Sewer network design layout optimisation using ant colony algorithms

Uncertainties in the South African wind load design formulation

Reliability assessment of the South African wind load design formulation

Factors influencing the quality of design documentation on South African civil engineering projects

A numerical investigation on hydro-mechanical behaviour of a high centreline tailings dam

The influence of health and safety practices on health and safety performance outcomes in small and medium enterprise projects in the South African construction industry
<table>
<thead>
<tr>
<th>Page</th>
<th>Title</th>
<th>Authors</th>
<th>DOI</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>Sewer network design layout optimisation using ant colony algorithms</td>
<td>N de Villiers, G C van Rooyen, M Middendorf</td>
<td>10.17159/2309-8775/2018/v60n3a1</td>
</tr>
<tr>
<td>41</td>
<td>Factors influencing the quality of design documentation on South African civil engineering projects</td>
<td>E Akampurira, A Windapo</td>
<td>10.17159/2309-8775/2018/v60n3a4</td>
</tr>
<tr>
<td>49</td>
<td>A numerical investigation on hydro-mechanical behaviour of a high centreline tailings dam</td>
<td>M Naeini, A Akhtarpour</td>
<td>10.17159/2309-8775/2018/v60n3a5</td>
</tr>
<tr>
<td>61</td>
<td>The influence of health and safety practices on health and safety performance outcomes in small and medium enterprise projects in the South African construction industry</td>
<td>J N Agumba, T C Haupt</td>
<td>10.17159/2309-8775/2018/v60n3a6</td>
</tr>
</tbody>
</table>
Sewer network design layout optimisation using ant colony algorithms

N de Villiers, G C van Rooyen, M Middendorf

The optimal design of sewer networks typically comprises two sub-problems. The first is to determine an optimal layout of the network elements, and the second to optimally design the network components. In this article the focus is on the optimisation of gravity sewer network layouts, which requires simultaneous optimisation of hydraulic design. The layout is optimised using ant colony algorithms with four proposed node and edge-based selection strategies, while a heuristic optimisation algorithm is used for the hydraulic optimisation. The resulting simultaneous optimisation algorithm is shown to perform very well. The selection strategies are shown to be effective, but no clear best strategy is identified, as the performance of the layout algorithms is shown to depend heavily on characteristics of the network under consideration.

However, some strategies are shown to perform inconsistently and worse than others on average.

INTRODUCTION

The optimisation of sewer networks typically consists of two sub-problems. The first is to determine the layout of network elements and the second to determine all hydraulic parameters — such as diameters, slopes, etc. — of the network components. The two sub-problems of the optimisation are strongly linked — for each layout a unique set of hydraulic parameters exists. Consequently, if an optimal design is to be found, the sub-problems have to be solved simultaneously. However, this does not mean that a single selection strategy or algorithm has to be used for both sub-problems. Rather, the fitness of a solution should be determined based on both layout and component sizes simultaneously. Due to the complexity of such algorithms, most research has been done on one of the sub-problems, while the other remains static, usually the layout (Lejano 2006).

There are three approaches to simultaneous layout and element sizing optimisation:

1. Complete enumeration

In this approach all feasible layouts are generated and the hydraulic design of each is completed individually (Diogo et al 2000; Diogo & Graveto 2006). While this approach is very useful for finding the best layout, its application is only practical for small-scale problems.

2. Separated design

This approach separates the two design problems, either through manual layout design or by using individual objective functions for each sub-problem. Once the optimal layout is found, the optimal hydraulic parameters for this layout is determined by a separate algorithm (Pan & Kao 2009; Haghhighi 2013). While this approach is useful for large problems, it is difficult to determine true optima (Haghhighi & Bakhshipour 2015).

3. Simultaneous design

The layout and element size problems are optimised simultaneously (Li & Matthew 1990; Moeini & Afshar 2012; Haghhighi & Bakhshipour 2015). While this approach is the most promising in finding true global optima for large solutions its implementation is the most complex and requires complex formulation and specific design algorithms (Haghhighi & Bakhshipour 2015). The fitness of a solution is calculated taking both layout and hydraulic parameters into account simultaneously.

In this work the third approach (simultaneous design) is used to develop a hybrid Ant Colony Optimisation (ACO) algorithm by which the layout and all hydraulic parameters are simultaneously optimised. For each individual layout created by the ACO algorithms useful for finding the best layout, its application is only practical for small-scale problems.
algorithm the set of hydraulic parameters is optimised. The two sub-problems of sewer network optimisation are very different mathematically. The hydraulic design problem is a non-linear discrete programming problem, while the layout problem is a variant of the degree-constrained Minimum Spanning Tree (d-MST) problem in graph theory. In this article development of the layout optimisation algorithm and combining of the algorithm with the heuristic hydraulic optimisation procedure proposed by De Villiers et al. (2017) to create a hybrid algorithm capable of solving both sub-problems simultaneously, are addressed.

**MODELLING SEWER NETWORKS USING GRAPH THEORY**

In this investigation graph theory is used to model the layout of sewer networks. Layout design of sewer networks is again a two-part problem. The first is to determine the spatial location of manholes and pipes. The second is to determine the direction of flow of each pipe. A full network layout is only found once both parts of the problem have been solved.

Spatial design of the network

Referring to Figure 1, the nodes, representing manholes labelled 1, 2, 3 and 4 respectively, are connected by edges, representing pipes labelled 1, 2, 3, 4 and 5 respectively.

In this example the spatial positions of the manholes and pipes have already been determined, i.e. the first part of the design is complete. However, it may have been equally feasible to place the manholes at different locations and to connect them differently, for example by placing pipe 3 between manholes 1 and 4 rather than 2 and 3. This part of the design, deciding on the spatial location of manholes and how to connect them with pipes, is most often governed by existing or planned infrastructure, such as roads or buildings, and topographical considerations, such as hills or steep inclines. In this paper it is assumed that the positioning of manholes and pipes is completed prior to the optimisation process aimed at minimising the installation cost of the sewer network. The positioning of manholes and pipes is referred to as the base layout, or base graph of the layout. All pipes and manholes included in the base graph must be present in the final solution. The base layout is modelled mathematically as an undirected graph where the vertices represent manholes and the edges pipes:

\[ G_{\text{base}} = (V, E) \]

Where:

- \( G \) = the base graph
- \( V \) = the vertex set, whose elements are the vertices of \( G_{\text{base}} \) which represent manholes of the sewer network
- \( E \) = the edge set, whose elements are the edges of \( G_{\text{base}} \) which represent pipes of the sewer network.

**Directional design of the network**

The second part of the layout design is to determine the direction of flow for each pipe. This part of the layout design is deceptively complex, and the number of possible permutations grows exponentially as the number of vertices and edges in the base graph increases. This part of the layout design is the concern of the optimisation procedures which are discussed in detail in the section below titled “Objective function” on page 9. A brief overview of the required decisions to complete the directional design is given here. Figure 2 shows two directional graphs (directions are indicated by arrows).

Figure 2 shows two feasible final layouts of a possible \( 5^2 = 25 \) of the base layout shown in Figure 1. The choice of flow directions can heavily influence the final capital investment cost of the completed sewer network, especially when adverse topographical conditions are present. If, for example, manhole 2 has a much lower elevation than manhole 3, then using sound engineering intuition we can readily observe that the design of Figure 2(a) requires less excavation than that of Figure 2(b). This reduction in required excavation can be expected to lead to a reduction in capital expenditure. However, the problem becomes increasingly difficult as the size of the base graph increases, since the change in flow direction of a single pipe may have significant effects on the cumulative downstream flow rates within pipes, and therefore their required diameters and slopes.

Notice that in both designs in Figure 2, cycles are present in the final layout designs. For Figure 2(a) the cycle 2-4-3 exists and for Figure 2(b) the cycle 2-3-4 exists. In this investigation only gravity sewer networks are considered, with no special structures present, such as divergence structures, pumping stations or rising mains. This implies that all manholes may only have a single outgoing pipe, often referred to as the single out-degree constraint. These assumptions drastically simplify the hydraulic analysis of the network.

Moeini and Afshar (2012) propose disconnecting pipes from their upstream manholes and creating what they term adjacency nodes, which are created artificially at the same location as the existing upstream manhole. The practical implication of this is that the pipe has no upstream
inflow from the manhole, and cycles are removed from the network. Referring to Figure 3, the networks shown are similar to those in Figure 2. In this case, however, some pipes have been disconnected from their upstream manholes and adjacency nodes created, indicated by perpendicular lines on the upstream end of the pipe.

**Constraints on the optimisation problem**

De Villiers et al (2017) provide an overview of sewer network design, and all constraints and equations described there are applicable to this paper. When constructing a network layout, however, the single out-degree constraint, described above, has to be considered. This avoids cycles and diversion structures, and the resulting layout is a simple branched gravity sewer network with no special structures.

To ensure that any proposed layout adheres to these simplifications, only branched network layouts are selected from the power set of the base layout, which contains all possible looped and branched layouts. Mathematically the restrictions for a branched layout in a network with $M$ manholes are (Moeini & Afshar 2012):

$$X_{ji} + X_{ij} = 1 \quad \forall j, l \in \{1, ..., M\}$$

$$\sum_{l=0}^{M} X_{ji} = 1 \quad \forall j \in \{1, ..., M\}$$  \hspace{1cm} (1)

Where:

$$X_{ji} = \begin{cases} 1 & \text{if and edge with flow from } j \text{ to } l \text{ exists} \\ 0 & \text{otherwise} \end{cases}$$

This constraint is augmented with the continuity requirement at each node $j$:

$$\sum_{l=0}^{M} X_{ji} Q_{lj} - \sum_{l=0}^{M} X_{ij} Q_{ij} = 0$$

$$\forall j, l \in \{1, ..., M\} \land j \neq s$$  \hspace{1cm} (2)

Where:

$$Q_{ij} = \text{flow rate in pipe } i \text{ between nodes } l \text{ and } j \text{ with either node as source or target}$$

$s = \text{the outlet node}$

The network is defined with a single outlet in this paper. The continuity equation is not enforced at the outlet, since only the inflow is modelled for the outlet node. Note that this restriction does not affect the generality of the proposed method, since the same method may be applied for multiple outlets simultaneously.

**ANT COLONY OPTIMISATION**

ACO algorithms have been successfully applied to various constrained optimisation problems, and achieve state-of-the-art results for several important problem classes, such as the Quadratic Assignment Problem (QAP) and scheduling and routing (Dorigo & Stützle 2004) to name a few. The precursor algorithm to all ACO algorithms, Ant System (AS), was inspired by observing the pheromone-based trail-laying-trail-following of real ants (Dorigo et al 1996). Though modern ACO algorithms have come a long way from the initial AS model, the analogy of a colony of foraging ants is still useful in understanding the behaviour of the algorithm.

In ACO a number of individual ants each generates solutions independently and in parallel, over many iterations. The ants make decisions using a so-called ‘pheromone-value’, which models the fitness of an eligible decision at a decision point. The best solution in an iteration is used for trail-laying, i.e. the pheromone value is increased along the best trail, while some pheromone on all other trails is evaporated. Through this process of pheromone deposition and evaporation the search near good solutions is intensified over time, while initially maintaining diversity within the search space. The steps of a general ACO algorithm are now described (Dorigo et al 1996):

1. Select a suitable size for the set of ants $|A|^k$ for each generation $k$ and set initial pheromone values on all available selections to some suitable, but equal, value. Set generation count $k = 0$.

2. Starting from either a predetermined or randomly selected point, construct a solution for each individual ant, $a \in |A|^k$ of the current generation, using the standard transition rule to make a decision:

$$p_{ij}^k = \frac{[\tau_{ij}^k]^{\alpha} [\eta_{ij}]^{\beta}}{\sum_l ([\tau_{ij}^k]^{\alpha} [\eta_{ij}]^{\beta})}$$  \hspace{1cm} (3)

Where:

$k = \text{the generation number}$

$p_{ij}^k = \text{probability of decision } j \text{ at decision point } i$, hereafter “decision $ij$” in generation $k$

$\tau_{ij}^k = \text{pheromone value of decision } ij \text{ in generation } k$

$\eta_{ij} = \text{heuristic influence value at decision } ij$

$\alpha = \text{relative pheromone influence factor}$

$\beta = \text{relative heuristic influence factor}$

3. Using a problem-specific objective function, determine the fitness $f(a)$ of each ant’s solution, $a \in |A|^k$.

4. Acquire the generation-best solution $f(\text{best})^k$. Compare acquired generation best solution to current global best solution $f(\text{best})^{\text{global}}$, replacing the global best if the generation best solution is better.

5. Perform pheromone evaporation on all paths and increase the pheromone along the path selected by the generation-best solution, using the following update rule:

$$\tau_{ij}^{k+1} = \tau_{ij}^k (1 - \rho) + \Delta \tau_{ij}$$  \hspace{1cm} (4)

with $\rho$ the evaporation rate and $\Delta \tau_{ij}$ the pheromone increase of the generation-best solution, defined as:

$$\Delta \tau_{ij} = \begin{cases} \frac{C}{f(\text{best})^k} & \text{if decision } ij \text{ was made} \\ 0 & \text{otherwise} \end{cases}$$  \hspace{1cm} (5)
6. Check convergence of the algorithm. Usually convergence is accepted if a minimum number of generations have been completed, and for a number of generations thereafter the global-best solution has not improved. Alternatively, if all individuals of a generation produce the same solution, the algorithm has converged. If the algorithm has converged, accept current global-best solution as final solution, otherwise return to step 2 and repeat the process.

Many modifications have been proposed in the literature to improve the behaviour of the ACO algorithm. The modifications used in this implementation and their effects are now described.

- **Changing the evaporation rate.** The evaporation rate $\rho$ determines the convergence speed of the algorithm. In general, when a large search space is to be investigated a low value of $\rho$ is beneficial, since the algorithm will be allowed more time to explore the different regions of the search space before focusing on a small region (Merkle et al. 2002). Merkle et al. (2002) found that, when the maximum number of iterations is restricted, a higher value of $\rho$ usually performs better. Therefore it is proposed by Merkle et al. (2002) that two different values of $\rho$ be used during the run of an ACO algorithm. Initially, a low $\rho$ is used which remains constant for the majority of the generations. For the last generations of the algorithm a high $\rho$ value is used to perform a final intensive search near the best solution that has been found.

- **Modified elitist strategy.** Using an elitist strategy is a common modification to ACO algorithms. This entails using a pheromone update from both the generation-best and current global-best solution at the end of each generation. The pheromone update rule is modified to reflect this:

$$
\tau_{ij}^{k+1} = \tau_{ij}^{k}(1 - \rho) + \Delta \tau_{ij} + \Delta \tau_{ij}^{global}
$$

(6)

The elitist strategy has the advantage that the search is intensified around the current global-best solution. However, if the global-best solution remains unchanged for many generations it has a great influence on the pheromone values which may, during long runs, cause the algorithm to converge prematurely to the current global-best solution (Merkle et al. 2002). This is especially true if the current global-best solution is a single good solution, with no other good solution in the neighbourhood.

