horizontal design sliding force.

\[
H_d < A_{\text{eff}} \cdot c_d + V_d \cdot \tan \phi
\]

(4)

\[
H_d < A_{\text{eff}} \cdot c_d
\]

(5)

One of the most important criteria of a foundation is to prevent overturn of the structure by an acceptable factor of safety. Put simply, the sum of the stabilising moments, \( \sum M_R \), must outweigh the sum of the overturning moments, \( \sum M_I \), by a factor, \( F_i \):

\[
F_i > \frac{\sum M_R}{\sum M_I}
\]

(6)

The stiffness of the foundation is incorporated into the FEM model using linear springs, calculated according to the equations from section 8.4 of DNV/Risø (2002) for vertical, horizontal, rocking and torsional stiffness. The formulae are based on work done by Gazetas (1983) and Elsabee (1973).

The flexural reinforcing in the foundation is present in both the top and bottom edges to resist the tensile stresses caused by the overturning moment from the wind loading, as shown in Figure 6. The design of the reinforced concrete foundation is performed according to EN 1992-1-1:2004 (EN 1992 2004).

The foundations were designed to negate the need for punching shear reinforcing. The requirements were checked using section
6.4.3 of EN 1992-1-1:2004. The shear force, $V_{ED}$, at the first control perimeter, $u_1$, with an effective depth, $d_{eff}$, must be less than the allowable shear stress for sections without shear reinforcing, $V_{Rd,c}$:

$$V_{ED} = \frac{\beta \cdot V_{ED,\text{reduced}}}{u_1} < V_{Rd,c}$$

$$= C_{Rd,c} \cdot k \left( 100 \cdot \rho_1 \cdot f_{ck} \right)^{0.5}$$

where:
- $\beta = 1 + 0.6 \pi \left( \frac{e}{D_{\text{tower}} + 4d_{\text{eff}}} \right)$
- $e$; $D_{\text{tower}}$ = eccentricity of applied load; tower diameter
- $V_{ED,\text{reduced}}$ = reduced vertical force acting on control perimeter
- $C_{Rd,c}$
- $\gamma_{mc}$ = partial material factor for concrete
- $k = 1 + \frac{200}{\rho_{I} \cdot d_{\text{eff}}}$
- $\rho_1$ = average reinforcing ratio
- $f_{ck}$ = characteristic cylinder strength of concrete

**Steel tower design**

The design of the steel tower is generally governed by fatigue (not covered here), buckling resistance or stiffness requirements (either to limit deflections or to satisfy natural frequency requirements). The buckling requirements, as laid out in DNV/Risø (2002), are satisfied through the inequality as shown in Equation 8. The buckling requirements consider the axial force, $N_{d}$, bending moment, $M_{d}$, tower shell thickness, $t$, tower radius, $R$, and the Euler elastic buckling load, $N_{El}$. A combination of the applied axial force and bending moment must be less than the critical compressive stress, $\sigma_{cr}$:

$$\frac{N_{d}}{2\pi R t} \leq \frac{N_{El}}{(N_d - N_{El})} \cdot \frac{M_{d}}{\pi R t} \leq \sigma_{cr}$$

(8)

The stiffness of the tower is acceptable when the natural frequency of the tower is within the acceptable limits, as shown in Figure 5. Wind turbine manufacturers sometimes limit the maximum lateral deflection of the tower, but such a limit is not included in this paper.

**Concrete tower design**

The design of the concrete tower is dominated by the tensile resistance (or the lack thereof) of the concrete. The tower therefore employs post-tensioning in order to limit the tensile stresses in the tower. The tower is designed so that the stresses that develop in the concrete sections are greater than the design mean tensile strength, $f_{\text{cd, cylinder}}$, of the 50 MPa concrete (taking tension as negative and compression as positive) as shown in Equation 9. The symbols $A$, $y$, $N_{ps}$, and $M_{ps}$ denote the cross-sectional area, distance to the extreme fibre of section, axial force and moment caused by the post-tensioning, respectively.

$$f_{\text{cd, cylinder}} < \frac{N_{ps}}{A} + \frac{M_{ps} \cdot y}{I} \leq \frac{M_{ps} \cdot y}{I}$$

(9)

The losses in pre-stressing force associated with post-tensioning (wobble friction, curvature friction, strand relaxation, elastic shortening, as well as anchoring) were considered using Equation 10, adapted from EN 1992-1-1:2004 in conjunction with the values shown in Table 1.

$$\delta_{\text{total}} = \left( e^{-\mu \cdot x \cdot a} + \delta_{\text{anchor}} + \delta_{\text{relax}} \right)$$

(10)

where:
- $\delta_{\text{total}}$ = fraction of pre-stressing force lost to friction effects
- $\mu$ = curvature friction coefficient
- $\mu$ = angle change along tendon length (rad)
- $K$ = wobble friction coefficient (rad/m)
- $x$ = tendon length (m)
- $\delta_{\text{anchor}}$ = fraction of prestressing force lost due to anchorage
- $\delta_{\text{relax}}$ = fraction of prestressing force lost to tendon relaxation effects

**Concrete-steel hybrid tower design**

As previously mentioned, the hybrid tower consists of a lower precast concrete section and a tubular steel tower upper section. The individual concrete and steel parts are designed with the same criteria as mentioned above for the steel and concrete sections respectively.

### Table 1 Pre-stressing losses information

<table>
<thead>
<tr>
<th>Tower</th>
<th>$\mu$</th>
<th>$\kappa$</th>
<th>$x$</th>
<th>$\alpha$</th>
<th>$\delta_{\text{anchor}}$</th>
<th>$\delta_{\text{relax}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete 80 m</td>
<td>0.1</td>
<td>0.0016</td>
<td>80</td>
<td>0.028</td>
<td>3</td>
<td>2.5</td>
</tr>
<tr>
<td>Hybrid 80 m</td>
<td>0.1</td>
<td>0.0016</td>
<td>40</td>
<td>0.036</td>
<td>3</td>
<td>2.5</td>
</tr>
<tr>
<td>Concrete 100 m</td>
<td>0.1</td>
<td>0.0016</td>
<td>100</td>
<td>0.023</td>
<td>3</td>
<td>2.5</td>
</tr>
<tr>
<td>Hybrid 100 m</td>
<td>0.1</td>
<td>0.0016</td>
<td>60</td>
<td>0.024</td>
<td>3</td>
<td>2.5</td>
</tr>
<tr>
<td>Concrete 120 m</td>
<td>0.1</td>
<td>0.0016</td>
<td>120</td>
<td>0.019</td>
<td>3</td>
<td>2.5</td>
</tr>
<tr>
<td>Hybrid 120 m</td>
<td>0.1</td>
<td>0.0016</td>
<td>80</td>
<td>0.018</td>
<td>3</td>
<td>2.5</td>
</tr>
</tbody>
</table>