It is therefore proposed (Merkle et al. 2002) to set a maximum number of generations, $g_{max}$, during which an elitist solution is allowed to remain unchanged. When the elitist solution has exceeded its maximum number of generations it is replaced by the current generation's best solution, even if this solution is worse than the current global-best. The replacement is only in terms of pheromone updates; the solution is, however, retained as the current best solution of the optimisation. When an elitist solution has good solutions in its neighbourhood it is likely the ants will discover it within reasonable time. Otherwise it does not matter that the elitist solution has been discarded, as no improved solutions are in its neighbourhood.

### LAYOUT OPTIMISATION

Layout optimisation of a sewer network is one part of the two-part network optimisation problem, in which the flow direction of pipes has to be determined for a given base layout. This part of the problem has been studied less than the hydraulic optimisation problem. However, some researchers have proposed algorithms for the simultaneous solution of both sub-problems. Walters (1985) used Dynamic Programming (DP) for simultaneous layout and size optimisation, and his method could be used to drain a set of sources with fixed positions. Li and Matthew (1990) used Discrete Differential Dynamic Programming (DDDP), which utilised an iterative procedure to generate the layout, and then to size the sewers and pumps while keeping the layout fixed. DDDP has some significant drawbacks – it restricts the search space and reduces the probability of locating the global optimum. The DDDP stages must be manually divided for each individual problem and this reduces its practicability. Pan and Kao (2009) used a Genetic Algorithm (GA) combined with Quadratic Programming (QP). In their approach a majority of the constraints were formulated in QP, while other parameters, such as layout and pipe diameters, were determined by the GA. Moeini and Afshar (2012) proposed an Ant Colony Optimisation (ACO) algorithm combined with a Tree Growing (TG) algorithm which performs both the layout construction and selects diameters simultaneously. In their approach it is assumed that all pipe flow rates are at maximum relative flow depth, allowing for the calculation of pipe slopes. Haghighi and Bakshishpour (2015) combined previous works, namely the loop-by-loop cutting algorithm (Haghighi 2013) and an Adaptive Genetic Algorithm (Haghighi & Bakshishpour 2012), with a Tabu Search (TS) algorithm to create an effective hybrid algorithm for simultaneous layout and element size optimisation.

Despite the suitability of ACO algorithms to the layout optimisation problem of sewer networks they have seen limited use. The main reason for this is the two-part nature of the problem. ACO algorithms require a significant number of function evaluations, for each of which both layout and hydraulic optimisation have to be performed simultaneously if the best results are to be obtained. Because of this the algorithm can be extremely computationally expensive. To overcome this problem Moeini and Afshar (2012) in their ACO-TG algorithm use the ants to simultaneously select both layout and diameter. This strategy, of using a single algorithm for both layout and diameter selections, is also employed by Pan and Kao (2009) in their GA-QP algorithm. The major disadvantage of this is the potential for fitness warping. If a very good layout is produced early on in the iterations, it is very likely that a poor set of diameters will be selected with it, and consequently the fitness of the entire network is compromised and the algorithm is unable to identify that a good layout has been found. In order to overcome fitness warping a separate optimisation algorithm is employed for each sub-problem in this paper. This approach, also used by Haghighi and Bakshishpour (2015) in their hybrid Tabu-Search algorithm, can be extremely computationally expensive. Haghighi and Bakshishpour (2015) overcome the computational restrictions of this approach by using an efficient layout-generating algorithm combined with a relatively efficient meta-heuristic for element size optimisation. In this work, the computationally expensive ACO algorithm, combined with a TG algorithm as proposed...
by Moeini and Afshar (2012), is used to determine layouts and then combined with the computationally efficient heuristic optimisation algorithm developed by De Villiers et al (2017).

In this implementation the networks are restricted to gravity sewer networks. Additionally no cycles nor diversion structures are allowed within the network. This is achieved by restricting the out-degree, the number of outlet pipes, of each node to one. This assumption creates a variant of the d-MST problem, not simply because only the out-degree is constrained, but also because no clear definition of a minimum exists. The lack of a minimum definition means that traditional graph theory algorithms for minimum spanning tree construction, such as Prim’s or Kruskal’s algorithms, cannot be used without significant modifications. Bui and Zrncic (2006) showed that ACO algorithms perform well for the solution of d-MST problems. These applications offer valuable insights which may assist in understanding the nature of optimal layouts of sewer networks. Bau et al (2008) compared Prim’s algorithm, which uses node-based selection, and Kruskal’s algorithm, which uses edge-based selection, and found Kruskal’s algorithm to be superior. Both node-based and edge-based layout creation strategies are developed and compared:

1. Edge-based selection which directly queries the base graph to construct a spanning tree, similar to Moeini and Afshar (2012) – henceforth referred to as the “direct-edge” strategy.

2. Node-based selection which directly queries the base graph to construct a spanning tree – henceforth referred to as the “direct-node” strategy.

3. Constructing a spanning tree using a permutation of unique edge identities – henceforth referred to as the “permutation-edge” strategy.

4. Constructing a spanning tree using a permutation of unique node identities – henceforth referred to as the “permutation-node” strategy.

For all the selection strategies above the hydraulic optimisation is performed by the Heuristic Optimisation Algorithm developed by De Villiers et al (2017). The hydraulic optimisation algorithm is deployed for each individual layout created by the layout creation algorithm to determine the optimal set of hydraulic parameters. Once the layout creation and hydraulic design are complete the fitness of the solution may be calculated using Equation 7.

Figure 1 shows a small example network’s base layout, which will be used to describe the spanning tree construction strategies. The base layout, alternatively referred to as the base graph, of the network shows the position of all the manholes and all the pipes that are required in the network. Additionally, the elevations and design inflow rates at each manhole are known. The layout optimisation algorithm does not move the pipe around, but is rather used to determine the flow direction of pipes. In this example node 1 is the outlet node.

Figure 4 shows all the possible paths of nodes and edges for the example network in Figure 1. The nodes are shown in circles, while the edges which would result in the addition of the next node are shown in square brackets.

Figure 4 is not determined by using any of the selection strategies. This simply shows all the possible paths to follow to construct a spanning tree of this network. The selection strategies are employed to determine which decisions are eligible and decide how to present the eligible decisions at each point to the layout creation algorithm. For example, at the start the layout creation algorithm could be presented with either the eligible edges 1, 2, or the eligible nodes 2, 3. If edge 1, or node 2, was selected, then the eligible set of edges and nodes at the following iteration are 2, 3, 4 and 3, 4 respectively.

In an ACO that uses a single pheromone matrix, the ants can only make one choice based on the pheromone. If another choice has to be made, some mechanism, usually a heuristic, is required to resolve it. For the sewer network layouts, a useful parameter proposed my Moeini and Afshar (2012) is the hop-rank. This parameter ranks nodes based on the number of preceding nodes in its branch. Referring to Figure 1, if a spanning tree consisting of edges 1, 3 and 5 is assumed, the hop-ranks of nodes 1 through 4 are respectively 0, 1, 2 and 3. The hop-rank parameter can be used to favour selections which do not increase the length of already long branches. If required, the hop-rank parameter can be used to make a heuristic selection. In all cases no heuristic influence value $\alpha_{ij}$ is used, since that would render direct comparison of the effectiveness of the strategies impossible.
**Direct-edge layout creation**

This strategy mimics the edge selection behaviour of the algorithm proposed by Moeini and Afshar (2012). The tree-growing algorithm compiles a set of eligible edges, always starting from the static sink node. An edge is considered eligible for selection if only one of its vertices is already included in the growing spanning tree. The edge selection procedure is shown below.

1. **Define:**
   - $T$ = the spanning tree
   - $T_N$ = the set of nodes in the spanning tree
   - $E$ = the set of currently eligible edges
   - $n_i$ = target node of the new edge
   - $n_s$ = source node of the new edge

2. **Initiate $T$ and $T_N$.** Insert the sink node into $T_N$.

3. **Compile $E$, the set of all eligible edges; an edge is considered eligible if $T_N$ contains one of its nodes.**

4. **Select the next edge $e$ of $T$ from $E$ using the transition rule described in the section titled “Ant Colony Optimisation” on page 4.**

5. **Identify $n_i$ and $n_s$ of $e$. Select $n_i$, the target node, as the node already contained in $T_N$ and the other as $n_s$.**

6. **Add $e$ to $T$, add $n_s$ to $T_N$.**

7. **If $T_N$ contains all nodes, stop. Or else return to 3.**

Referring to Figure 4, if in the first iteration edge 1 was selected and in the second edge 2, then the set of eligible edges for iteration three would be $E = \{4; 5\}$. Edge 3 is not eligible since both its nodes are elements of $T_N$ and it would introduce a cycle into the network. The way in which edge 3 will be added to the network is described further down in the section titled “Layout completion” on page 8. Determining the source (upstream) and target (downstream) nodes of a selected edge is done simply by checking which node of the edge is already contained in $T_N$ and assigning that as the target.

**Direct-node layout creation**

In this strategy the tree-growing algorithm constructs sets of eligible source and target nodes. A node is considered to be an eligible source if any edge connected to it has a node which is already included in the growing spanning tree. This is best achieved by using the nodes already contained in the spanning tree as potential target nodes and finding their adjacency nodes, using the base graph that can serve as potential source nodes. The direct-node strategy is formally described below.

1. **Define:**
   - $N$ = set of all nodes
   - $T$ = the spanning tree
   - $T_N$ = set of nodes in the spanning tree
   - $e_i$ = eligible node $i$
   - $E$ = set of currently eligible nodes
   - $A_j$ = set of nodes adjacent to node $i$
   - $n_i$ = node being added to the spanning tree at current iteration

2. **Initiate $T$ and $T_N$.** Insert the sink node into $T_N$.

3. **Compile the set of eligible source nodes $E$.** A node $e_i$ is considered eligible for selection if it is not already contained in $T_N$ and its set of adjacent nodes $A_j$ contains at least one node already contained within $T_N$.

4. **Select the next node to be added to the growing spanning tree $n_i$ from $E$ using the transition rule described in the section titled “Ant Colony Optimisation” on page 4.** Add $n_i$ to $T_N$.

5. **Find the eligible target nodes for the new edge, with $n_i$ as its source node. Compile $A_{n_i}$, the set of nodes adjacent to node $n_i$. Compile the set of eligible target nodes $E$. A node is considered eligible to be a target node if it is both an adjacent node of $n_i$, i.e. an element of $A_{n_i}$, and currently contained in $T_N$.**

6. **If $E$ contains more than one element, select $e_i$ from $E$ with the lowest hop-rank as the target node. If nodes have equal hop-ranks, make a random choice between them. Alternatively take the single element of $E$ as the target node.**

7. **Add a new edge from the source node to the target node to $T$.**

8. **If $T_N$ contains all nodes, stop. Or else return to step 3.**

The direct node strategy places some limitations on the networks that can be produced by the algorithm. Referring to Figure 1, if only edges 2 and 5 have been included in the growing spanning tree, only node 2 is eligible as the next source node. The set of eligible target nodes of node 2 is $E = \{1, 3, 4\}$. Due to the hop-rank heuristic it would only ever be possible to select node 1 as the target node.

**Permutation-edge layout creation**

The permutation strategies are used as alternatives to the previous methods in which the base graph is queried directly. In this case an ant colony algorithm is used to construct a permutation of unique edge identities from which a spanning tree of the base graph is eventually created. The steps that create the edge permutation are listed below.

1. **Compile the set $N$ of all base graph edges. Initialise permutation $P = \emptyset$, the empty set.**

2. **Compile set of eligible edges $E$. An edge is considered eligible if it is not contained in $P$. If $E = \emptyset$, stop.**

3. **Using the transition rule described in the section titled “Ant Colony Optimisation” on page 4, select the next edge $e_i$ from $E$ to be added to the permutation. Concatenate $e_i$ to the end of $P$.**

4. **Return to 2.**

Once a permutation has been composed it can be used to construct a layout by simply using the order of the edges in the permutation as the order in which to add edges to a growing spanning tree. The permutation-edge layout construction, which is very similar to its direct-edge counterpart, is described below.

1. **Define:**
   - $T$ = the spanning tree
   - $P$ = the permutation
   - $T_N$ = the set of nodes in the spanning tree
   - $E$ = set of currently eligible edges
   - $n_s$ = the target node of the new edge
   - $n_i$ = source node of the new edge

2. **Initiate $T$ and $T_N$. Insert the sink node into $T_N$.**

3. **Compile $E$, the set of eligible edges. Similar to the direct-edge strategy, an**
edge is considered eligible if \( T_N \) contains one of its nodes.

4. Iterate \( \mathbf{P} \). The first edge encountered in \( \mathbf{P} \) which is also in \( E \) is selected as the next edge to add to \( T \); however, it is not added to \( T \) at this point as its direction is only determined during the next step.

5. Identify the source node \( n_i \) and target node \( n_j \) of the new edge by selecting \( n_i \) as the node which is already in \( T_N \). Now add the \( n_i \) to \( T_N \).

6. Add \( e \) to \( T \).

7. If \( T_N \) contains all nodes, stop. Or else return to 3.

Assume \( \mathbf{P} = \{12345\} \). Then, referring to Figure 1, after two iterations both edges 1 and 2 have been added and \( E = \{4, 5\} \). Iterating through \( \mathbf{P} \) encounters edge 4 prior to 5, so edge 4 is the next one to be added to the spanning tree.

**Permutation-node layout creation**

In this case spanning tree layouts are constructed using permutations of unique node identities. An ant colony algorithm is used to create the node permutations as described below.

1. Compile the set \( \mathbf{N} \) of all base graph nodes. Initialise permutation \( \mathbf{P} = \emptyset \), the empty set.

2. Compile set of eligible nodes \( E \). A node is considered eligible if it is not contained in \( \mathbf{P} \). If \( E = \emptyset \), stop.

3. Using the transition rule described in the section titled “Ant Colony Optimisation” on page 4, select the next node \( e_i \) from \( E \) to be added to the permutation. Concatenate \( e_i \) to the end of \( \mathbf{P} \).

4. Return to 2.

From the node permutation a spanning tree can be constructed by adding nodes to the spanning tree in the same order that they appear in the permutation, as described below.

1. Define:
   - \( \mathbf{N} \) = set of all nodes
   - \( T \) = the spanning tree
   - \( \mathbf{P} \) = the permutation
   - \( T_N \) = set of nodes in the spanning tree
   - \( e_i \) = eligible node \( i \)
   - \( E \) = set of currently eligible nodes
   - \( A_i \) = set of nodes adjacent to node \( i \)
   - \( n_j \) = node being added to the spanning tree in the current iteration

2. Initiate \( T \) and \( T_N \). Insert the sink node into \( T_N \).

3. Compile the set of eligible source nodes \( E \). A node \( e_i \) is considered eligible if it is not already contained in \( T_N \) and its set of adjacent nodes \( A_i \) contains at least one node already contained in \( T_N \).

4. Iterate \( \mathbf{P} \). The first node \( n_j \) encountered in \( \mathbf{P} \), which is also in \( E \), is selected as the next source node to be added to the growing spanning tree. Add \( n_j \) to \( T_N \).

5. Determine the target node for the new edge. Compile \( A_{n_j} \), the set of nodes adjacent to node \( n_j \). Compile the set of eligible target nodes \( E \). A node is considered eligible as the target node if it is contained in both \( T_N \) and \( A_{n_j} \).

6. If \( E \) contains more than one element, iterate over \( \mathbf{P} \). The first node \( n_j \) encountered in the iteration, such that \( n_j \) is in \( E \), is taken as the target for the new edge. Alternatively take the single element of \( E \) as the target node. Add \( n_j \) to \( T_N \).

7. Add a new edge from the source node \( n_i \) to the target node \( n_j \) to \( T \).

8. If \( T_N \) contains all nodes, stop. Or else return to step 3.

Note that in the direct-node method, the hop-rank heuristic was used to select the target node, while in this case the permutation is used to choose the target node by selecting the first node encountered in \( \mathbf{P} \). This heuristic decision again places some restriction on the spanning trees that can be produced, as demonstrated using Figure 5. Figure 5(a) shows the base layout of a network, while Figure 5(b) shows a spanning tree of this network which cannot be created by the permutation-node approach. This is due to the fact that for node 4 to connect to node 3, rather than node 2, node 3 has to appear before node 2 in the permutation. Then, however, node 5 will also connect to node 3. The opposite is also true – if node 2 appeared before node 3 in the permutation, then both nodes 4 and 5 will connect to node 2.

**Layout completion**

Once a spanning tree has been created, all of the available edges are to be added to complete the network. These edges have to be reintroduced in such a way that cycles are not formed. The adjacency node technique described in the section titled “Directional design of the network” on page 3 is used to avoid cycle formation. The source and target node selection of an edge is performed using the hop-rank heuristic, choosing the target node as the one with the lowest hop-rank. If the hop-ranks of the nodes are equal, the direction of the edge is determined randomly. This technique is used for all the strategies investigated in this paper.
Objective function

It should be noted that under certain conditions an infeasible solution may be obtained. Most notably, the maximum allowable cover depth may be exceeded if a layout which results in excessive excavation is produced. For this reason a Penalty function formulation of the objective function is used to guide the ants away from the infeasible solution space as much as possible. The objective function is then:

\[
\text{Minimize } P = C + \alpha \sum_{i} g_i \tag{7}
\]

Where:

- \( P \) = the penalised fitness value
- \( C \) = cost of the sewer network
- \( g_i \) = violation of constraint \( i \), 0 if unviolated
- \( \alpha \) = a sufficiently large constant to ensure feasible solutions have a better fitness than infeasible solutions

The network cost function \( C \) is obtained using the following cost function:

\[
C = \sum_{i=0}^{N} K_i (d_i, E_i^\text{ave}) + \sum_{j=0}^{M} K_j (h_j) \tag{8}
\]

Where \( K_i \) is a unit cost function for pipes and \( K_j \) is a unit cost function for manholes. The unit cost functions used in this study are as proposed by Afshar et al (2011):

\[
K_i = 1.93a^{3.43}d_i + 0.812E_i^{1.53} + 0.437d_iE_i^{1.47}
\]

and

\[
K_j = 41.46h_j
\]

If a feasible solution is found, the value of the second term in Equation 7 will be zero.

In all the algorithms described above the sink node is static. However, the algorithms can be modified to allow for dynamic sink node placement with relative ease. The required modifications are summarised below:

- **Direct-edge**: Modify the spanning tree construction algorithm to have the ants initially select any edge, and assume its end with the lowest ground elevation is the sink node.
- **Direct-node**: Add an additional initial decision for the ants where a sink node has to be selected from the list of all nodes.
- **Permutation-edge**: Similar to the direct-edge case, modify the spanning tree construction algorithm to use the first edge in the permutation as the starting edge, again using its lowest end as the sink node.
- **Permutation-node**: Use the first node in the permutation as the sink node.

RESULTS

Three example networks with varying size and topology characteristics were created to test the effectiveness of the proposed layout optimisation strategies. The four proposed ACO strategies combined with the heuristic hydraulic optimisation algorithm of De Villiers et al (2017), as well as the ACO-TG algorithm of Moeini and Afshar (2012), are used to solve each example network. The example networks proposed by Moeini and Afshar (2012) all have the same topology and only vary in size and specify multiple outlet nodes. It is the intention of this study to investigate the effects different topology and network sizes have on the performance of different layout creation strategies, which the examples of Moeini and Afshar (2012) do not allow. Furthermore, the multiple outlet nodes are not supported here. This does not affect the generality of the algorithm, as the same layout creation strategy can be applied from multiple outlets simultaneously without any modifications. The inclusion of multiple outlets does have the undesired effect of effectively reducing the size of the network under consideration, as, with two outlet nodes, two entirely independent sub-networks of the base graph are produced of approximately equal size due to the single outlet constraint and acyclic nature of the layout creation algorithms. Consequently, their example problems are not employed. Instead their algorithm is reproduced and applied to the three example networks proposed here for comparison.

Algorithm parameters for the ACO-TG algorithm are as used by Moeini and Afshar (2012) for examples of a similar size. The proposed ACO algorithms were calibrated using 500, 1 000, 2 500, 5 000 and 10 000 for potential generation limits with population sizes of 10, 20, 50 and 100, and the values which resulted in the best average fitness selected. Evaporation rates were chosen which resulted in a gradual convergence of the optimisation, to avoid rapid convergence to local optima. The proposed example networks are all grids of varying size and topological characteristics. The sink node is static and marked with a dark fill in Figures 6, 9 and 12. Relevant elevations are shown and all slopes are assumed to be linear. In all three cases inflow hydrographs are defined at each manhole as if serving 250 very high income residential units, of which the unit hydrograph is shown in Table 1. The peak value is used to scale the unit values listed in the table and leakage value added to provide a base flow rate.

In all cases the evaporation rate \( \rho \) is changed after 80% of the generation limit is reached. The current best solution is allowed to persist for a maximum of 25% of the generation limit, after which it is no longer eligible for pheromone deposit and instead the current generation’s best solution, regardless of its fitness, is used for pheromone deposit. The initial pheromone is always taken as 5.0. Computations were performed using Stellenbosch University’s RHASATSHA HPC: http://www.sun.ac.za/hpc. The results are averaged over 20 randomly initialised runs for each case. The heuristic optimisation method described in De Villiers et al (2017) is seeded with the following constraint values:

- Minimum allowed cover depth \( E_{\text{min}} = 1.2 \text{ m} \).
- Maximum allowed cover depth \( E_{\text{max}} = 10 \text{ m} \).
- Minimum allowed slope \( S_{\text{min}} = 0.01 \).

### Table 1 Very high income residential unit hydrograph

<table>
<thead>
<tr>
<th>Hour</th>
<th>Peak (ℓ/min)</th>
<th>Leakage (ℓ/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.15</td>
<td>1.69</td>
</tr>
<tr>
<td>2</td>
<td>0.05</td>
<td>0.26</td>
</tr>
<tr>
<td>3</td>
<td>0.05</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>0.07</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>0.07</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>0.08</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>0.05</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>0.06</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>0.08</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>0.08</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>0.08</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>0.08</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>0.08</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>0.08</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>0.09</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>0.08</td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>0.09</td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>0.08</td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>0.09</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>0.08</td>
<td></td>
</tr>
<tr>
<td>21</td>
<td>0.09</td>
<td></td>
</tr>
<tr>
<td>22</td>
<td>0.08</td>
<td></td>
</tr>
<tr>
<td>23</td>
<td>0.09</td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>0.08</td>
<td></td>
</tr>
</tbody>
</table>

Volume 60 Number 3 September 2018
Minimum allowed velocity $v_{min} = 0.7\, \text{m/s}$.

Maximum allowed velocity $v_{max} = 5.0\, \text{m/s}$.

Minimum required spare capacity $SC_{min} = 30\%$.

The set of commercially available pipe diameters $\{D\} = \{150\, \text{mm}, 200\, \text{mm}, 250\, \text{mm}, 315\, \text{mm}, 355\, \text{mm}, 400\, \text{mm}, 450\, \text{mm}, 525\, \text{mm}, 600\, \text{mm}, 675\, \text{mm}, 750\, \text{mm}, 825\, \text{mm}, 900\, \text{mm}, 1\, 050\, \text{mm}, 1\, 200\, \text{mm}, 1\, 350\, \text{mm}, 1\, 500\, \text{mm}, 1\, 650\, \text{mm}, 1\, 800\, \text{mm}\}$. PVC is used for pipes with diameters below 450 mm, with a Manning coefficient $n = 0.009$. For all other pipes concrete is used with a Manning coefficient $n = 0.02$.

$\alpha$ in Equation 7 is taken as 1e8.

The value of $\gamma$ (De Villiers et al 2017) was calibrated beforehand.

The layout of the final best solution obtained for each example problem is included. Flow directions of the pipes are shown with arrows. Adjacency nodes are indicated by straight lines at the end of a pipe. The numbers are identities assigned during the optimisation to the elements. Example 3’s solutions contours are not shown for each solution.

**Example Network 1**

The first example network, shown in Figure 6, is a small network on a flat surface.

### Table 2 Example Network 1 – parameters and results

<table>
<thead>
<tr>
<th>Algorithm</th>
<th>Generation limit</th>
<th>Population size</th>
<th>$\rho_{initial}$</th>
<th>$\rho_{final}$</th>
<th>$C$</th>
<th>$\gamma$</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACO-TGA</td>
<td>1 000</td>
<td>100</td>
<td>0.05</td>
<td>–</td>
<td>1 000</td>
<td>–</td>
</tr>
<tr>
<td>Direct-node</td>
<td>1 000</td>
<td>20</td>
<td>0.01</td>
<td>0.02</td>
<td>25</td>
<td>0.15</td>
</tr>
<tr>
<td>Direct-edge</td>
<td>1 000</td>
<td>20</td>
<td>0.0125</td>
<td>0.025</td>
<td>25</td>
<td>0.35</td>
</tr>
<tr>
<td>Permutation-node</td>
<td>1 000</td>
<td>20</td>
<td>0.01</td>
<td>0.02</td>
<td>25</td>
<td>0.35</td>
</tr>
<tr>
<td>Permutation-edge</td>
<td>1 000</td>
<td>20</td>
<td>0.0125</td>
<td>0.025</td>
<td>100</td>
<td>0.15</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Algorithm</th>
<th>Average final cost</th>
<th>Standard deviation</th>
<th>Best cost</th>
<th>Worst cost</th>
<th>Average computation time (s)</th>
<th>Average infeasible solutions (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACO-TGA</td>
<td>18 880.67</td>
<td>569.86</td>
<td>18 662.45</td>
<td>19 362.00</td>
<td>21</td>
<td>0.32</td>
</tr>
<tr>
<td>Direct-node</td>
<td>18 829.87</td>
<td>172.44</td>
<td>18 758.40</td>
<td>18 955.44</td>
<td>18</td>
<td>0</td>
</tr>
<tr>
<td>Direct-edge</td>
<td>18 707.55</td>
<td>74.23</td>
<td>18 673.90</td>
<td>18 756.22</td>
<td>11</td>
<td>0</td>
</tr>
<tr>
<td>Permutation-node</td>
<td>18 715.46</td>
<td>115.83</td>
<td>18 695.22</td>
<td>18 810.33</td>
<td>16</td>
<td>0</td>
</tr>
<tr>
<td>Permutation-edge</td>
<td>18 842.32</td>
<td>70.91</td>
<td>18 835.59</td>
<td>18 937.63</td>
<td>12</td>
<td>0</td>
</tr>
</tbody>
</table>
The main purpose of this example problem is to demonstrate that all algorithms are performing correctly and comparably when the space for heuristic influences is minimal. Table 2 shows the algorithm parameters used during analysis and averaged results for this network. Figure 7 shows the fitness progress with function evaluations of the best result produced by each of the five algorithms. The node strategies have slower computation time than their edge counter parts, as is expected due to the additional target-node decision required by these algorithms. The ACO-TGA has the slowest computation time of all, if only slightly worse than the node algorithms for such a small problem. On average the algorithms perform very similarly, while the node strategies and ACO-TGA are less consistent in their final results. The permutation edge approach had the worst final best solution of all the algorithms. While the ACO-TGA did find the overall best solution, it is only 0.05% better than its nearest competitor. It also had the worst overall final solution. Even for such a small example fitness warping is present in the ACO-TGA, as on average 0.32% of its total solutions were infeasible designs due to poor diameter selections. Figure 8 refers.

Example Network 2

The second example, shown in Figure 9, is a medium-size network. The elevation topology mimics a hill with a flat surrounding.
The algorithm parameters during analysis and averaged results for Example Problem 2 are shown in Table 3. The ACO-TGA performs significantly worse on average, and in its best run, than the other algorithms, while having four times as many function evaluations. It is also far less consistent. The ACO-TGA is again the only algorithm to produce infeasible solutions. On average the direct-edge strategy performs the best, while the three other newly proposed algorithms show comparable average performance. The overall best solution was found by the direct edge-strategy, while the best solution obtained by the ACO-TGA is worse than the worst solution obtained by any of the algorithms. Again, the ACO-TGA requires the most computation and the node strategies are slightly slower than their edge counter parts. The progress of the best solutions is shown in Figure 10. The secondary X-axis shows the function evaluations for the ACO-TGA. The effect of fitness warping can be observed in the ACO-TGA, as for approximately the first 50 000 function evaluations its current solution is worse than the solutions obtained by the other algorithms within 100 function evaluations. The convergence of the ACO-TGA stagnates in this example. This is similar to the results presented by Moeini and Afshar (2012).

Figure 11 shows the direct-edge solution of Example 2.

Example Network 3

The final example, shown in Figure 12, is a large network with a gradual 1% slope. Table 4 shows the algorithm parameters and results for this example. The perm-edge algorithm performs the best of all algorithms. It obtained the best overall solution, has the best average final solution cost, and its worst solution obtained across the 20 runs is better than the best solution obtained by algorithms other than the direct-node. Fitness warping is again observed in the ACO-TGA during its early trials. The ACO-TGA performs significantly worse than the other algorithms on average and it is the most inconsistent in producing final costs. The computation time of the ACO-TGA is again the highest, while the other algorithms have comparable computation times. The rapid improvement in the ACO-TGA, whereafter it plateaus, is consistent with the results of Moeini and Afshar (2012).

Figure 13 shows the perm-edge solution of Example 3, while Figure 14 shows the fitness progression of Example 3.
To further demonstrate the effect of fitness warping, ten trial solutions from the early stages of the ACO-TGA were randomly selected from Example Problem 1. The heuristic hydraulic optimisation algorithm of De Villiers et al. (2017) was then used to perform the optimisation of the layouts of these ten trial solutions. Figure 15 shows the fitness of the ten solutions – on the left obtained by the heuristic method and on the right the fitness obtained by the ACO-TGA. The severity of the fitness warping is observed. A trial solution of the ACO-TGA with a cost of 71 303 024 produces a cost of 31 260 when the heuristic optimisation method (De Villiers et al. 2017) is used to perform the hydraulic optimisation of its layout. This extreme warping of the layout’s fitness due to poor diameter selections is seen in all ten cases, even for such a small example problem.