### Table 2 Effect of mesh size on analysis accuracy

<table>
<thead>
<tr>
<th>Element size (mm)</th>
<th>Elements over shell thickness</th>
<th>Analysis time (s)</th>
<th>Natural frequency (Hz)</th>
<th>Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 000</td>
<td>1</td>
<td>2</td>
<td>0.88</td>
<td>4.97</td>
</tr>
<tr>
<td>1 000</td>
<td>1</td>
<td>2</td>
<td>0.91</td>
<td>1.73</td>
</tr>
<tr>
<td>600</td>
<td>1</td>
<td>3</td>
<td>0.918</td>
<td>0.86</td>
</tr>
<tr>
<td>500</td>
<td>1</td>
<td>3</td>
<td>0.92</td>
<td>0.65</td>
</tr>
<tr>
<td>250</td>
<td>1</td>
<td>13</td>
<td>0.924</td>
<td>0.22</td>
</tr>
<tr>
<td>125</td>
<td>2</td>
<td>117</td>
<td>0.925</td>
<td>0.11</td>
</tr>
<tr>
<td>80</td>
<td>4</td>
<td>20 935</td>
<td>0.926</td>
<td>0.00</td>
</tr>
</tbody>
</table>
when the stresses and strains across the shell thickness are not of importance.

The models consisted of 8-node linear hexagonal “brick” elements for the tower and non-inclined portions of the foundations, with 4-node linear tetrahedral elements being used for the inclined portion of the foundations.

Each of the nine tower and foundation combinations was subjected to three analyses. First, a modal frequency analysis was performed to accurately determine the natural frequency of the model and determine if it satisfied the natural frequency range requirements.

The second analysis involved the calculation of the buckling stress of each tower. In this analysis, the ultimate loads are applied to the structure and the analysis determines a factor relative to the buckling load to describe how safe or under-designed the model is. A factor of 1 indicates that the model is exactly on the brink of failure due to buckling. Values less than 1 indicate failure, and values over 1 indicate safety against buckling failure.

The final analysis was a two-step static load analysis with the ultimate limit state loads applied to the wind turbine structure. This was carried out with the intent of verifying the hand-calculated stresses and to check tower-top deflections. The second part of the analysis then used the deflections of the towers to determine the additional moments exerted on the tower due to the permanent loads acting at an eccentricity from the static centre of gravity (known as the $P-\Delta$ effect).

The loads for the static and buckling analyses included, as illustrated by Figure 7, are:

- Own weight of tower and turbine
- Torsional moment on the tower
- Tower top weight eccentricity
- Vertical post-tensioning loads on the tower, where applicable

Effect of underlying soil

The effect of the underlying soil is taken into account through the use of linear springs. The spring values are based on the previously-mentioned work by Gazetas (1983) and Elsabee (1973) found in DNV/Risø (2002). The springs were distributed around the underside of the foundation in the Abaqus model, simulating the vertical, horizontal, rocking and torsional stiffness of the underlying soil.

RESULTS

Tower and foundation dimensions

The final tower and foundation dimensions that satisfy the requirements as described above are shown in Tables 3 and 4.
Table 5 Results of FEM analyses

<table>
<thead>
<tr>
<th>Tower height</th>
<th>Tower type</th>
<th>Natural frequency (Hz)</th>
<th>Buckle value</th>
<th>Tower deflection (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>80</td>
<td>Steel</td>
<td>0.285</td>
<td>1.65</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td>Concrete</td>
<td>0.432</td>
<td>10.1</td>
<td>0.37</td>
</tr>
<tr>
<td></td>
<td>Hybrid</td>
<td>0.407</td>
<td>2.25</td>
<td>0.52</td>
</tr>
<tr>
<td>100</td>
<td>Steel</td>
<td>0.251</td>
<td>2.84</td>
<td>1.35</td>
</tr>
<tr>
<td></td>
<td>Concrete</td>
<td>0.333</td>
<td>4.65</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td>Hybrid</td>
<td>0.338</td>
<td>2.25</td>
<td>0.80</td>
</tr>
<tr>
<td>120</td>
<td>Steel</td>
<td>0.238</td>
<td>3.48</td>
<td>1.48</td>
</tr>
<tr>
<td></td>
<td>Concrete</td>
<td>0.261</td>
<td>2.38</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td>Hybrid</td>
<td>0.297</td>
<td>2.27</td>
<td>1.06</td>
</tr>
</tbody>
</table>

The asterisk in Table 3, under the 120 m steel tower, indicates that this tower does not satisfy the natural frequency stiffness requirements. A frequency separation of only 12% could be achieved between the 1P and tower natural frequency, despite keeping the tower diameter at 4.5 for the first 45 metres of the tower. As can be seen, the concrete sections are notably thicker than their steel counterparts. This is due to concrete’s relatively limited capacity for tension and compression strength, as well as a lower material stiffness in comparison to steel.

As can be seen in Table 4, the foundation volume is proportional to the tower height. An increase in tower height requires an increase in foundation concrete volume for one of the following reasons: to provide weight to counter the tower overturning moment, to provide extra foundation height negating the need for punching shear reinforcing, to increase the spread area of the foundation to prevent bearing capacity failure, or to provide more weight at the base in order to raise the natural frequency of the system.

An example of the shear force and bending moment diagram for the 80 m steel tower foundation is shown in Figure 8. The maxima and minima from the graphs will be even greater for IEC wind classes II and I.

The foundation designs were all governed by the case where the water table is at ground level, as a considerable buoyancy force acts upwards on the foundation. The steel and hybrid tower foundations were all governed by the need for foundation weight to counter the overturning moment from the wind loading. The 80 m concrete tower was governed by the same case, but as the weight increased, the 100 m tower foundation was governed by simultaneous overturn and bearing capacity failure.

As the weight increased even more, the 120 m concrete tower foundation design was governed by the bearing capacity of the soil. The fictional site had quite favourable soil conditions, and so the bearing capacity was not prevalent. In weaker soils (lower $c_d$ and $\phi$ values), bearing capacity is likely to be more prevalent as the governing foundation design parameter. Thus in the presence of weak soils, the steel and hybrid towers will be more appropriate, as they are lighter than the concrete towers.

FEM Results

Natural frequency analyses

A natural frequency analysis was carried out on all nine of the models, all of which, except the 120 m steel tower, satisfied the natural frequency requirements ($0.245 \, \text{Hz} < f_n < 0.544 \, \text{Hz}$). The hand calculations compared well with the FEM results, the greatest error in natural frequency hand calculation being 12.6%. The first four natural frequency modes of the 80 m concrete tower are shown in Figure 9. The first (lowest) natural frequency of each tower can be seen in Table 5. As can be seen in Table 5, the concrete and hybrid towers do not have problems associated with low natural frequencies (lack of tower stiffness). Conversely though, they can sometimes have natural frequencies that are too high (overly stiff tower), depending on the choice of turbine.

Buckling analyses

Similarly, a buckling analysis was carried out on all nine models, the results of which compare well with the buckling stress hand calculations and confirm that the buckling
capacity is not a governing factor in the design of the towers studied in this project. The results of the buckling analyses can be seen in Table 5. It should be noted that the hand calculations were acceptably conservative for all nine designs.