CONCLUSION

In this paper the optimisation of sewer network layout is investigated in combination with simultaneous hydraulic optimisation of the two-part sewer network design problem. Specifically, the concept of attaching a tree-growing algorithm, as proposed by Moenini and Afshar (2012), to ACO algorithms to produce feasible layouts was advanced. Four selection strategies relying on either edge-based or node-based selection were investigated. The node-based strategies require an additional decision during network construction and these decisions were resolved heuristically, but with the consequence that the network layouts that can be created are restricted. The edge-based strategies do not have this drawback and were expected to yield superior results.

Three example problems were solved using the four proposed spanning tree creation strategies as well as the original ACO-TGA proposed by Moenini and Afshar (2012) for comparison. The ACO-TGA produced, on average, the worst results for all three

### Table 4 Example Network 3 – parameters and results

<table>
<thead>
<tr>
<th>Algorithm</th>
<th>Generation limit</th>
<th>Population size</th>
<th>$\rho_{\text{initial}}$</th>
<th>$\rho_{\text{final}}$</th>
<th>$c$</th>
<th>$\gamma$</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACO-TGA</td>
<td>2 000</td>
<td>200</td>
<td>0.05</td>
<td>–</td>
<td>1 000</td>
<td>–</td>
</tr>
<tr>
<td>Direct-node</td>
<td>5 000</td>
<td>50</td>
<td>0.004</td>
<td>0.01</td>
<td>100</td>
<td>0.3</td>
</tr>
<tr>
<td>Direct-edge</td>
<td>5 000</td>
<td>50</td>
<td>0.005</td>
<td>0.02</td>
<td>100</td>
<td>0.25</td>
</tr>
<tr>
<td>Permutation-node</td>
<td>5 000</td>
<td>50</td>
<td>0.003</td>
<td>0.01</td>
<td>100</td>
<td>0.1</td>
</tr>
<tr>
<td>Permutation-edge</td>
<td>5 000</td>
<td>50</td>
<td>0.004</td>
<td>0.01</td>
<td>100</td>
<td>0.35</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Algorithm</th>
<th>Average final cost</th>
<th>Standard deviation</th>
<th>Best cost</th>
<th>Worst cost</th>
<th>Average computation time (s)</th>
<th>Average infeasible solutions (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACO-TGA</td>
<td>110 660.85</td>
<td>69 775.71</td>
<td>86 981.20</td>
<td>180 721.21</td>
<td>19 m 41 s</td>
<td>1.576</td>
</tr>
<tr>
<td>Direct-node</td>
<td>81 998.65</td>
<td>2 849.34</td>
<td>80 819.48</td>
<td>84 292.63</td>
<td>10 m 23 s</td>
<td>0.002</td>
</tr>
<tr>
<td>Direct-edge</td>
<td>85 418.28</td>
<td>3 064.40</td>
<td>83 365.34</td>
<td>88 133.87</td>
<td>9 m 45 s</td>
<td>0.011</td>
</tr>
<tr>
<td>Permutation-node</td>
<td>87 047.27</td>
<td>9 560.85</td>
<td>83 828.32</td>
<td>96 908.87</td>
<td>11 m 8 s</td>
<td>0.002</td>
</tr>
<tr>
<td>Permutation-edge</td>
<td>80 690.40</td>
<td>3 914.31</td>
<td>78 349.31</td>
<td>82 616.73</td>
<td>11 m 11 s</td>
<td>0.031</td>
</tr>
</tbody>
</table>
examples. The method used in the ACO-TGA of simultaneous layout and element size selection led to severe fitness warping in the initial trial solutions. It is difficult to say definitively that this is the only or dominant factor for the poor performance of the ACO-TGA, especially in the larger examples, but it is certainly a significant drawback which warrants caution in development of future algorithms.

For the other layout creation strategies, combined with the heuristic optimisation method of De Villiers et al. (2017), performance depended on the example in question and no clear winner emerged. The edge-strategies, while not always the best option, consistently produced results at least comparable to the best strategy for all examples, making them attractive options. The most important outcome of the results is the importance of the effects the minor changes in selection strategy had on the final result. The permutation-edge strategy performed significantly better than any other for the large example problem, while performing the worst for the small problem. This can only be ascribed to the heuristic differences in the algorithms and makes a strong case for further investigation into alternative heuristics, specifically specialised heuristics coupled to terrain topology and problem size. It is recommended that at this stage no method be discarded from the investigation and, where possible, further alternatives be developed. This is due to the observed importance of heuristics and the ability to apply different heuristics to different selection strategies, which may lead to significantly improved results.

Although no clear indicator as to which selection strategy is more effective has been found in this study, the results have offered valuable insight into network optimisation in general, and indicate that further investigation into sewer network layout optimisation is warranted. Further, it has been demonstrated that the strategy where a single algorithm is used for both layout and element size selection simultaneously

![Figure 13 Perm-edge solution of Example 3](image-url)
leads to fitness warping of early trial solutions. The set of eligible element sizes could be limited in an intelligent way, for example using the diameters of the heuristic method of De Villiers et al. (2017) to reduce eligible diameter sizes to those around the size obtained heuristically, to reduce the severity of the fitness warping in, especially but not limited to, the early trial solutions. The alternative approach (of using a separate algorithm for each sub-problem) does not suffer from this drawback and should, if the challenge of computational efficiency can be overcome as with the proposed hybrid algorithms presented in this article, be favoured in future work.

REFERENCES


INTRODUCTION

The quantification of wind load uncertainties provides the basis for reliability assessment and calibration of design wind load standards. The random occurrence of severe wind storms provides the primary basis for reliability-based design procedures. However, the derivation of the wind load proper on the structure from the related storm conditions introduces significant biases and uncertainties into the design process. Whereas wind storm conditions are closely related to the strong wind climate of the region, standardised design load procedures have a direct bearing on the additional uncertainties, including provision for all design conditions within the scope of the standard.

Probabilistic wind load models, which represent the uncertainties inherent in the design wind load formulation, are used to calibrate wind load standards in order to achieve target levels of reliability. During the updating process of the South African loading code from SABS 0160:1989 to the current SANS 10160:2010 (republished in 2011), a lack of substantiating information regarding wind load uncertainties was identified by Retief and Dunaiski (2009). The existing South African probabilistic wind load model as presented by Kemp et al (1987) and used in the calibration of SABS 0160:1989 resulted in low reliability requirements for wind loads, as evidenced by comparison with European wind load models and calibration (Gulvanessian and Holicky 2005). Furthermore, investigation of international wind load models revealed scant background information and details regarding the development of those models. The need was therefore clear for an investigation of wind load uncertainties and the development of a new probabilistic wind load model.

This paper forms part of a series of investigations to update reliability provisions for wind loading in SANS 10160 to account for the South African strong wind climate and wind load uncertainties. Reliability models for strong winds are presented by Kruger et al (2013 a; b). The historic development of strong wind characteristics and the reliability basis for wind loading are presented by Goliger et al (2017), including differentiation in the presentation of the wind climate into the spatial representation of the characteristic wind speed and a temporal model for the random nature of extreme wind. Mapping of the characteristic or basic wind speed for design purposes is presented by Kruger et al (2017). This paper provides probability models for the main sources of variability and uncertainty of wind loading. The final paper incorporates the probability models into a wind load reliability model that could be used to derive a wind load partial factor and to assess the effects of the proposed changes to SANS 10160 on wind loads across the country (Botha et al 2018).
The ultimate purpose of this paper is to lay the foundation for the development of a probabilistic wind load model and the reliability assessment of SANS 10160:2011 using the best available representation of South African wind load uncertainties. To this end this paper presents a general framework for the investigation of wind load component uncertainties and a summary of the results obtained from investigations of those uncertainties within the South African context. The current lack of rational and transparent reliability models of design wind loads is discussed, and serves as the primary motivation for the development of a new model. A generalised overview of the investigations by Botha et al. (2014; 2015; 2016) regarding the quantification of wind load component uncertainties is then presented. Both the uncertainties related to the South African strong wind climate and the wind engineering models used to derive wind loads from the fundamental values of the basic free field wind speed are considered.

WIND LOAD PROBABILITY MODELLING

The concept of the “wind loading chain” was introduced by Davenport (1961) and forms the basis of the general design wind load formulation used in most major international wind load standards. The basic probabilistic interpretation of the wind load chain used in the South African wind load standard is given as:

\[ w = Q_{ref} \cdot c_r \cdot c_a \]  

(1)

In this formulation the wind load on a structure \( (w) \) is defined as the product of the reference gust free-field wind pressure \( (Q_{ref}) \), the local terrain roughness factor \( (c_r) \) and the pressure coefficient \( (c_a) \). In the general Davenport formulation additional wind load components are included to provide for general model uncertainty \( (c_w) \) or other sources of uncertainty. The level of approximation of the wind loading chain formulation may be improved by considering additional factors such as the effects of the surrounding topography, the wind directionality and the dynamic response of the structure.

Davenport (1983) emphasised the importance of reliability treatment of wind loads when using the wind loading chain. It was proposed that each wind load component be regarded as a statistically independent random variable, resulting in a full probabilistic description of \( w \). The reliability performance of the codified semi-probabilistic design wind load \( w_d \) may then be assessed by combining the uncertainties of the individual components as expressed by the reliability performance function:

\[ g = w_d - Q_{ref} \cdot c_r \cdot c_a \]  

(2)

The probability of failure is then given as \( P_f = P(g < 0) \), or conveniently expressed in terms of the reliability index \( \beta \):

\[ \beta = \Phi^{-1}(P_f) \]  

(3)

where \( \Phi^{-1} \) is the inverse normal distribution function.

Review of wind load uncertainties

A convenient framework for the probabilistic estimation of wind loads on structures, considering the wind climate, building exposure to the wind, dynamic properties of the structure and its shape and dimensions, is provided by the Probabilistic Model Code (PMC) (ICSS 2001). Included are terrain categories and associated wind parameters, exposure factors, aerodynamic shape factors, and uncertainties and statistics of related random variables. An assessment of the influence of uncertainties on estimates of extreme wind effects and wind load factors is provided by Minciarelli et al. (2001), with consideration of its implications for the provisions of ASCE-7. Subsequent developments, mainly focusing on the implications of tropical storms, have been reported by Vickery et al. (2010). Wind load uncertainties are also reviewed by Hansen et al. (2015), Holmes (2015), Hong et al. (2001) Kasperski (2001; 1993), and Pagnini (2010). The representation of wind load uncertainties in the reliability calibration of design provisions is demonstrated by Baravalle and Kühler (2018).

A review of the development and advances in models for wind pressure on low-rise buildings is provided by Alrawashdeh and Stathopoulos (2015). Although the determination of pressure coefficients related to North American design standards is emphasised, comparison with other standards is included. An outline is given of progress made with the basis for determining pressure coefficients and measurements on which values are based. An extensive set of wind tunnel measurements and full-scale observations are referenced (see also Chen and Zhou (2007)).

The relationship between upwind roughness and turbulence influencing the exposure of low-rise buildings is reviewed by Tieleman (2003a), providing guidance on roughness estimation (Tieleman 2003b). An exposure model which can be used to determine the wind speed to account for homogeneous and inhomogeneous upstream terrain conditions is provided by Wang and Stathopoulos (2007).

Wind tunnel investigations to determine aerodynamic wind pressure coefficients are reported by Endo et al. (2006), Doudak et al. (2009), Zisis and Stathopoulos (2009), among others. An example of a comparison between wind tunnel measurement and various wind load standards is provided by St Pierre et al. (2005).


Existing probabilistic wind load models

In the reliability calibration of semi-probabilistic loading procedures, basic single-parameter expressions are often used (Ellingwood et al. 1980; Kemp et al. 1987; Ellingwood & Tekie 1999; Gulvanessian and Holicky 2005; Holicky 2009). An intermediate approach is to assign probability distributions to the components of the Davenport chain as expressed by Equation (2).

Distribution parameters for the various components and a composite model such as those included in the PMC (ICSS 2001) are listed in Appendix A. Model information includes the probability distribution, the mean relative to the characteristic value \( \mu_X/X_0 \) and the standard deviation \( \sigma_X \), from which a coefficient of variation can be obtained \( V \). Intervals of the distribution parameters indicate ranges of values not only for the basic wind pressure which can be expected to be related to wind climate conditions, but also for the components for converting basic pressure to load. A specific version of the PMC model is used by Gulvanessian and Holicky (2005) to assess Eurocode action combination effects. Another model was developed by Holicky (2009) to validate the Eurocode
wind load partial factor, considering the PMC parameter ranges. A probabilistic model was developed by Milford (1985) to reflect South African conditions and serve as a basis for the introduction of a semi-probabilistic limit states South African Loading Code SABS 0160:1989 (Kemp et al. 1987). The full set of distribution parameters for each model is listed in Appendix A.

A coherent comparison of the diverse set of probability models for wind load can be obtained by performing a First Order Reliability Method (FORM) analysis by applying model component distributions to a performance function based on Equation (2). The upper tail of the composite model distribution can be obtained by varying \( w_d \) parametrically and determining the exceedance probability \( P_F \). It is convenient to express \( P_F \) in terms of the corresponding \( \beta \)-value as given by Equation (3), although this practice should not be confused with the target \( \beta \)-value used to characterise structural reliability. Since all the models are normalised with respect to characteristic values, they can be compared directly, as shown in Figure 1. Normalisation also implies that \( w_d = 1.0 \) represents its characteristic value, and its parametric value represents the partial wind load factor (\( \gamma_w \)) related to the corresponding \( \beta \)-value shown on the graph.

It should be noted that a higher variability results in a flatter graph, implying that a larger value of \( \gamma_w \) (or \( \gamma_w \)) is required to achieve a given \( \beta \)-value. Figure 1 provides a graphic illustration of the range of probability models for wind loading, both as given by the shaded region representing the PMC ranges, and between the various models. There is some clustering between the two models directed towards Eurocode assessment by Gulvanessian & Holický (G&H) and Holický within the mid-to conservative PMC range. The Milford and Kemp models directed towards South African wind load conditions show similar clustering, but with significantly lower values of \( \gamma_w \) required to achieve a given \( \beta \)-value, just breaching the PMC range. The main difference between the Eurocode and South African clusters consists of a shift to the left of the latter cluster that can be related to low values of the relative mean \( (\mu_X / X_k) \). Differences in slope, related to \( \sigma_X \) for the variables, are more subtle.

This investigation is motivated by the large differences between the effective probability models for wind loading as demonstrated by Figure 1, differences between the models for the wind load components, systematic apparent underestimating of reliability requirements obtained from the South African models, and a general lack of background information on the models.

### FREE-FIELD WIND PRESSURE UNCERTAINTIES

#### General approach

The free-field wind pressure is the primary source of uncertainty in the design wind load formulation in South Africa. A review of the strong wind climate of South Africa is provided by Kruger et al (2010; 2012), and probability models for the annual extreme wind speed \( V_{e,i} \) for a set of 76 Automatic Weather Station (AWS) locations \( i \) across the country are given by Kruger (2011) and Kruger et al (2013a). These \( V_{e,i} \) models form the basis of the efforts of this investigation to quantify South African free-field wind uncertainties. A brief summary of the pertinent features of the information on which the models are based is therefore given below.

The spatial resolution of the strong wind climate is improved substantially
in comparison to previous studies by the increase in AWS locations. Observations are resolved into wind-generating conditions, broadly classified into synoptic scale frontal events, meso-scale convective thunderstorms and mixed climate conditions where both synoptic and meso-scale occurrences are observed. AWS observations allow for the determination of 3 s gust wind speeds which capture the influence of all climatic events and can be applied directly in the design procedure. The observed data were fitted to General Extreme Value (GEV) probability models, with shape parameters (κ) ranging between –0.4 (indicating Fisher-Tippett Type III distributions with an upper bound) and 0.5 (Type II, unbounded). Since no systematic trend in the type of distribution could be discerned, the Gumbel (FTI) distribution (κ = 0) is regarded as a reasonable approximation, being conservative or more realistic in comparison to Type III and Type II distributions respectively. Peak-Over-Threshold (POT) models are used to extend the number of observations, applying both the Exponential (EXP) and General Pareto Distribution (GPD). The diversity of the South African strong wind climate is reflected in the number of Extreme Value probability models that are required to fit the data. This diversity is a significant source of uncertainty for the reliability model for wind loading for the country.

Models for V_{adj} are used to derive the 2% fractile values (50-year return period) as the characteristic wind speed (v_{k,i}) for each location (Kruger et al. 2013b). The set of v_{k}-values serves as basis for determining the map of the basic reference wind speed (v_{k,0}) as specified by SANS 10160-3 (Kruger et al. 2017). This information provides an opportunity to revise the 50-year free-field wind pressure probability model (Q_{ref}) (see Equation (2)) developed by Milford (1985) and incorporated by Kemp et al. (1987).

A differentiated approach was followed to estimate the distribution parameters of Q_{ref}. Differences in the probability models for V_{adj} are converted into an estimate of the variability (σ_Q) of Q_{ref}. Systematic differences between the characteristic wind speed v_k and the mapped basic wind speed v_{k,0} are used to estimate the mean (μ_Q) of Q_{ref}. In both cases wind speed is converted to wind pressure using Equation (4).

\[ q = \frac{1}{2} \rho v^2 \]  (4)

The Bayesian hierarchical approach is used on the premise that the parameters of a given distribution are also random variables (Ang & Tang 1984). Values for the hyper-parameters (α_{μ,σ_Q}, β_{μ,σ_Q}) and (α_{σ_Q}, β_{σ_Q}) shown in the hierarchical model (Figure 2, where θ = Q_{ref}) are obtained from the statistics of the 76 samples of the mean (μ_Q) and the variability (σ_Q) respectively. The probability models for μ_Q and σ_Q are regarded as prior distributions from which the posterior distribution for Q_{ref} is obtained. Monte Carlo simulation is used by repeatedly sampling the sets of hyper-parameters to calculate samples of the parent distribution for Q_{ref} from which statistics for (μ_Q, σ_Q) are obtained. The procedure for Bayesian hierarchical analysis is elaborated by Botha et al. (2016).

### Prior distribution for the Variability σ_Q

The probability models for the annual extreme wind speed (V_{adj,0}) are used to derive models for the 50-year free-field wind pressure (Q_{50}) as samples for Q_{ref}.
For this investigation equivalent Gumbel distribution parameters were found for \( V_{a,i} \), for stations where Kruger et al (2013a,b) applied exponential distributions using a regression function on the tail end of the distribution. The regression procedure is described by Botha (2016). \( V_{a,i} \) can then be converted to a 50-year Gumbel distribution \( V_{50,i} \). The set of probability models for \( V_{50,i} = V_{a,i}/V_k \) is presented graphically in Figure 3.

Since \( V_{a,i} \) is based on the best available observations of annual extreme wind, it is assumed that it provides an unbiased model, hence \( Q_{50,i} \) is considered to be unbiased as well. Therefore \( Q_{50,i} \) is used only to determine the dispersion of \( Q_{\text{ref}} \), providing for both the dispersion of 50-year extreme wind at location \( i \) and differences between the set of locations. The standard deviation (\( \sigma_{Q_{50,i}} \)) of \( Q_{50,i} \) is used as the parameter to characterise its dispersion. A histogram of \( \sigma_{Q_{50,i}} \) is presented in Figure 4.

A log-normal distribution is fitted to the data set, with the hyper-parameters as the mean (\( \alpha_{\sigma_{Q_{50,i}}} \)) = 0.27 and standard deviation (\( \beta_{\sigma_{Q_{50,i}}} \)) = 0.07 as shown. The mean standard deviation of 0.27 can be related to the average slope of the \( Q_{50,i} \) probability models depicted in Figure 3.

Prior distribution for the mean \( \mu_Q \)

The ratio of the “true” characteristic wind speed at each AWS location (\( V_k \)) obtained from Kruger (2011) (see Figure 5(a)) and the mapped basic wind speed (\( v_b,0 \)) proposed for SANS 10160-3 (see Figure 5(b) in Kruger et al (2017)) represents a systematic bias in \( Q_{50,i} \). A probability distribution for the bias of \( Q_{\text{ref}} \) can be obtained from the statistics of the set of wind speed ratios, squared to represent wind pressure. A histogram of wind pressure bias is shown in Figure 6, including a fitted normal distribution, with the hyper-parameters as the mean (\( \alpha_{\mu_{Q_{\text{ref}}}} \)) = 0.92 and standard deviation (\( \beta_{\mu_{Q_{\text{ref}}}} \)) = 0.14 as shown. The mean of less than 1 implies a conservative bias of 8% resulting from the specified basic wind speed, although this is less than one standard deviation from an unbiased mean.

Bayesian hierarchical analysis of \( Q_{\text{ref}} \)

Based on the distribution hyper-parameters for the priors of the mean and standard deviation as summarised in Table 1, a set of realisations of \( Q_{\text{ref}} \) were determined as given by the histogram and probability distribution shown in Figure 7, using the Monte Carlo simulation technique outlined above.
The upper tail of the probability distribution for the 50-year base free-field wind pressure $Q_{ref}$ is shown in Figure 8. The exceedance probability is expressed in terms of the reliability index ($\beta$) using Equation (3).

**Assessment**
As the comparison of free-field wind pressure distributions for different regions is not directly applicable, the most important result from this analysis is the comparison of the new model and the previous South African model developed by Milford (1985). It is seen that the new model is significantly different from the Milford model, with a lower systematic bias and higher variability. The high variability of $Q_{ref}$ can be directly related to the statistics for the annual extreme wind ($V_{a,i}$) with an average CoV of 0.12 (ranging from 0.04 to 0.25) for wind speed, and approximately 0.24 for wind pressure. The variability of $Q_{ref}$ indicated by a CoV of 0.31 therefore seems to reflect strong wind conditions for the country. No substantial systematic bias could be expected, except for that resulting from the mapping of the characteristic wind speed. A different picture could be expected from an improvement in the underlying data set due to the extended recording period and geographical coverage of the AWS network. Nevertheless, the present model is based on a substantial improvement in information on the South African wind climate.

**RELIABILITY METHODS FOR INVESTIGATION OF TIME INVARIANT COMPONENTS**

In order to quantify the uncertainties inherent in the time invariant wind load components, potential sources of information needed to be identified and reliability methods for the treatment of that information had to be established. Two such sources of information were identified. The first was the direct comparison of codified values with data obtained from observations, and the second was using the comparison of the codified values stipulated in different major international wind load standards as an indicator of wind load uncertainties.

**Comparison of codified values and observed values**
Direct comparison of model values and measured values is a standard statistical technique, and is the most effective way of quantifying model uncertainties. By this method the codified values of a given
standard are compared to data obtained from wind tunnel and full-scale tests. The statistical parameters estimated by this method predominantly reflect the aleatoric uncertainties of the component. The epistemic uncertainties inherent in measuring data (Holicky et al. 2015) should, however, be noted when wind tunnel or full-scale observations are used. Furthermore, the lack of standardised testing methods and the use of tests of various types may reflect greater variability than the true variability of the pressure coefficients. These factors are described in greater detail by Botha (2016).

The greatest drawback of this method relates to the scope of the investigation. As this study aims to investigate a representative portion of the scope of the South African wind load standards, this method requires a large number of observations from multiple tests as each observation only provides information regarding a specific design situation. It is not feasible to obtain information across a sufficient range of design situations within the sample space of this investigation, and therefore the statistical parameters estimated by this method will be based on a limited set of observations.

**Comparison of wind load standards**

The second source of reliability information used was an expert opinion analysis approach in which the wind load standards are considered as experts (or bodies of experts). The use of comparison wind load standards in reliability analyses is not without precedent (Kasperski 1993; Bashor & Kareem 2009; Kwon & Kareem 2013). Furthermore, expert opinion analysis is a well-established and accepted reliability technique in situations where limited observed data are available, but experts with empirical knowledge may be consulted. By combining these methodologies, an effective reliability technique for the quantification of wind load uncertainties was developed.

Wind load standards are developed by wind engineering experts using the best available information at the time of development. It therefore stands to reason that the wind load standard itself is representative of the empirical and theoretical knowledge from all sources used in its development. By accepting that wind load standards may effectively be regarded as “experts”, it may be concluded that differences between the stipulations of different wind load standards are indicative of the uncertainty in the wind load formulation. The primary advantage of this method is that all design situations which fall within the scope of applicability of the wind load standards may be investigated and a truly representative statistical model of wind load uncertainties may be developed.

There are a number of weaknesses in the use of comparison of standards as a reliability technique. These weaknesses are a result of uncertainties in the codification process of the standards, such as the simplification of the formulation to allow operational models which accommodate a large scope of design situations, and the potential conservatism built into components in order to achieve a desired total wind load from the overall formulation of the standard. Certain measures were taken during the implementation of the method in order to counteract these weaknesses and ensure that the most representative results possible were obtained. To this end a comparative algorithm was developed by Botha (2016) to ensure unbiased sampling. It was verified that the wind load standards considered were completely independent and did not share the same background information, and all the comparisons were done across a large scope of design situations in order to smooth out any specific discrepancies between the standards to obtain results representative of the general case.

**PRESSURE COEFFICIENT UNCERTAINTIES**

Pressure coefficients are subject to a large number of uncertainties, both aleatoric and epistemic. The first and arguably largest sources of uncertainty in pressure coefficient values are the tools used to measure them, namely boundary layer wind tunnel tests. Comparison of more than 200 research papers on low-rise buildings by Uematsu and Isyumov (1999) found significant variation in the results obtained from wind tunnel tests on the same structures. This variability in wind tunnel test results is primarily due to different wind tunnel configurations. Furthermore, it stands to reason that the equivalent static wind load distributions selected for the purposes of codification contribute to the epistemic uncertainty of pressure coefficients. Finally, the inherent aleatoric uncertainty in pressure coefficients also contributes to the variability of the results.

The uncertainties inherent in the codified pressure coefficients given in SANS 10160-3 were quantified using the two methodologies described in the previous section. The investigation was limited to external pressure coefficients resulting in global wind loading on regular low-rise structures. Local peak pressure coefficients such as component and cladding pressure coefficients were not considered. The scope of the structures considered in the investigation included flat, mono- and duo-pitched roof structures with a pitch angle between 0° and 30°.

**Comparison of codified values with observed values**

The first methodology used in the investigation of pressure coefficient uncertainties was direct comparison of codified pressure coefficients with observed values from wind tunnel and full-scale tests. After a rigorous literature study, ten studies were selected from international journals, with a focus on those which presented both full-scale and wind tunnel test results to obtain representative results. The observed values from these studies were compared to the SANS pressure coefficient values. This was done by determining the codified pressure coefficients at the positions across the reference structures used in the above-mentioned studies where pressure coefficients were measured. The measured values could therefore be directly compared to the codified values, and by using standard normalised variables the results across all the observation points for each structure could be sampled to obtain a statistical representation of the global pressure coefficient uncertainty. The full methodology is presented by Botha (2016). In addition to the statistical parameters of the overall structure, the parameters were also calculated separately using the systematic bias values measured on walls and roofs. A summary of these properties is presented in Table 2.

It is shown from the bias values that the measured pressure coefficients are systematically higher than the codified values for roofs and lower than the codified values for walls. This is effectively hidden when the bias across the entire structure is considered, as the roof and wall bias values effectively cancel each other out and a total systematic bias near unity is obtained. The disparity between the roof and wall systematic bias is, however, reflected in the increased variability seen for the overall structure parameters.

**Comparison of international wind load standards**

The second methodology used to investigate pressure coefficient uncertainties was the comparison of codified values from
major international wind load standards. A software package was developed as an implementation of the comparative algorithm developed by Botha (2016) to perform a parameter study of codified pressure coefficients. The wind load standards considered were: SANS 10160-3 (South Africa), BS NA EN 1991-1-4 (United Kingdom), AS/NZS 1170-2 (Australia and New Zealand), ASCE 7-10 (USA), NBCC 2010 (Canada).

A full description of the calculation procedure used in the parameter study is given by Botha (2016). The software package was developed and used to perform over 3.5 million individual comparisons in the parameter study, resulting in 2,512 data points for comparison. Steps were taken to avoid unbiased sampling of the parameter space, resulting in a reduced set of 60 statistically independent values which were ultimately sampled. A hierarchical model was used to combine the information obtained about the pressure coefficient systematic bias and variability into a single posterior representative distribution. The two prior distributions and the final representative distribution of SANS pressure coefficients are given in Table 3.