Tower deflection
The deflection of the towers at the ULS is also shown in Table 5. Deflections are not calculated at SLS, as wind speeds at SLS only reach 25 m/s before the turbine automatically shuts down and its blades are feathered out of the wind to reduce excess drag at higher ULS wind speeds (52.5 m/s). Furthermore, due to the nature of the structure, SLS deflections do not hinder the functionality of the turbine and are thus of little concern, whereas excessive ULS deflections are more likely to damage the turbine.

Similar studies prescribe a rule-of-thumb maximum tower top deflection of 1.25% of the tower height in order to protect the turbine against deflection-induced damage (Nicholson 2011). One can see that the deflections of the steel towers are considerably higher than that of the stiffer concrete and hybrid towers. Tower top deflections are sometimes limited by turbine suppliers and it is thus a possibility that the steel towers may have to be further stiffened in order to comply with this requirement, although all of them are lower than 1.25% of the tower heights.

Tower stresses
The tower stresses were obtained from the static analysis carried out on all the designs. As predicted, the hand calculations are conservative in comparison to the FEM results, as illustrated in the graphs in Figure 10. This figure illustrates the tension and compression side (windward and leeward side) of the 80 m concrete tower. Tensile stresses are shown as negative, and compression as positive values.

One can see that the hand-calculated tension stresses vary from the FEM values. This is likely due to the fact that the hand calculations assume that the tower is completely fixed at the base, whereas the FEM model considers the non-fixity of the base (in the form of the springs on the underside of the foundation, allowing small deflections due to the elasticity of the soil).

Most importantly, the extreme values (tension in concrete and compression in steel) in the FEM outputs are less critical than the hand calculation, which indicates safe design.

Pre-stressing
The loss in pre-stressing force for the concrete towers is considerably higher than that of the hybrid towers. This is due to the extra length of pre-stressing required in the concrete towers. Even though the concrete portion of the hybrid tower is shorter than the concrete tower, the hybrid tower requires more overall pre-stressing force than the concrete towers, as can be seen in Table 6. This is mainly due to the concrete sections in the hybrid towers being thinner than the concrete towers.

Material cost comparison
The costs used in the material cost comparison were obtained from South African manufacturers and suppliers, and are exclusive of labour, due to the material cost comparison nature of this project. The costs used are an average of the 2014 prices obtained from

---

### Table 6 Tower pre-stressing force requirements

<table>
<thead>
<tr>
<th>Height</th>
<th>Tower</th>
<th>Losses</th>
<th>Total PS force in tower (MN)</th>
<th>PS force for costing (MN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>80</td>
<td>Concrete</td>
<td>17.8%</td>
<td>23.9</td>
<td>33.4</td>
</tr>
<tr>
<td></td>
<td>Hybrid</td>
<td>12.0%</td>
<td>33.2</td>
<td>43.4</td>
</tr>
<tr>
<td>100</td>
<td>Concrete</td>
<td>20.5%</td>
<td>39.3</td>
<td>56.8</td>
</tr>
<tr>
<td></td>
<td>Hybrid</td>
<td>14.9%</td>
<td>49.5</td>
<td>66.8</td>
</tr>
<tr>
<td>120</td>
<td>Concrete</td>
<td>23.1%</td>
<td>60.3</td>
<td>90.2</td>
</tr>
<tr>
<td></td>
<td>Hybrid</td>
<td>17.7%</td>
<td>64.6</td>
<td>90.2</td>
</tr>
</tbody>
</table>

### Table 7 Component/material cost summary

<table>
<thead>
<tr>
<th>Material/Component</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foundation concrete</td>
<td>(R/m³)</td>
<td>1 400</td>
</tr>
<tr>
<td>Tower concrete</td>
<td>(R/m³)</td>
<td>2 007</td>
</tr>
<tr>
<td>Reinforcing steel</td>
<td>(R/ton)</td>
<td>15 251</td>
</tr>
<tr>
<td>Tower plate steel</td>
<td>(R/ton)</td>
<td>18 912</td>
</tr>
<tr>
<td>Pre-stressing: anchors and couplers</td>
<td>(R/tower)</td>
<td>33 440</td>
</tr>
<tr>
<td>Pre-stressing: tendons and support clips</td>
<td>(R/MN/m)</td>
<td>120</td>
</tr>
</tbody>
</table>

---

**Figure 10 Comparison of stresses from hand calculations and FEM results**

---
various suppliers/manufacturers in South Africa and can be seen in Table 7. The material cost of the tower and foundations are shown in Figure 11 and Table 8 and include the tower material used, the foundation concrete and reinforcing and the pre-stressing material costs.

Material costs are not the only costs associated with the production and erection of the tower and foundation. Other costs associated with transport, and labour and lifting costs, amongst others, also play a role in determining the cost of a finished product. As can be seen from Figure 11, the concrete and hybrid towers are less material-cost-intensive than the steel towers, particularly for the 100 and 120 m towers. The trend seems to show that the hybrid towers will become more cost effective than the concrete towers at hub heights greater than 110–115 m. The steel towers are shown to be disproportionately material-cost-intensive at hub heights greater than 100 m.

Increase in revenue generation
The data obtained from the WASA project for the Napier mast was used to calculate the increase in revenue generated as a function of increases in hub height. This specific site has a middle-of-the-range increase in wind speed as function of hub height, and serves as a good example of how increasing the hub height can be beneficial to both investor and power utility. In Table 9 the annual average wind speed for the three years of data (2011 to 2013) are given for the 80 m hub height, as well as the logarithmically extrapolated values for the 100 and 120 m hub heights. A single Vestas V112 3MW turbine was used in the calculation of the increase in energy generated, with the 80 m hub height as reference. For the total annual energy generated, the following expression is used:

\[ E_a = \eta_{\text{elec}}\eta_{\text{unavail}} \sum_{i=1}^{N} \frac{1}{6} P(v_i) \]  

(11)

where \( v_i \) is the wind velocity for the particular site obtained from WASA and extrapolated to the relevant hub height, \( P(v_i) \) is the power generated by the particular turbine at the particular velocity \( v_i \), \( \eta_{\text{elec}} \) is a coefficient correcting for losses in generation and feeding into the grid, and \( \eta_{\text{unavail}} \) is a coefficient for losses due to unavailability of the turbine. See Figure 12 for the particular power curve used in this paper. Note further that \( N \) is the total number of wind readings per year, i.e. six per hour for 24 hours and 365 days per year, and typical values of \( \eta_{\text{elec}} = 0.97 \) and \( \eta_{\text{unavail}} = 0.95 \) have been used here (Feng & Tavner 2010). Subsequently the yearly revenue is calculated by multiplying the annual energy \( E_a \) with the unit price R0.74/kWh (see Figure 1) of REIPPP Round 3. For an increase in hub height from 80 to 100 m and from 80 to 120 m, an increase in revenue of 3.52% and 6.28% was found. The results can be seen in Table 9. It should be noted that the 20 year average shown in Table 9 is an extrapolation of the three years of data that was analysed.