### Table 2: Mean (μ) and standard deviation (σ) of SANS pressure coefficient systematic bias for each study considered in the investigation

<table>
<thead>
<tr>
<th>Study</th>
<th>Description</th>
<th>Roof</th>
<th>Walls</th>
<th>Overall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>μ</td>
<td>σ</td>
<td>μ</td>
</tr>
<tr>
<td>Levitan et al 1991</td>
<td>Full-scale measurements on TTU building</td>
<td>1.06</td>
<td>0.14</td>
<td>0.83</td>
</tr>
<tr>
<td>Surry 1991</td>
<td>Wind tunnel test results of model of TTU building</td>
<td>1.15</td>
<td>0.19</td>
<td>0.68</td>
</tr>
<tr>
<td>Hoxey 1991</td>
<td>Full-scale measurements on portal frame structure</td>
<td>1.10</td>
<td>0.55</td>
<td>0.57</td>
</tr>
<tr>
<td>Milford et al 1992</td>
<td>Full-scale and wind tunnel measurements on hanger</td>
<td>1.32</td>
<td>0.65</td>
<td>–</td>
</tr>
<tr>
<td>Ginger and Letchford 1999</td>
<td>Full-scale measurements on TTU building with openings</td>
<td>1.28</td>
<td>0.43</td>
<td>0.93</td>
</tr>
<tr>
<td>Uematsu and Isyumov 1999</td>
<td>Compilation of multiple full-scale and wind tunnel tests</td>
<td>1.18</td>
<td>0.36</td>
<td>–</td>
</tr>
<tr>
<td>Endo et al 2006</td>
<td>Wind tunnel test results of model of TTU building</td>
<td>1.21</td>
<td>0.30</td>
<td>0.66</td>
</tr>
<tr>
<td>Chen and Zhou 2007</td>
<td>Full-scale measurements on TTU building</td>
<td>1.48</td>
<td>0.42</td>
<td>0.74</td>
</tr>
<tr>
<td>Doudak et al 2009</td>
<td>Full-scale and wind tunnel measurements on flat roof building with parapets</td>
<td>1.06</td>
<td>0.65</td>
<td>0.69</td>
</tr>
<tr>
<td>Zisis and Stathopoulos 2009</td>
<td>Full-scale and wind tunnel measurements on low-rise duo-pitched roof building</td>
<td>–</td>
<td>–</td>
<td>0.57</td>
</tr>
<tr>
<td>All studies</td>
<td></td>
<td>1.21</td>
<td>0.44</td>
<td>0.71</td>
</tr>
</tbody>
</table>

### Table 3: Distribution parameters of representative pressure coefficient probability model

<table>
<thead>
<tr>
<th>Variable</th>
<th>Distribution type</th>
<th>Mean</th>
<th>Coefficient of variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Systematic bias</td>
<td>Normal</td>
<td>0.98</td>
<td>0.08</td>
</tr>
<tr>
<td>Variability</td>
<td>Log-normal</td>
<td>0.22</td>
<td>0.03</td>
</tr>
<tr>
<td>Pressure coefficients</td>
<td>Normal</td>
<td>0.98</td>
<td>0.23</td>
</tr>
</tbody>
</table>

### Figure 9: Summary of new pressure coefficient probabilistic model

- **Representative posterior distribution**
  - \( \mu \) (code comparison)
    - Distribution: Normal
    - Mean: 0.98
    - Standard deviation: 0.23

- **Prior distribution of systematic bias**
  - \( \mu \)
    - Distribution: Normal
    - Mean: 0.98
    - Standard deviation: 0.08

- **Prior distribution of standard deviation**
  - \( \sigma \)
    - Distribution: Log normal
    - Mean: 0.22
    - Standard deviation: 0.03

- **Geometric average**
  - \( \mu \)
    - Distribution: Normal
    - Mean: 0.99
    - Standard deviation: 0.31

- **Upper bound**
  - \( \sigma \_D \)
    - Distribution: Normal
    - Mean: 0.99
    - Standard deviation: 0.43
New pressure coefficient probability model

A summary of the representative probability distributions of pressure coefficients \(c_a\) calculated in each of the investigations is shown in Figure 9. The new model is not a single model, but rather three separate models consisting of lower and upper bound approximations and an average distribution selected from the range between the limits. Even though a single distribution will be selected and used for reliability assessments of the South African loading code, the final results of this investigation should not be viewed as a single distribution of pressure coefficient uncertainties, but rather as an envelope of possible values that may be narrowed through future research.

The new pressure coefficient model was compared to the corresponding component distributions in existing probabilistic models. The tail-end reliability indices of these distributions are shown in Figure 10. All three new distributions (lower bound, upper bound, and geometric average) defining the pressure coefficient uncertainty envelope are included in the figure.

With the exception of the Milford model, all the models have similar bias values, with a large spread in the variability values. The new models have a significantly greater variability than most of the existing models, resulting in a flatter distribution with lower reliability indices at higher pressure coefficient values. Nonetheless, the region bounded by the new model mostly falls within the JCSS envelope of recommended values. Although the sources used to develop the JCSS models are not clear, they suggest that the high variability obtained for pressure coefficients in this investigation is reasonable.

TERAIN ROUGHNESS FACTOR UNCERTAINTIES

There are two primary sources of uncertainty which contribute to the uncertainty of terrain roughness factors. The first is the use of terrain roughness factor profiles to divide a continuum of possible values into zones based on terrain categories. Whenever a designer selects a representative terrain category for a specific site, the terrain roughness factor is rounded up to the closest approximate terrain category as stipulated in the wind load standard considered. This makes the general wind load standard representation of terrain roughness factors inherently conservative. The second source of uncertainty is the variability inherent in the definition of representative terrain categories. Although the qualitative descriptions of terrain categories provided by most sources are similar, there is no consensus on the exact parameters used to define the terrain roughness factor profiles for those terrain categories. The lack of agreement between wind load standards when considering the same terrain type is an indication of the epistemic variability of terrain roughness factors.

The systematic bias distribution of the terrain roughness factor was determined using the method of the comparison of codified values and observed values. The SANS terrain roughness factor stipulations were compared with a baseline model by Wang and Stathopoulos (2007), which has been verified using experimental data. The method used to quantify the bias prior was described by Botha et al. (2015). Briefly described, the equivalent terrain roughness factor profiles from SANS 10160-3 and the baseline model were compared at 1 m height increments up to a height of 50 m. The systematic bias due to the use of terrain categories was calculated for the zones bounded by the baseline model between the equivalent SANS Terrain Categories A and D. The results of the investigation are given in Table 4. As expected, the terrain roughness factors show a fair degree of conservatism.

In order to quantify the epistemic variability in the selection of terrain roughness profiles, a comparative study of wind load standards was done using the comparative algorithm developed by Botha (2016). The set of standards used in the investigation consisted of SANS 10160-3 (South Africa), EN 1991-1-4 (Europe), AS/NZS 1170-2 (Australia and New Zealand), ASCE 7-10 (USA), NBCC 2010 (Canada) and ISO 4353 2009 (International). Three representative terrain categories were chosen, namely Terrain Categories A, B and C in SANS. To be consistent, the free-field wind speed roughness factors used by SANS, Eurocode, AS/NZS and ISO were squared to make them equivalent to the ASCE and NBCC values based on free-field wind pressure.
The equivalent roughness factor profiles for each representative terrain category after squaring the appropriate values are shown in Figure 11. The comparison of these profiles was used to determine a distribution of the terrain roughness factor standard deviation. The results are given in Table 5.

**New terrain roughness factor component model**

As in the free-field wind pressure and pressure coefficient investigations, the new component model was compared to the equivalent component models from existing probabilistic wind load models. The tail-end component reliability indices of the distributions are shown in Figure 12.

The new model shows lower reliability indices than most of the existing models due to a higher bias. The variability of the new model corresponds well to the existing models. As a result, the new model falls within the JCSS envelope at higher reliability indices, albeit close to the upper limit.

**CONCLUSIONS**

This paper addresses the underlying reliability basis for the design for wind loading on structures, specifically as provided for in SANS 10160-3:2010. The need for such an investigation is due to inconsistencies between the provisions in the standard and corresponding Eurocode procedures and standardised practice for loading in general, specifically with regard to the partial load factor. The diversity of probability models for wind load compounds the difficulties of deriving design procedures.

The approach followed was to use the latest information on the South African strong wind climate to develop probability distributions for wind load. However, wind load probability models include provision for the wind engineering processes of converting free-field wind pressure into the distributed load across the structure.

**Table 6** Distribution parameters of representative primary wind load component models

<table>
<thead>
<tr>
<th>Component</th>
<th>Distribution type</th>
<th>Mean</th>
<th>Coefficient of variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Free-field wind pressure $Q_{ref}$ (50 year)</td>
<td>Gumbel</td>
<td>0.92</td>
<td>0.31</td>
</tr>
<tr>
<td>Pressure coefficients</td>
<td>Normal</td>
<td>0.99</td>
<td>0.31</td>
</tr>
<tr>
<td>Terrain roughness factors</td>
<td>Normal</td>
<td>0.88</td>
<td>0.18</td>
</tr>
</tbody>
</table>
The approach followed to reconsider time-independent components of the Davenport wind loading chain was to compare measurement-based results to standardised model results for limited but representative situations. The scope of the investigation was then extended and complemented by comparing procedures from a set of design standards. The investigation considered provisions for terrain roughness and pressure coefficients for global load on the structure as the most significant time-independent wind load components.

**General results**

The resulting probability models for the three Davenport wind loading components are summarised in Table 6 in terms of the distribution and its parameters. As a general observation, in comparison to the models summarised in Appendix A, these show less conservative bias, with the mean value closer to 1.0, together with larger coefficients of variation. This implies that insufficient reliability is achieved for design procedures based on present models. Reduced conservatism in the bias and larger variability apply not only to the wind climate probability model, which may be ascribed to better information on South African conditions, but also to the time-independent components, which could be expected to be consistent with international practice. The results should therefore be scrutinised to ensure that they are reasonable.

**Free-field wind pressure $Q_{ref}$**

Although the distribution for $Q_{ref}$ can be regarded as uniquely related to the wind climate, the present results lie at the upper limit of the range indicated by the ICSS model. As indicated in the assessment of the results for the $Q_{ref}$ probability model, the basic data for the annual extreme wind speed with an average CoV of 0.12 place a lower limit of about 0.24 on the CoV of wind pressure. The additional uncertainty due to regional differences in wind speed statistics results in an increased CoV for $Q_{ref}$ of 0.31.

The use of a Gumbel distribution for $Q_{ref}$ may be open for review. It is nevertheless clear that regional differences in the EV tail is so significant that refining the Gumbel tail approximation is not justified at this stage. Furthermore, the extreme value extrapolation is limited to return periods well below 1 000 y in accordance with the reliability target for structures within the scope of SANS 10160. The application of the Gumbel distribution is, however, not inconsistent with the practice followed by the models given in Appendix A. The exception is the ICSS model where a log-normal distribution is indicated, which is somewhat unusual for modelling extreme value phenomena. The use of the Gumbel distribution is consistent with the background information provided by Kruger et al. (2013a, b).

A recent assessment of the use of the Gumbel distribution for the calibration of wind load reliability parameters by Baravalle and Köhler (2018) confirms the validity of the approach taken here. In addition to using a Gumbel tail approximation for the calibration of standardised wind load provisions, the combination of the mean Gumbel tail with uncertainties due to regional differences is used to derive a single model for the region as a whole.

Additional examples of the use of the Gumbel approximation for wind storm modelling are provided by Hansen et al. (2015), Xu et al. (2014), and Holicky (2009). Unreasonable GEV distribution parameters for 235 stations across Canada are reported by Hong and Ye (2014), which are similar to the results obtained for South Africa (Kruger et al. 2013a). Harris (2014) advises that Fisher-Tippett Type II and TYPE III asymptotes are not confirmed by wind data, and that the Gumbel distribution is the safe and sensible assumption when the analysis indicates the Type III distribution.

The true nature of EV asymptotes can be investigated from a time series simulation of the parent macrometeorological wind speed from which simulated annual extreme wind speed values are extracted (see Harris 2014; Torrielli et al. 2013). Various EV models can then be assessed against the simulated EV record. Indications are that Gumbel extrapolation to reliability levels for important structures may be inadequate, but may be mildly conservative for the reliability classes provided for in SANS 10160. Such general trends can also be observed from the assessment of alternative EV models reported by Rózsás and Sýkora (2016). Direct simulation through reanalysis of synoptic wind conditions provide an alternative approach to reflect the South African wind climate (Larsén and Kruger 2014), although convective wind storms are excluded from all these simulation techniques.

**Pressure coefficients**

A surprising result is the large uncertainty obtained for the basic set of pressure coefficients for a well-defined scope of structures for which the static equivalent wind loading procedure is widely accepted. Most significant is that this outcome is primarily based on the direct comparison between experimental measurements and code pressure coefficients as summarised in Table 2. The large standard deviation of the model factor for pressure coefficients can be observed not only for individual test series, but also as a result of differences in mean values between data sets, even systematic differences between roof and wall loads. Although the comparison between various standards is intended primarily to allow the scope of application to be covered more comprehensively, it turns out that the model for pressure coefficients is dominated by the model uncertainty based on measurement (see Figure 9). Comparison of standards nevertheless provides useful insight into the differences between reputable design standards.

**Terrain roughness factor**

The most significant contribution to uncertainty in provision for site conditions results from the representation of the wind speed profile as a result of upstream terrain roughness. The comparison is done for clearly defined terrain categories. In this case the uncertainties are primarily derived from the comparison of standardised procedures. The wind speed profiles shown in Figure 11 clearly demonstrate the significant differences between standards, which can only be accounted for as a form of epistemic uncertainty. Direct comparison between standard and experimentally based wind speed profiles mostly result in systematic bias.

Although uncertainties in the representation of terrain roughness for standardised conditions are the lowest for all cases under consideration, with a standard deviation of $\sigma_C = 0.18$, they are nevertheless significant, and again beyond the upper ranges of values reported in the literature.

**The way ahead**

The next logical step is to apply the probability model for wind loading to reassess the reliability performance of South African standard SANS 10160-3. This should be done not so much to establish the implications of the results presented here, but because the need for such an assessment was the primary motivation for the investigation. At the same time, the new map for the basic wind speed proposed by Kruger et al. (2017) results on...
average in a reduction in wind speed. On the other hand, the proposed probability model is bound to result in an increase in the wind load factor to comply with reliability levels that apply to design standards. Simultaneous implementation will limit the impact of the changes while improving the consistency of the intended reliability of structural design.

Each step taken in the assessment of the probability model for wind loading is open to refinement. The procedure presented here could therefore be extended by using updated information on the wind climate, increasing the scope of experimental data to quantify model uncertainty, and even using more advanced models for wind loading components to reduce epistemic uncertainty.