CONCLUSIONS
The designs of the wind turbine foundations were highly dependent on the choice of underlying soil parameters. Given the favourable soil conditions used in this project, the design of the foundation for the steel and hybrid towers was governed by the weight of the foundation needed to stabilise the structure.

### Table 8 Tower material use and cost

<table>
<thead>
<tr>
<th>Tower</th>
<th>Steel in tower (ton)</th>
<th>Concrete in tower (m³)</th>
<th>Pre-stressing force (MN)</th>
<th>Tower length (m)</th>
<th>Pre-stressing cost (R)</th>
<th>Tower cost (R)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>80</td>
<td>183.6</td>
<td>–</td>
<td>–</td>
<td></td>
<td>3 471 608</td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>330.6</td>
<td>–</td>
<td>–</td>
<td></td>
<td>6 252 465</td>
<td></td>
</tr>
<tr>
<td>120</td>
<td>685.7</td>
<td>–</td>
<td>–</td>
<td></td>
<td>12 968 527</td>
<td></td>
</tr>
<tr>
<td>Concrete</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>80</td>
<td>–</td>
<td>306</td>
<td>33.4</td>
<td>80</td>
<td>354 157</td>
<td>968 705</td>
</tr>
<tr>
<td>100</td>
<td>–</td>
<td>457</td>
<td>56.8</td>
<td>100</td>
<td>714 963</td>
<td>1 632 051</td>
</tr>
<tr>
<td>120</td>
<td>–</td>
<td>608</td>
<td>90.2</td>
<td>120</td>
<td>1 332 343</td>
<td>2 552 493</td>
</tr>
<tr>
<td>Hybrid</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>80</td>
<td>72.2</td>
<td>147</td>
<td>43.4</td>
<td>80</td>
<td>450 372</td>
<td>1 661 436</td>
</tr>
<tr>
<td>100</td>
<td>72.2</td>
<td>221</td>
<td>66.8</td>
<td>100</td>
<td>835 232</td>
<td>1 808 977</td>
</tr>
<tr>
<td>120</td>
<td>72.2</td>
<td>434</td>
<td>90.2</td>
<td>120</td>
<td>1 332 343</td>
<td>2 236 468</td>
</tr>
</tbody>
</table>
against overturning. This was for the case of the water table being at ground level. The foundation design of the concrete towers was governed by a combination of bearing capacity and foundation weight required to stabilise against overturning, also for the case of water table at ground-surface level.

The amount of reinforcing steel in the foundation was governed by the minimum reinforcing requirement in EN 1992-1-1, although this may not be the case when designing for IEC wind classes II or I. None of the foundations are subject to punching shear failure, even without punching shear reinforcement, although the concrete towers were just under the limit for requiring punching shear reinforcement. Foundations for concrete towers taller than 120 m will not be able to provide sufficient shear resistance without shear reinforcing, while the steel and hybrid tower foundations may still have adequate shear resistance.

The steel tower design was governed by the natural frequency stiffness requirements of the tower, primarily, to ensure that the natural frequency lies within the acceptable limits as determined by the choice of turbine, but also to limit deflections. The design of both the concrete and hybrid towers was dictated by the lack of tensile resistance of the concrete.

The tensile stresses in the sections (reduced by post-tensioning) thus governed the design of the concrete sections, although buckling may become an important design factor when designing towers taller than 120 m.

According to the results from this project, it can be seen that the material requirements associated with the foundation of concrete and hybrid wind turbine towers are lower than those of the steel towers for the given design assumptions. Consequently, and additionally, the material cost of the studied steel towers and foundations in a South African context are higher than their concrete and hybrid counterparts, particularly for hub heights in excess of 100 m.

The increased revenue, due to increases in hub height from 80 m to 100 m and 120 m for a Vestas V112-3.6 MW turbine, was shown to be in the vicinity of 3.52% and 6.28% respectively, with average capacity factor increases of the same magnitudes. It remains to be verified whether these additional revenues exceed the added total costs to realise the higher towers.

### Table 9 Revenue generation summary

<table>
<thead>
<tr>
<th>Year</th>
<th>Parameter</th>
<th>Unit</th>
<th>Year</th>
<th>Parameter</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>2011</td>
<td>Annual average wind speed (m/s)</td>
<td>8.71</td>
<td>2012</td>
<td>Annual average wind speed (m/s)</td>
<td>8.91</td>
</tr>
<tr>
<td></td>
<td>Wind speed increase (%)</td>
<td>0.00</td>
<td></td>
<td>Wind speed increase (%)</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>Energy generated (MWh)</td>
<td>13.136</td>
<td></td>
<td>Energy generated (MWh)</td>
<td>13.930</td>
</tr>
<tr>
<td></td>
<td>Revenue generated (Rm)</td>
<td>9.72</td>
<td></td>
<td>Revenue generated (Rm)</td>
<td>10.31</td>
</tr>
<tr>
<td>2012</td>
<td>Annual average wind speed (m/s)</td>
<td>8.53</td>
<td></td>
<td>Annual average wind speed (m/s)</td>
<td>9.20</td>
</tr>
<tr>
<td></td>
<td>Wind speed increase (%)</td>
<td>3.18</td>
<td></td>
<td>Wind speed increase (%)</td>
<td>3.21</td>
</tr>
<tr>
<td></td>
<td>Energy generated (MWh)</td>
<td>13.593</td>
<td></td>
<td>Energy generated (MWh)</td>
<td>14.422</td>
</tr>
<tr>
<td></td>
<td>Revenue generated (Rm)</td>
<td>10.10</td>
<td></td>
<td>Revenue generated (Rm)</td>
<td>10.67</td>
</tr>
<tr>
<td>2013</td>
<td>Averaged 20-year revenue (Rm)</td>
<td>205.62</td>
<td></td>
<td>Averaged 20-year revenue (Rm)</td>
<td>198.64</td>
</tr>
<tr>
<td></td>
<td>Revenue increase (%)</td>
<td>3.52</td>
<td></td>
<td>Revenue increase (%)</td>
<td>0.00</td>
</tr>
</tbody>
</table>

### REFERENCES


The chloride conductivity index (CCI) is a quality control parameter used to assess the resistance of concrete to chloride ingress. It is determined from the chloride conductivity (CC) test which has been in use in South Africa for close to two decades. During this time, it has undergone a number of changes to improve on its robustness, reproducibility, and repeatability. Plans are now under way to incorporate the test and other durability index tests (oxygen permeability and water sorptivity) into the SANS standards, and it is important that the end user is aware of the changes and why they were made. Specifically, aspects concerning a new design of the CC testing rig which is already in use, and a modified old design which is still being used in some laboratories, are highlighted, such as the significant but variable differences in CCI results that could occur between the old and new CC test rigs. Experimental investigations were carried out to establish the cause of these differences, which was identified as largely due to possible incomplete filling of the original device. Modifications were therefore made to the old CC test rig to ensure that both the new and the modified CC test rigs give similar results.

INTRODUCTION
The paradigm shift from prescriptive to performance-based durability design and specification of concrete structures has faced many challenges, not only in South Africa, but also around the world. These challenges include possible increased initial project costs, changing the mindset of practising concrete engineers and contractors, selection of suitable materials, quantification of severity of exposure environments, and development and standardisation of durability tests. Nevertheless, strides have been made to overcome these challenges, and in South Africa the combined efforts of researchers, engineers, contractors and cement manufacturing companies have seen the concrete industry adopt the durability index (DI) tests for concrete durability design and specification. The DI tests encompass chloride conductivity (CC), oxygen permeability (OP) and water sorptivity (WS) tests, and were developed as a result of collaborative research efforts at the Universities of Cape Town and the Witwatersrand, with support from the concrete industry. The DI tests have been in use in South Africa for close to two decades, and have been subjected to various tests to assess their robustness, reproducibility and repeatability, including international round-robin tests (Gouws et al 2001; Stanish et al 2006; Beushausen & Alexander 2008). The ultimate goal is to incorporate these tests into the South African National Standards (SANS), and plans are already under way to realise this. This paper focuses on the chloride conductivity test, which is used to assess the resistance of concrete to chloride penetration. Like any other test, the testing equipment, operator experience and materials used are just as important as the testing process itself. Common sources of error in this test include improper specimen conditioning (i.e. over-drying followed by vacuum-saturation with 5M NaCl solution), use of wrong concentration of the salt solution in the test, incorrect circuit connection (incorrect test set-up), and incomplete filling of the luggin capillaries and the anode and cathode compartments with the salt solution. The latter source of error (i.e. incomplete filling of the luggins and the compartments) mainly relate to the design of the CC test rig, although operator experience can also contribute to the errors.

In an attempt to limit the probability of occurrence of these errors, a number of changes have been made to the design of the CC test rig over the years, and it is important that the end user is made aware of these changes, i.e. why the changes were made, and how they affect the results, if at all. This is the purpose of this paper, but first a brief background of the CC test is given.

BRIEF BACKGROUND ON THE CHLORIDE CONDUCTIVITY TEST
The CC test, developed by Streicher (1997), is a rapid chloride conduction test used to assess the intrinsic potential of a given concrete (or mortar) to resist the ingress of chlorides by diffusion. However, in the test, chlorides penetrate the test specimen by migration (also
referred to as conduction, electro-diffusion or accelerated diffusion) due to a potential difference induced in the test set-up. The two transport mechanisms (diffusion and migration) can be related by the Nernst-Planck equation (Bockris et al. 1981; Andrade 1993):

\[ J = \left( \frac{D zF}{RT} \right) \frac{dU}{dx} \]  

(1)

where

- \( J \) = unidirectional flux of the ionic species (mol/cm²·s)
- \( D \) = diffusion coefficient of the ionic species (cm²/s)
- \( z \) = electrical charge of ionic species (ionic valence)
- \( F \) = Faraday’s constant (96500 C/mol)
- \( T \) = absolute temperature (K)
- \( U \) = potential difference (voltage) across the sample (V)
- \( x \) = a distance variable (cm)
- \( R \) = universal gas constant (8.314 J/mol·K)

A detailed test procedure is available in the Durability Index Testing Manual (DI Manual 2010). In summary, nominally 70 ± 2 mm diameter, 30 ± 2 mm thick concrete discs are prepared, typically from cored specimens. Four specimens (discs) are tested for each concrete. Prior to testing, the specimens are dried in an oven at 50°C for not less than seven days and not more than eight days. The specimens are then vacuum-saturated with 5M NaCl solution before being placed in a test rig with a cell filled with the same salt solution on either side. A 10 V potential difference is applied across the specimen, and the current through the specimen is measured in a very short time (typically 10 seconds). The chloride conductivity index (CCI) is then calculated using the formula (Streicher & Alexander 1995):

\[ \sigma = \frac{id}{VA} \]  

(2)

where

- \( \sigma \) = chloride conductivity index of the specimen (mS/cm)
- \( i \) = electric current through the specimen (mA)
- \( V \) = voltage difference across the specimen (V)
- \( d \) = average specimen thickness (cm)
- \( A \) = cross-sectional area of the specimen (cm²)

Chloride penetration resistance of a given concrete increases with decrease in \( \sigma \) and vice versa. Values of \( \sigma \) typically vary from < 0.5 mS/cm for dense chloride-resistant concretes to > 2.5 mS/cm for very penetrable concretes. The chloride conductivity is fundamentally related to steady state diffusivity (\( D_s \)) by the Nernst–Einstein equation, which relates the conductivity of a bulk material to its \( D_s \) as follows (Bockris et al. 1981; Garboczi & Bentz 1992):

\[ Q = \frac{D_s}{D_o} = \frac{\sigma}{\sigma_o} \]  

(3)

where

- \( Q \) = diffusivity ratio
- \( \sigma \) = conductivity of concrete (S/m)
- \( \sigma_o \) = conductivity of the pore solution (S/m)
- \( D_s \) = steady state diffusivity of chloride ions through concrete (m²/s)
- \( D_o \) = diffusivity of chloride ions in the equivalent pore solution (m²/s)

However, in reality non-steady-state conditions exist, and the chloride diffusion is represented by an apparent diffusion coefficient (\( D_a \)). This limits the application of the Nernst–Einstein equation. Therefore the CCI (\( \sigma \)) is empirically related to the (apparent) chloride diffusion coefficient (\( D_a \)) which is
used in service life design (Mackechnie & Alexander 1996). Practically the diffusion coefficient determined from \( \sigma \) is dependent on the binder type, age of concrete and marine exposure environment (Mackechnie 2001). This means that concretes made using different binder types can exhibit the same CCI, but different diffusion coefficients. This is mainly due to differences in chloride binding capacities of different binders (Arya et al 1990).

**THE CHLORIDE CONDUCTIVITY TEST SET-UP AND TESTING RIG**

The design of the CC test rig and set-up can contribute to systemic errors. The initial CC test set-up developed by Streicher (1997) is shown in Figure 1. Initially, 5M NaCl in saturated Ca(OH)\(_2\) solution was used. However, Streicher and Alexander (1995) showed that Ca(OH)\(_2\) has a negligible effect on the conductivity of the salt solution. Therefore, subsequent test set-ups used only 5M NaCl solution. Furthermore, the Cu/CuSO\(_4\) half cells were also omitted in subsequent test set-ups, as the voltage across the specimen could be accurately measured directly by replacing the porous plugs with stainless steel plugs (Figure 2). The modified CC test rig, shown in Figures 3 and 4, will be referred to as the “old CC test rig” in this paper.

The CC test rig shown in Figures 3 and 4 has been used virtually since the inception of this test. More recently (circa 2012), a “new CC test rig” was developed. A schematic representation of the new CC test rig is shown in Figures 5 and 6. The new CC test rig encompasses the following changes (with respect to the old CC test rig):

i. The carbon anode is replaced with a stainless steel anode, i.e. both the anode and cathode are now made using stainless steel – Figure 7(a) and (b).

ii. The fixed length luggin capillaries are replaced with telescopic spring-loaded ones – Figures 7 and 8.

iii. A hole has been drilled in both the anode and cathode compartments to facilitate easier filling with 5M NaCl solution – Figures 5 and 9.