REFERENCES


Journal of the South African Institution of Civil Engineering Volume 60 Number 3 September 2018 27
APPENDIX A
Distributions of Davenport wind load components as provided by various models for wind load (see Equation (1))

<table>
<thead>
<tr>
<th>Table A.1 JCSS probabilistic model (JCSS 2001)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Variable</td>
</tr>
<tr>
<td>Basic wind pressure $Q_{ref}$</td>
</tr>
<tr>
<td>Pressure coefficient $c_p$</td>
</tr>
<tr>
<td>Gust factor $c_g$</td>
</tr>
<tr>
<td>Roughness factor $c_r$</td>
</tr>
<tr>
<td>Total wind pressure $w$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table A.2 G&amp;H probabilistic model (Gulvanessian &amp; Holický 2005)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Variable</td>
</tr>
<tr>
<td>Basic wind pressure $Q_{ref}$</td>
</tr>
<tr>
<td>Pressure coefficient $c_p$</td>
</tr>
<tr>
<td>Gust factor $c_g$</td>
</tr>
<tr>
<td>Roughness factor $c_r$</td>
</tr>
<tr>
<td>Model coefficient $c_m$</td>
</tr>
<tr>
<td>Total wind pressure $w$</td>
</tr>
</tbody>
</table>
### Table A.3 Holický probabilistic model (Holický 2009)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Distribution</th>
<th>Relative mean</th>
<th>Standard deviation</th>
<th>Coefficient of variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic wind pressure $Q_{ref}$</td>
<td>Gumbel</td>
<td>0.80</td>
<td>0.20</td>
<td>0.25</td>
</tr>
<tr>
<td>Pressure coefficient $c_p$</td>
<td>Normal</td>
<td>1.00</td>
<td>0.20</td>
<td>0.20</td>
</tr>
<tr>
<td>Gust factor $c_g$</td>
<td>Normal</td>
<td>1.00</td>
<td>0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>Roughness factor $c_r$</td>
<td>Normal</td>
<td>0.80</td>
<td>0.12</td>
<td>0.15</td>
</tr>
<tr>
<td>Total wind pressure $w$</td>
<td>Gumbel</td>
<td>0.64</td>
<td>0.24</td>
<td>0.38</td>
</tr>
</tbody>
</table>

### Table A.4 Milford probabilistic model (Milford 1985)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Distribution</th>
<th>Relative mean</th>
<th>Standard deviation</th>
<th>Coefficient of variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic wind pressure $Q_{ref}$</td>
<td>Gumbel</td>
<td>1.02</td>
<td>0.17</td>
<td>0.16</td>
</tr>
<tr>
<td>Exposure factor $c_e$</td>
<td>Normal</td>
<td>0.70</td>
<td>0.14</td>
<td>0.20</td>
</tr>
<tr>
<td>Roughness factor $c_r$</td>
<td>Normal</td>
<td>0.80</td>
<td>0.16</td>
<td>0.20</td>
</tr>
<tr>
<td>Model coefficient $c_m$</td>
<td>Normal</td>
<td>1.00</td>
<td>0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>Directional factor $c_{dir}$</td>
<td>Normal</td>
<td>0.90</td>
<td>0.09</td>
<td>0.10</td>
</tr>
<tr>
<td>Total wind pressure $w$</td>
<td>Gumbel</td>
<td>0.52</td>
<td>0.25</td>
<td>0.48</td>
</tr>
</tbody>
</table>

### Table A.5 Kemp probabilistic model (Kemp et al 1987)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Distribution</th>
<th>Relative mean</th>
<th>Standard deviation</th>
<th>Coefficient of variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total wind pressure $w$</td>
<td>Gumbel</td>
<td>0.41</td>
<td>0.21</td>
<td>0.52</td>
</tr>
</tbody>
</table>
Reliability assessment of the South African wind load design formulation

J Botha, J V Retief, C Viljoen

The representation of the strong wind climate and the reliability calibration of wind loading are the main components of the provisions for local conditions in the South African wind load standard. The reliability implications of new probability models for wind load and an updated map of the characteristic wind speed are assessed in this paper. Wind load probability distributions are based on a combination of new models for free-field wind pressure, pressure coefficients and terrain factors with existing information. The results from both assessments show that the wind load partial factor of 1.3 does not result in adequate reliability performance for typical design situations. A partial factor of 1.6 is recommended. Where an increased partial factor for wind loading will result in a corresponding increase in the design wind load, the introduction of an updated map of the characteristic wind speed for South Africa is shown to result in an overall reduction of wind load. Regional analysis, however, indicates that there are regions in the Western and Eastern Cape that will have increased design wind loads. Combined with an updated partial factor of 1.6, the aggregate increase in design wind loads across South Africa is shown to be 11%.

INTRODUCTION
Provisions for wind loading formed an important component of the advancement achieved by the introduction of the South African Loading Code SANS 10160:2010 to national structural design practice. The considerations for using the Eurocode standard EN 1991-1-4:2005 as the primary reference for SANS 10160-3:2010 for wind loading were recorded by Goliger et al. (2009a; 2009b). Differences between the general strong wind climatic conditions for which EN 1991-1-4 is formulated and the conditions found in South Africa could, however, not be accounted for in SANS 10160-3. An extensive assessment of the South African strong wind climate was required to effect the adaptation of the Eurocode wind loading procedures to South African conditions.

The reliability assessment reported in this paper touches on two major considerations when the Eurocode was adapted to South African conditions and practice for the South African Loading Code. Although the need to adapt environmental loads to local conditions is obvious, substantial efforts are required to characterise these conditions properly in order to derive design measures that are calibrated to South African reliability requirements. The default safety levels on which Eurocode procedures are based are significantly more conservative than the levels on which South African standards are calibrated (Retief & Dunaiski 2009).

In the reliability assessment the geographical representation of the strong wind climate across South Africa, as expressed by the characteristic or basic wind speed (\(v_{BA}\)), is considered, together with a probability model for wind loading that can be used to derive a partial wind load factor (\(\gamma_W\)) for the country as a whole. This paper firstly develops a set of wind load probability models that can be used to assess the reliability performance of wind loading based on revised SANS 10160-3 procedures. Changes in wind loading as a result of the combined effects of the revised specified wind speed and the adjusted load factor are then determined to assess the integral implications of the updated reliability information and wind load design specifications.

GENERAL APPROACH
The first step in the process of reliability assessment is to develop a probability model for wind load for South Africa. The model \((W_{SANS})\) incorporates the probability distribution for the free-field wind pressure \((Q_{ref})\) to represent the complex strong wind climate of the country. This
is combined with other structural wind loading components, as described by the Davenport wind loading chain (Davenport 1961, 1983). The probability models for pressure coefficients $(c_p)$ and terrain roughness $(c_z)$ provided by Botha et al. (2018) are complemented by estimates of secondary wind load components from the literature. Various combinations of new and existing component models are used to derive ranges of reliability results, similar to the indicative models provided by the Joint Committee on Structural Safety (JCSS) Probabilistic Model Code (JCSS 2001).

Assessment of partial factors for wind loading $(\gamma_W)$ is done in two steps: a preliminary derivation of $\gamma_W$ from the wind load model on its own provides an indicative value directly, allowing easy comparison between alternative models. This is followed in the second step by reliability modelling in which structural resistance is parametrically represented and compared directly with design procedures.

The final step in the assessment is to provide an indication of the influence of the combined effect of changes to $\gamma_W$ and the specification of the geographical distribution of the characteristic or basic wind speed $(v_{h0})$ proposed by Kruger et al. (2017).

### Related wind load probability models

Milford (1985) provided both a set of wind component models and an integral model $(W_{Mil})$. An integral model reported by Kemp et al. (1987) was used for reliability assessment of SANS 10160:2010. Gulvanessian and Holický (2005) and Holický (2009) provided component and integral models for the reliability assessment of wind loading for the Eurocode. The JCSS Probabilistic Model Code (JCSS 2001) provides ranges for the distribution parameters of the Davenport components and also serves as a background to the wind load models used to assess the Eurocode. The probability distributions of the various models have been summarised by Botha et al. (2018).

The wind load component and integral probability distributions provide useful information against which $W_{SANS}$ can be assessed. Updated versions of the Milford $(W_{Up-Mil})$, Gulvanessian and Holický $(W_{Up-G&H})$ and Holický $(W_{Up-H})$ models are used to complement the reliability assessment based on $W_{SANS}$. This approach leads to a range of results similar to those of the JCSS model, but incorporating the information on $Q_{ref}$ for South Africa and for $c_p$ and $c_z$ as reported by Botha et al. (2018). The way in which $W_{SANS}$ and the updated models have been derived is outlined below.

The influence of changes in the basic wind speed as provided by Kruger et al. (2017) (see also SANS 2017) in comparison with SANS 10160-3:2011 (SANS 2011b) is used to determine geographical changes in design wind loading, in addition to the influence of the modification of $\gamma_W$. Goliger et al. (2017) provide a review of changes in the representation of wind speed in South Africa in terms of both the updated information and the reliability basis of design.

### Free-field wind pressure

The probability model for the free-field wind pressure $(Q_{ref})$ is based on the set of probability models provided by Kruger et al. (2013) for the annual extreme wind speed $(V_q)$ for a set of 74 recording stations across the country. Numerous extreme value probability models were used to obtain appropriate distributions for the complex strong wind climate. These models were:

- General Extreme Value (GEV) models, predominantly using the Type I distribution (Gumbel)
- The Peak-Over-Threshold (POT) method, including the General Pareto Distribution (GPD) and Exponential Distribution (EXP)
- Mixed climate models for regions where both synoptic and meso-scale strong winds are observed

Due to the diversity of the climate and the limited recording periods, no representative probability distribution for the country as a whole could be established. The observed shape parameters $(\kappa)$, however, varied between values of $-$0.4 and 0.5, indicating both bounded and unbounded distributions, with both cases having reliability as well as phenomenological implications.

The selection of the Gumbel distribution $(\kappa = 0)$ may therefore be accepted as a reasonable generalised approximation for fitting the upper tails of the diverse distributions to the country for the purpose of developing a single representative reliability model. The dispersion of the single model incorporates both the inherent variability of strong wind occurrence and their geographical differences. Although Kruger et al. (2013) accounted for uncertainties due to the selection of appropriate extreme value distributions and limited recording periods for individual positions, representation of all positions by a single model is the most important epistemic uncertainty (Botha et al. 2018).

The use of the Gumbel distribution for wind load calibration is widely reported in the literature (see, for example, Baravalle & Köhler 2018; Hansen et al. 2015; Xu et al. (2014); Holický 2009). Although Simiu et al. (2001) report a better fit of the reverse Weibull distribution than the Gumbel distribution for a set of 100 stations across the United States, it is noted that in many instances the differences were small. A similar survey by Hong and Ye (2014) for 235 stations across Canada concludes that the GEV distribution provides a better fit to the data than the Gumbel distribution. Unrealistically low values for the upper bound for certain regions raise concerns regarding the suitability of GEV models with variable $x$ values for reliability assessment. Using the Gumbel models, good agreement is obtained between site and regional analysis.

The probability model for free-field wind pressure $(Q_{ref})$ is based on extreme value occurrences of wind storms and is therefore time variant. The reference time for $Q_{ref}$ is 50 years (Botha et al. 2018), which is consistent with the 50-year return period characteristic wind speed $(v_{h0})$ specified in SANS 10160-3 (Kruger et al. 2017) and the target reliability index value $(\beta_T)$ as applied in the basis of design (SANS 10160-1 2011a) for South Africa (Retief & Dunaisky 2009). This practice is also consistent with the practice followed by other wind load probability models involved in this assessment.

The target level of reliability for a 50-year reference period is taken at $\beta_T = 3.0$, in accordance with the value introduced by Kemp et al. (1987) for SABS 0160:1989 and maintained for SANS 10160:2010, as recorded by Retief and Dunaisky (2009). This is consistent with the practice followed in, for example, ASCE-7 (Ellingwood et al. 1980), although a more conservative value of $\beta_T = 3.8$ is the default value for the Eurocode (ECCS 1996).

### Probabilistic wind load models

The set of four alternative probabilistic wind load models $(W_{SANS}, W_{Up-Mil}, W_{Up-G&H}, W_{Up-H})$ was developed on the basis of the way in which the underlying probability distributions for the Davenport wind load components were determined. A summary of the compilation of the wind load models from the load components is provided in
The combined exceedance probability for the set of wind load components can be obtained from a First Order Reliability Method (FORM) analysis (see, for example, Ang & Tang 1984; Holický 2009). The multivariate expression for the product of the wind load components \( c_i \) can be set up in a reliability performance function as given by Equation (1), where \( w_d \) is a deterministic design wind load which can be varied parametrically to obtain the relationship between wind load and exceedance probability.

\[
\prod c_i - w_d = 0
\]

Fitting of a Gumbel distribution to the upper tail of the distribution provides a convenient single probability model for \( W_{\text{SANS}} \). The same approach was followed by Milford (1985), Gulvanessian & Holický (2005) and Holický (2009) to derive single expressions for wind load probability. (Note that a direct calculation of the mean and standard deviation of \( W_{\text{SANS}} \) from the mean and standard deviation of the components gives values of 0.65 and 0.48 respectively, as compared with the values of 0.71 and 0.39 given in Table 1.)

### Bayesian updating of existing probabilistic models

As summarised in Figure 1, three additional probability models for wind load were derived by combining the new distributions for the primary wind load components with the distributions provided by Milford (1985), Gulvanessian and Holický (2005), and Holický (2009) respectively. The original distributions for the free-field wind pressure \( Q_{\text{ref}} \) were simply replaced with the model specifically developed for the South African climate. Original models for secondary wind load components were retained.

The pressure coefficient and terrain roughness factor indices in the existing

---

**Figure 1 Summary of the development of the full probabilistic wind load models**

The new probability model for wind loading \( (W_{\text{SANS}}) \) is based on the South African strong wind climate and SANS 10160-3 design models for pressure coefficient \( (c_p) \) and terrain roughness \( (c_d) \) provisions, complemented by models that have been chosen to represent, conservatively, the influence of the additional factors on total wind load uncertainty. The wind directionality factor accounts for extreme wind load observations and probability models that neglect wind direction, resulting in a reduced probability of orthogonal wind loading.

Wind directionality effects were included in the model as a deterministic variable with a value of 0.85. This follows directly from the results obtained from the studies by Ellingwood and Tekie (1999) and Rigato et al (2001) regarding the effects of wind directionality on design wind loads on structures. Although wind directionality certainly has an inherent variability, not enough information was available to estimate the variability accurately with an acceptable degree of confidence. Judging from existing probabilistic models which include directionality effects, such as the Milford (1985) model, the variability is almost negligible in comparison with the variability of the other wind load components. The decision to include the factor as a deterministic variable was therefore deemed to be reasonable.

Other wind load uncertainties, such as the lack of both spatial and temporal correlation of wind pressures across the structure, topographical effects and “hidden safety factors” in the design wind load formulation (Holický et al 2016), were included in the full model through the use of a model uncertainty factor. Although these factors reduce the systematic bias of wind loads, it is difficult to quantify this influence. An upper limit approximation based solely on engineering judgement was made by selecting a normal distribution with a bias value of 0.95 and a nominal standard deviation value of 0.10. The true influence of these factors will decrease the bias further. However, as this decision is primarily subjective, it was decided to err conservatively by not reducing the bias by more than a nominal 5%.

The \( W_{\text{SANS}} \) model is summarised in Table 1 in terms of the distribution parameters for the various wind load components. It is, however, convenient to derive a single probability distribution for \( W_{\text{SANS}} \) to be used in reliability assessment.

---

### Table 1 New SANS 10160-3 probability model \( (W_{\text{SANS}}) \)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Distribution</th>
<th>Relative mean</th>
<th>Standard deviation</th>
<th>Coefficient of variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic wind pressure</td>
<td>Gumbel</td>
<td>0.92</td>
<td>0.31</td>
<td>0.34</td>
</tr>
<tr>
<td>Pressure coefficient</td>
<td>Normal</td>
<td>0.99</td>
<td>0.31</td>
<td>0.31</td>
</tr>
<tr>
<td>Roughness factor</td>
<td>Normal</td>
<td>0.88</td>
<td>0.18</td>
<td>0.20</td>
</tr>
<tr>
<td>Directional factor</td>
<td>Deterministic</td>
<td>0.85</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Model coefficient</td>
<td>Normal</td>
<td>0.95</td>
<td>0.10</td>
<td>0.11</td>
</tr>
<tr>
<td>Design wind pressure ( W_{\text{SANS}} )</td>
<td>Gumbel</td>
<td>0.71</td>
<td>0.39</td>
<td>0.55</td>
</tr>
</tbody>
</table>
models were updated using Bayesian updating. The updated probability distributions were calculated by taking linear combinations of the previous models and the new models using standard combination rules for normally distributed random variables (Holický 2009). The existing models and new models were weighted equally, thereby assigning equal importance to both sources.