These design changes were motivated for the following reasons:

i. Use of stainless steel as both the anode and cathode electrode – this was not only driven by practical and cost considerations, but also by similar test set-ups used in other standard tests in which stainless steel is used as both the anode and cathode material, e.g. the Standard test method for electrical induction of concrete’s ability to resist chloride ion penetration (ASTM 2012). Early proving trials showed that
Replacing the carbon anode with stainless steel had no effect on measurements.

ii. Replacement of fixed length luggin capillaries with telescopic spring-loaded ones – even though the 5M NaCl solution has very high conductivity (213.7 mS/cm (Streicher 1997)), the luggin capillaries are meant to facilitate accurate measurement of the voltage across the test specimen only. The tips of the luggin capillaries should therefore be as close to the specimen surface as possible. The telescopic luggin capillaries enable this to be achieved, especially when specimens of varying thickness are tested.

iii. A hole drilled in both the anode and cathode compartments – these ensure complete filling of the anode and cathode compartments with the 5M NaCl solution before testing.

In order to ensure that the robustness, accuracy and repeatability of the CC test were not compromised, it was necessary to assess the effect(s), if any, of these changes on the test values, which are current (i) through, and voltage (V) across, the specimen. It was required that the old (Figure 3) and the new (Figure 5) CC test rigs should give similar results if the same concrete specimen is tested at the same age in both rigs. An initial detailed assessment of the performance of the new CC test rig in comparison to the old one was conducted by Mukadam (2014) whose study showed that (Figure 10):

i. In general, for a given concrete (binder type and w/b ratio) the new CC test rig consistently gave higher CCI values than the old one. However, there was no consistency in the differences in the CCI values between the two CC test rigs. For a given binder type, concretes with higher w/b ratios resulted in higher differences in CCI values between the two rigs than those with lower w/b ratios.

ii. Even though there were differences in CCI values between the two rigs, the trends in CCI values for a given binder type and w/b ratio were consistent for the two test rigs, with CCI values increasing with increasing w/b ratio and vice versa, as expected. Mukadam (2014) suggested three possible causes of this difference, summarised as follows:

i. Difference in electrode material: the use of stainless steel for both the anode and cathode material in the new CC test rig as opposed to stainless steel cathode and inert carbon anode in the old one.

ii. Use of telescopic (new CC test rig) versus fixed length (old CC test rig) luggin capillaries.

iii. Presence of air bubbles in the old CC test rig which could potentially block the migration of chloride ions, causing a decrease in the current registered, and as a result a lower conductivity. However, it was not clear in his work whether the “air bubbles” were entrapped in the luggin capillaries, anode and cathode compartments, or both.

The findings by Mukadam (2014) were the genesis of an exploratory investigation to find out the cause(s) of the differences in CCI values between the old and new CC test rigs. However, after a critical review of the three possible causes, (i) and (ii) were found unlikely to be the source(s) of the difference in CCI values between the two test rigs due to the reasons given earlier. Focus was therefore placed on (iii) even though the reason given by Mukadam (i.e. “… air bubbles blocking the migration of chloride ions”) was not clear because it could potentially occur in either of the rigs, and would therefore be random in nature.

Further experimental tests were needed, but before these could be carried
out, a critical review of Mukadam’s raw data (Mukadam 2014) was necessary. The review showed that the difference in the CCI values between the old and new CC test rigs emanated only from the difference in the measured current through the test specimen; all the other parameters (i.e. specimen geometry and voltage across the specimen) remained unchanged. Mukadam’s results showed that a higher current through the test specimen was recorded in the new CC test rig than in the old one, even though the voltage across the specimen was, as required, kept constant at 10 V. The review of Mukadam’s results and a careful assessment of the old CC test rig led to the hypothesis that the difference in the CCI values obtained using results from the old and new CC test rigs was due to incomplete or variable filling of the anode and cathode compartments with 5M NaCl solution (especially the cathode cell compartment – see Figure 11(a)) during test assembly.

This is not the case in the new CC test rig where the anode and cathode compartments are filled through the holes in the compartment ends (see Figures 5 and 9). However, care should still be taken when filling the new CC test rig with the salt solution to avoid entrapping air bubbles in either the anode or cathode compartments (and even in the telescopic luggin capillaries).

**Cause and effect of incomplete filling of the anode and/or cathode compartments with 5M NaCl solution**

**Cause:** The design of the old CC test rig and the procedure used to assemble it during the test makes it susceptible to incomplete filling of the anode and/or cathode compartments (especially the cathode cell compartment – see Figure 11(a)) with the 5M NaCl solution. This is not the case in the new CC test rig where the anode and cathode compartments are filled through the holes in the compartment ends (see Figures 5 and 9). However, care should still be taken when filling the new CC test rig with the salt solution to avoid entrapping air bubbles in either the anode or cathode compartments (and even in the telescopic luggin capillaries).

**Effect:** Incomplete filling of the anode and cathode compartments in the old CC test rig leads to over-estimation of the cross-sectional area of the specimen when calculating the CCI index using Equation 2, which assumes that the whole surface area of the specimen’s cross-section is in contact with the salt solution during the test when it is carried out with the test rig assembly in a horizontal position. If the whole cross-section area of the specimen is not in contact with the salt solution during the test (Figure 11(b)), the current through the specimen is smaller than if it were fully in contact with the salt solution – this will be shown later in the results obtained from tests carried out. Theoretically, the conductivity of a given material will increase with increasing cross-sectional area (Urone et al 2013), i.e. conductivity is inversely proportional to cross-sectional area (in contact with the 5M NaCl solution). Over-estimation of the specimen cross-section leads to under-estimation of the CCI and explains the lower currents reported by Mukadam (2014) in the old CC test rig compared to the new one. A further critical analysis of Mukadam’s results (Mukadam 2014) showed that, in order to obtain similar CCI values in the old CC test rig as those in the new one, lower cross-sectional areas are required (see Figure 12). This confirms that the whole surface area of the specimen was not in contact with the salt solution during the test. However, determining the actual surface area of the specimen in contact with the salt solution during the test is not practically feasible or desirable.

![Figure 10](Image) A comparison of CCI values obtained using the old and new CC test rigs (Mukadam 2014)

![Figure 11](Image) (a) Schematic showing portion of anode cell in the old CC test rig that may not be filled with 5M NaCl solution during assembly; (b) Typical cross-section showing surface of specimen in contact with salt solution during testing in the old CC test rig


SERIES I EXPERIMENTS: ASCERTAINING MUKADAM’S (2014) RESULTS

In order to ascertain Mukadam’s (2014) results with respect to the trends in CCI values obtained using both the old and new CC test rigs, three concrete mixes were made using CEM I 42.5R and three w/b ratios (0.40, 0.50 and 0.65). The mix proportions are summarised in Table 1. Three 100 mm cubes were cast for each mix. After casting, the concrete cubes were covered with a plastic sheet in the laboratory for 24 hours after which they were demoulded and cured under water at 23 ± 2°C for 28 days. After curing for 28 days, two 100 mm concrete cubes for each mix were cored to make test specimens (70 ± 2 mm diameter × 30 ± 2 mm discs) for testing in the old and new CC test rigs.

The circuitry used for the CC tests using either the old or new CC test rig was slightly modified to enable the recording of the supply voltage ($V_s$) and current ($A_s$) – see Figures 13 and 14. In the standard CC test, only the luggin capillary voltage ($V_c$) and supply current ($A_s$) are measured – these are adequate for the calculation of the CCI using Equation (2). The additional parameters were measured because this was an exploratory experimental investigation. In summary, the following parameters were measured during the CC testing (see Figures 13 and 14):

i. Supply voltage ($V_s$), i.e. voltage supplied by the power supply

ii. Capillary voltage ($V_c$), i.e. voltage across the test specimen

iii. Current “to” the test specimen ($A_t$), i.e. from the power supply

iv. Current “from” the test specimen ($A_f$).

The above measurements were recorded simultaneously using an Agilent 34970A data logger (fitted with Agilent 34901A 20 channel multiplexer module) connected to a desktop computer. Theoretically, for the series circuits shown in Figures 13 and 14:

i. $V_s > V_c$

ii. $A_s = A_t$

The specimens were tested following the procedure outlined in the DI Manual (2010).

### Table 1 Summary of concrete mix proportions (kg/m$^3$) – Series I experiments

<table>
<thead>
<tr>
<th>w/b ratio</th>
<th>Mix 1</th>
<th>Mix 2</th>
<th>Mix 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.40</td>
<td>475</td>
<td>1 040</td>
<td>725</td>
</tr>
<tr>
<td>0.50</td>
<td>380</td>
<td>1 040</td>
<td>805</td>
</tr>
<tr>
<td>0.65</td>
<td>292</td>
<td>1 040</td>
<td>879</td>
</tr>
</tbody>
</table>

### Figures 12 Variation in cross-sectional areas required for $\text{CCI}_{\text{old cell}} = \text{CCI}_{\text{new cell}}$ (data from Mukadam 2014)
to the letter, except for the assembly procedure for the new CC test rig with respect to filling the luggin capillaries and anode and cathode compartments with the 5M NaCl solution. For a given concrete, the same set of four specimens for each concrete mix was tested in both the old and new CC test rigs. During the testing process using the two rigs, the following were noted:

i. It was easier and less time-consuming to assemble the new CC test rig than the old one. This is attributed to the improved ergonomic design and the colour-coding used in this rig (see Figure 6).

ii. Filling of the anode and cathode compartments with 5M NaCl solution took a slightly longer time in the new CC test rig than in the old one.

Series I results and discussion
The results obtained from the Series I experiments showed the following:

i. As expected, the average 28-day CCI values increased with increase in w/b ratio regardless of the CC test rigs used (old or new) – Figure 15. Increase in CCI depicts decreased resistance of a concrete to chloride penetration (Alexander et al 1999).

ii. The CCI values calculated using results from the new CC test rig were consistently higher than corresponding ones calculated using results from the old CC test rig – Figure 15.

iii. For a voltage of ~10 V across the specimen, a higher current through the specimen is measured in the new CC test rig than in the old one – Table 2.

iv. The results also show that, as expected, \( V_S > V_C \) and \( A_S = A_C \) in both the new and old CC test rigs – Table 2. Furthermore, the results show that a higher supply voltage \( (V_S) \) is needed in the old CC test rig than in the new one to ensure that the voltage across the test specimen \( (V_C) \) is ~10 V as required.

These results, except for (iv), corroborate those of Mukadam (2014); Mukadam (2014) only reported \( V_C \) and \( A_C \). The trend in (iv) further confirmed the hypothesis that the differences in CCI values between the old and new CC test rigs are due to incomplete filling of the anode and/or cathode cell compartments with the 5M NaCl solution in the old CC test rig. It is interesting to note from Figure 15 that the error bars from the old and new CC test rigs overlap completely, or largely, for the different w/b ratios. Thus, statistically the sets of results can be considered statistically similar for each w/b ratio, implying that in practice differences are relatively small. As stated earlier, the conductivity of a test specimen decreases with decrease in “wetted” cross-sectional areas. Therefore, for a series circuit like the one used in the CC test, a higher voltage thrust will be required to ensure that the voltage across the specimen in the old CC test rig, whose whole cross-section area is not in contact with the salt solution during testing (Figure 11), is ~10 V. The increased resistance of the test specimen, due to decreased cross-sectional area conducting the current, results in a lower current through the specimen in the old CC test rig than in the new one. This is consistent with Ohm’s law, i.e. \( V = IR \) where \( V \) is the voltage, \( I \) is the current through
the conductor and $R$ is the resistance (or its inverse, conductance) of the material (Abbott 1989). However, it was suspected that the difference in the supply voltages ($V_s$) between the old and new CC test rigs was, to some extent, also due to the difference in electrode materials used in both rigs – carbon anode and stainless steel cathode in the old CC test rig, and stainless steel anode and cathode in the new CC test rig. This was explored in the next series of tests presented later in this paper in which the influence of incomplete filling of the anode and cathode compartments was eliminated.

It was necessary to test experimentally the hypothesis that incomplete filling of the anode and/or cathode cell compartments with the 5M NaCl solution in the old CC test rig was the cause of the difference in CCI values between the old and new rigs. A second series of tests was therefore carried out. This is presented in the following section.

**SERIES II EXPERIMENTS: TESTING THE HYPOTHESIS OF INCOMPLETE FILLING OF THE ANODE AND CATHODE COMPARTMENTS WITH SALT SOLUTION IN THE OLD CC TEST RIG**

In order to test experimentally the hypothesis that the differences in CCI values between the old and new CC test rigs were due to incomplete filling of the anode and/or cathode cell compartments in the old CC test rig, two options were available:

i. develop a method to quantify the actual surface area of the test specimen in contact with the 5M NaCl salt solution during the test, or

ii. ensure that the whole cross-section of the test specimen is in contact with the 5M NaCl salt solution during the test.

After a careful assessment of the practical feasibility of each option, option (ii) was pursued. In order to ensure that the whole cross-section of the test specimen is in contact with the 5M NaCl salt solution during the test, it was necessary to make sure that the anode and cathode compartments were completely filled with the salt solution. This was achieved by modifying the old CC test rig by drilling a hole (~8.7 mm diameter) in both the anode and cathode compartments of the old CC test rig (Figure 16), similar to the holes in the new CC test rig (Figure 5 and 9). After filling the compartments with the salt solution, the holes were closed using PVC screw caps, as is the case in the new CC test rig.

In this series, chloride conductivity tests were then carried on in laboratories at

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### Table 2 Average measured voltages and currents in the new and old CC test rigs – Series I tests

<table>
<thead>
<tr>
<th>w/b ratio (CEM I 42.5R)</th>
<th>Average voltage (V)</th>
<th>Average current (mA)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$V_s$</td>
<td>$V_c$</td>
</tr>
<tr>
<td>0.40</td>
<td>Old*</td>
<td>New**</td>
</tr>
<tr>
<td></td>
<td>14.28</td>
<td>13.37</td>
</tr>
<tr>
<td>0.50</td>
<td>13.60</td>
<td>14.47</td>
</tr>
<tr>
<td>0.65</td>
<td>14.61</td>
<td>13.54</td>
</tr>
</tbody>
</table>

* Old CC test rig; ** New CC test rig
See Figures 13 and 14 for definition of notations $V_s$, $V_c$, $A_s$ and $A_c$.
the Universities of Cape Town (UCT) and the Witwatersrand (WITS) for comparison purposes. Different specimens were tested in both laboratories, but this did not affect the objective of the tests, i.e. to test the same specimens in both the new and modified old CC test rig. Each laboratory had its own set of CC test rigs. The following should also be noted about the specimens tested in this series of tests at either the UCT or WITS laboratory:

i. At the WITS laboratory, the same concrete specimens that were used in Series I tests were again used in the new CC test rig and the modified old one (with holes for filling the anode and cathode cell compartments with salt solution). After Series I tests, the specimens were left submerged in 5M NaCl solution for 113 days; the specimens were therefore tested at the age of 148 days after casting. However, testing age was not important, so long as the same specimens were tested in both the modified old and new CC test rigs at the same age.

ii. At the UCT laboratory, test specimens were prepared from 100 mm cubes of the same concrete mix of a recent study; three 100 mm cubes were cast using the concrete with a w/b ratio of 0.58 and CEM II 52.5N (B-L), with minimal quantities of ground granulated corex slag and limestone. The mix proportions are summarised in Table 3. After casting, the concrete cubes were covered with a plastic sheet in the laboratory for 24 hours, after which time they were demoulded and cured under water at 23 ± 2°C for 28 days. After curing for 28 days, two 100 mm concrete cubes were cored to make test specimens (70 ± 2 mm diameter × 30 ± 2 mm discs) for testing. Each specimen was tested first in the modified old CC test rig, and then immediately in the new CC test rig.

### Series II results and discussion

The results obtained from the Series II experiments from both UCT and WITS laboratories can be summarised as follows:

i. The average 148-day CCI values (WITS results) increased with increase in w/b ratio, regardless of the CC test rigs used (new and modified old) – Figure 17. This trend was expected, and corresponds to that obtained in Series I experiments. However, the CCI values obtained in Series II experiments are considerably lower than those of Series I experiments due to the difference in age (i.e. microstructure densification due to continued cement hydration and a degree of chloride binding) of the specimens at testing – 28 days and 148 days in Series I and II experiments respectively.

ii. The CCCI values calculated using results from either the new or modified old CC test rig are consistently statistically the same for the various concretes tested in the two laboratories – Figure 17.

iii. For a voltage of –10 V across the specimen, similar currents through the specimen are measured in both the new and modified old CC test rigs – Table 4. In the case of the WITS results where $A_c$ and $A_p$ were measured, even though not reported here, $A_c = A_p$ in both the new and modified old CC test rigs, as in the Series I experiments.

iv. The WITS results show that, as expected, $V_a > V_c$ in both the new and modified old CC test rigs – Table 4. The results also show that, similar to Series I experimental results, a higher supply voltage ($V_s$) is still needed in the modified old CC test rig than in the new one to ensure that the voltage across the test specimen ($V_3$) is ~10 V as required.

In Series I experiments, the trend in (iv) was attributed partly to incomplete filling of the anode and/or cathode cell compartments with the 5M NaCl solution in the old CC test rig, and partly due to the difference in electrode materials in the old (carbon anode and stainless steel cathode) and new (stainless steel anode and cathode) CC test rigs. However, in Series II experiments, the influence of the contribution of incomplete filling of the anode and/or cathode compartments was eliminated by completely filling these compartments through the holes drilled in them as shown in Figure 16. Therefore, in the WITS results, the higher $V_s$ values recorded in the modified old CC test rig compared to the new one can be reasonably assumed to be due to difference in electrode material used in the two rigs. Nevertheless, the supply voltage ($V_s$) does not affect the CC test results, because a series circuit is used in the test. Regardless of the $V_s$ value during the test, it is important to ensure that a ~10 V
potential difference \( V_c \) is recorded, as was done consistently and correctly in Series I and II experiments. The results of Series II experiments therefore confirm that the differences in CCI values, which were essentially due to a difference in the measured currents through the different test specimens (see Table 2), had been caused by incomplete filling of the anode and/or cathode compartments of the old CC test rig with 5M NaCl solution. In the Series II experiments where the old CC test rig was modified to ensure the anode and cathode compartments are completely filled with the salt solution, the measured currents through the different test specimens were sensibly the same, as were the CCI values.

CO NCLUDING REMARKS AND THE WAY FORWARD

Based on the results of the experiments presented in this paper and those of Mukadam (2014), and taking into consideration that the chloride conductivity test is in the process of being incorporated into SANS standards, it is prudent that the larger South African concrete construction industry be aware of the design changes and modifications made with respect to the test rig. These are summarised as follows:

1. The old CC test rig can continue to be used, but it should be modified by drilling a hole in both the anode and cathode compartments to facilitate complete filling of these chambers with the 5M NaCl solution. This will ensure that:
   i. consistent chloride conductivity results (especially the current through the specimen) are obtained, and
   ii. results obtained using the new and modified old CC test rigs are similar (or statistically the same).

Laboratories using the old CC test rig can contact the authors of this paper to facilitate its modification as mentioned above. Contact can also be made with Mr Eike von Guerard of Secundum Engineering Services cc in Cape Town (T: +27 21 551 1734, M: +27 84 250 4430, E: secundum@blitzweb.com).

2. A new CC test rig is now being manufactured for use in the CC test. The old CC test rig will no longer be manufactured. The new CC test rig has the following characteristics:
   i. Telescopic spring-loaded luggin capillaries
   ii. A hole each in both the anode and cathode compartments
   iii. Stainless steel anode and cathode
   iv. It is colour-coded and has improved ergonomic design to facilitate easy assembly during the test.

Both the new and modified old CC test rigs will give similar results, and either can be used for testing the penetration resistance of a concrete to chloride ingress. However, it is important to highlight the following aspects of the CC test that will continue to require the attention of the user to ensure that reliable results are consistently obtained:

1. The luggin capillaries should be completely filled with the 5M NaCl solution before the test is carried out. In the process of filling the luggin capillaries, care should be taken to avoid entrapping air bubbles.

2. The CC cell assembly (both the new and modified old test rigs) should be placed standing vertically on a level surface when being filled with the 5M NaCl solution through the holes in the anode and cathode compartments to ensure that it is fully filled with the salt solution.

3. The test voltage across the specimen is read from the voltmeter (correctly connected across the luggin capillaries); it is not the voltage indicated in the DC power supply.

4. Inter-laboratory tests: laboratories are encouraged to participate in round-robin tests from time to time to check the reproducibility and repeatability of the test results.

REFERENCES


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