The three primary components have the greatest influence on the total uncertainty, and therefore the fact that those components are included in all models makes the models directly comparable. Furthermore, the comparison of these models provides an indication of the sensitivity of the total wind load uncertainty to the inclusion of secondary factors. The models are summarised in Tables 2 to 4. The same parametric FORM analysis used for the derivation of a single wind load model from the component distributions as applied to \( W_{\text{SANS}} \) was applied to each updated model.

### WIND LOADING RELIABILITY ASSESSMENT

The derivation of a probabilistic wind load model \( W_{\text{SANS}} \) reported above reflects the incorporation of diverse uncertainties across a large design space, in terms of both geographical variance and structural configurations, into a single expression given by Equation (2) with the distribution parameters, the mean \( \mu \) and standard deviation \( \sigma \), given in Table 1. Reliability assessment represents a continuation of the process of condensation, but with the inclusion of the pragmatic objective to assess (or derive) a suitable single partial wind load factor \( (y_W) \) that could be used in design practice, such as that used in SANS 10160-1 & 3. The scope is, however, extended, also requiring the joint effects of uncertainties from the set of combined loads and the resistance of the full range of structures.

\[
X = \mu - 0.577 \sqrt{\frac{\sigma}{\pi}} - \frac{\sigma}{\pi} \ln[–\ln(1 - P_f)] \quad (2)
\]

In this assessment the reliability implications of \( W_{\text{SANS}} \) on its own is first explored and compared to the set of updated models \( \{W_{\text{Up-Mil}}, W_{\text{Up-G&H}}, W_{\text{Up-H}}\} \). Indicative values for \( y_W \) are derived as the starting point for an extended assessment in which the combined effects of permanent load \( (G) \) and parametric representation of resistance \( (R) \) are included. Throughout the process a distinction should be made between the reliability models of the basic variables \( (W, G, R) \) and the selection of the design parameter \( (y_W) \) which are directly related to standardised design.

Through the use of numerous probabilistic wind load models, the reliability assessment performed serves as a sensitivity study in which the influence of the wind load uncertainty on the reliability performance of the South African loading code’s design functions is determined. A range of reliability requirements is obtained by using four models from different sources, similar to the range of values obtained when using the JCSS model (2001). The influence of the probabilistic models used can be clearly established, allowing a more informed decision regarding the choice of the wind load partial factor.

**Comparison of wind load probability models**

A direct comparison of the probability distributions for the four models as given in the final row of Tables 1 to 4 is provided in Figure 2. It is convenient to express the exceedance probability \( (P_f) \) in terms of a reliability index value \( \beta = \Phi^{-1}(1-P_f) \), where \( \Phi \) is the cumulative normal distribution function. Since all the wind load components are normalised to the characteristic values, the parametric value of the wind load \( w_d \) (see Equation 1) corresponding to \( \beta \) represents the ratio of design to characteristic wind load \( (w_d/w_k) \), which is the same as the load factor \( (y_W) \) for the value of \( \beta \) and \( P_f \).

Design standards allow the separation of the target reliability \( \beta \) into two parts \( \beta_T \) for loads and \( \beta_R \) for resistance respectively, based on the respective sensitivity factors \( a_L = 0.7 \) and \( a_R = 0.8 \) (ISO 2394:2015 and Eurocode EN 1990:2002); in ASCE7-10: 2003 the sensitivity factors are reversed to \( a_L = 0.8 \) and \( a_R = 0.7 \). Once again, the simplification of sensitivity factors into fixed values, as opposed to variable values based on load ratios, is a requirement in order to develop a single representative model effectively. The effects of the sensitivity factors are, however, not ignored.
Partial factors for wind loading ($\gamma_W$) can then be read off directly from the graph at values of $\alpha \beta = 0.7 \times 3.0 = 2.1$ (or $P_T = 0.018$) for South Africa, with the corresponding value of 2.66 as obtained for the Eurocode target reliability of 3.8 (or $P_T = 0.0039$), as shown in Figure 2. Values for $\gamma_W$, assuming these sensitivity factors and calculated from the Gumbel Equation (2), are listed in Table 5. However, values may also be obtained from the graph for other sensitivity factors as required.

The results in Figure 2 indicate the range of values for $\gamma_W$ obtained from the alternative new probability models. The general increase of values in comparison with those based on the original models demonstrates the effect of underestimating wind load uncertainties. By considering all four models, an indicative range of the partial factor values required to provide adequate reliability performance is established. For the South African baseline target reliability level this range is between 1.45 and 1.72.

When a value of $\alpha = 0.8$ is used, in accordance with ASCE-7, the values of $\gamma_W$ increase by 13%. When the adjustment also includes provision for resistance by taking $\alpha_R = 0.7$ to determine $\gamma_R$, the combined effect reduces to 10% – 5% for the resistance coefficient of variation ($w_R$) of 0.1 – 0.25. This should be compared with the 19% difference in the range of $\gamma_W$ values obtained from the different models for $W$.

A comparison of the new model envelope with the JCSS envelope is shown in Figure 3. The new model envelope overlaps the JCSS envelope on the upper bound, where higher partial factors are required for a given target reliability level. Provision for the specific South African strong wind climate and improved estimates for the time-invariant components of the wind load model both contribute to the narrowing of the range of results, in comparison with the broad JCSS envelope. More refined estimates based on additional information on component uncertainties should lead to further narrowing of this range.

### Reliability assessment considering combination of actions and resistance

The preliminary and indicative nature of deriving point estimate values of $y_W$ can be improved by extending the reliability performance function to include related basic variables. For this reason a more detailed assessment was performed using a combination of wind and permanent load structural resistance. Load combinations with other variable loads, such as imposed loads, were not considered, but the same method may be used to extend the assessment by including those load combinations.

The assessment was performed using the same method as used by Retief and Dunaiski (2009) in the previous reliability calibration of SANS 10160. The method consists of determining a single graph which represents the global safety factor (GSF) required to achieve a target level of reliability ($\beta$). Wind load standard design functions are then assessed through comparison with the GSF to ensure acceptable reliability performance. The primary

### Table 5 Partial factors required to achieve target reliability indices using the new models

<table>
<thead>
<tr>
<th>Model</th>
<th>$y_W (\alpha \beta_{SA} = 2.10)$</th>
<th>$y_W (\alpha \beta_{EU} = 2.66)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$W_{SANS}$</td>
<td>1.72</td>
<td>2.21</td>
</tr>
<tr>
<td>$W_{Up-Mil}$</td>
<td>1.45</td>
<td>1.86</td>
</tr>
<tr>
<td>$W_{Up-G&amp;H}$</td>
<td>1.48</td>
<td>1.89</td>
</tr>
<tr>
<td>$W_{Up-H}$</td>
<td>1.69</td>
<td>2.12</td>
</tr>
</tbody>
</table>

Figure 2 Results from the multivariate FORM analysis of new models

Figure 3 Comparison of the JCSS and new reliability model envelopes
advantage of this method is that partial factors can be adjusted and the reliability performance assessed without recalculating the GSF.

The method is based on a reliability performance function \( g(X) \), as given in Equation (3), in which the limit state is expressed as a simple linear combination of the basic variables, namely the structural resistance \( R \), permanent loads \( G \) and wind loads \( W \). A summary of the probabilistic models for the basic variables is given in Table 6. The assessment was done using the four full models derived earlier in this paper, as well as the model used in the previous reliability assessment of SANS 10160 by Retief and Dunaiski (2009) for comparison. It should be noted that the coefficient of variation or structural resistance was parametrically varied in 5% increments in order to determine the reliability requirement for different construction materials.

\[
g(X) = R - (G + W) = 0 \quad (3)
\]

The reliability requirement against which design functions are assessed is found by obtaining an inverse FORM solution to Equation (3) for a given target reliability. From the results the GSF is obtained as the ratio of the characteristic values of the resistance \( R_k \) and the combined permanent \( G_k \) and wind loads \( W_k \), as shown in Equation (4). By parametrically varying the ratio of wind actions to total actions \( \chi \) as defined in Equation (5), the reliability requirement is obtained for the full range of combinations of permanent and wind loads.

\[
GSF = \frac{R_k}{G_k + W_k} \quad (4)
\]

\[
\chi = \frac{W_k}{G_k + W_k} \quad (5)
\]

By using the general target reliability of \( \beta = 3.0 \) for the South African loading standard, the reliability requirement \( GSF_R \) may be determined. Figure 4 shows the reliability requirements obtained using the new wind load models, as well as the model used for the previous reliability assessment of the standard (Kemp et al 1987), denoted as the SABS model. A coefficient of variation of resistance of \( w_R = 0.15 \), which is representative of typical reinforced concrete structures, was used in the assessment.

A clear disparity is seen in the reliability requirements obtained using the new models and the SABS model. The low reliability requirement obtained using the SABS model led to the adequacy of the model being questioned. The inconsistency of the SABS model with general wind load probability models was confirmed and traced to an error in transferring results from background investigations (Botha 2016).

As the reliability requirements of the new wind load models are different, assessing the performance of a design function \( f(\chi) \) against the reliability requirements \( GSF_R \) obtained using the four models provides a good indication of how the uncertainty of the wind load affects the total reliability performance of the standard. The general equation for the design function for the combination of permanent and wind actions \( f(\chi) \) used in SANS 10160 (2011a) is given in Equation (6) in terms of the dimensionless load ratio \( \chi \).

\[
f(\chi) = y_G(1 - \chi)y_G + \chi y_W \quad (6)
\]

A noticeable feature of \( GSF_R \) shown in Figure 4 is its convex shape, where \( GSF_R \) is initially somewhat reduced due to the reduced probability that both the permanent and variable actions deviate substantially from the characteristic value. This well-known non-linearity of the performance function (Equation (3)) is the reason why dual linear design functions (Equation (6)) are often stipulated. In the case of SANS 10160–3, two sets of partial load factors \( (y_G, y_W) \) for permanent \( G \) and wind \( W \) loads are stipulated: the (STR) load

---

Table 6 Probability models for representative basic variables used in the reliability assessment

<table>
<thead>
<tr>
<th>Variable</th>
<th>Source</th>
<th>Distribution</th>
<th>Relative mean</th>
<th>Standard deviation</th>
<th>Coefficient of variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural resistance</td>
<td>Retief and Dunaiski (2009)</td>
<td>Log-normal</td>
<td>1.00</td>
<td>0.10</td>
<td>0.10</td>
</tr>
<tr>
<td>Permanent load</td>
<td>Retief and Dunaiski (2009)</td>
<td>Normal</td>
<td>1.05</td>
<td>0.11</td>
<td>0.10</td>
</tr>
<tr>
<td>Wind load</td>
<td>Kemp et al (1987)</td>
<td>Normal</td>
<td>0.41</td>
<td>0.21</td>
<td>0.52</td>
</tr>
<tr>
<td></td>
<td>SANS</td>
<td>Normal</td>
<td>0.75</td>
<td>0.41</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>Updated Milford</td>
<td>Normal</td>
<td>0.65</td>
<td>0.33</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>Updated Gulv/Holicky</td>
<td>Normal</td>
<td>0.67</td>
<td>0.33</td>
<td>0.49</td>
</tr>
<tr>
<td></td>
<td>Updated Holicky</td>
<td>Normal</td>
<td>0.83</td>
<td>0.36</td>
<td>0.43</td>
</tr>
</tbody>
</table>

Figure 4 Reliability requirement \( GSF_R \) for \( \beta = 3.0 \) for different models using a coefficient of variation of resistance \( w_R = 0.15 \).
case generally applies, with partial factors 1.2, 1.3, and for situations where permanent load dominates, i.e. for low values of $\chi$, the partial factors for the STR-P load case are 1.35, 1.0. Compliance is required for the most stringent load case. The sufficiency of the partial factor for wind loading $\gamma_w = 1.3$ for the STR load case is considered here, based on the set of wind load probability models derived above.

A series of parametric reliability evaluations was done to consider the effects of load combination and structural resistance. Table 7 provides a summary of the representative parametric range of models for structural resistance, as expressed by the coefficient of variability ($w_R$), and the partial factors for the STR and STR-P load cases. The resistance factor ($\gamma_R$) corresponds to a target level of reliability for resistance that is scaled by the sensitivity factor for resistance $\alpha_R = 0.8$, resulting in $\beta_R = (0.8)(3.0) = 2.4$ (or $P_f = 0.0082$), and therefore the factor varies for different values of the coefficient of variation of resistance $w_R$. The values for $\gamma_R$ will be reduced by 3% – 8% if $\alpha_R = 0.7$ in accordance with ASCE 7-10, with a proportional change to $f(\chi)$ as provided by Equation (6). The sensitivity of the reliability performance to $\alpha_R$ is clearly small in comparison with the difficulty of matching the nonlinear reliability requirement shown in Figure 4 with the linear design function $f(\chi)$. This is confirmed by subsequent results.

A reliability assessment using the method described above was performed using each of the four new probabilistic wind load models in turn. The reliability requirements obtained are shown in Figure 5 for parametrically varied values of the coefficient of variation $w_R$. The SANS design functions were plotted over the reliability requirements obtained, with the STR partial factor for wind loads increased.

### Table 7 SANS 10160-1 (2011a) basic variable partial factors

<table>
<thead>
<tr>
<th>Variable</th>
<th>Partial factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w_R = 0.10$</td>
<td>1.26</td>
</tr>
<tr>
<td>$w_R = 0.15$</td>
<td>1.42</td>
</tr>
<tr>
<td>$w_R = 0.20$</td>
<td>1.59</td>
</tr>
<tr>
<td>$w_R = 0.25$</td>
<td>1.79</td>
</tr>
<tr>
<td>STR Permanent load</td>
<td>1.20</td>
</tr>
<tr>
<td>Wind load</td>
<td>1.30; 1.60; 1.90</td>
</tr>
<tr>
<td>STR-P Permanent load</td>
<td>1.35</td>
</tr>
<tr>
<td>Wind load</td>
<td>1.00</td>
</tr>
</tbody>
</table>

![Figure 5](image-url) Reliability performance of SANS 10160 design functions for $\beta = 3.0$ across a parametric range of resistances
parametrically from the present value of 1.3 in increments of 0.3. This was done in order to assess the adequacy of the current partial factor value and, in the case that it is not sufficient, to determine what partial factor value would result in adequate reliability performance of the design functions.

It is immediately clear from Figure 5 that the existing partial factor for a wind load of $\gamma_{W, STR} = 1.3$ is not acceptable for the required reliability performance of $\beta = 3.0$ across the entire range of resistance values considered. From this result it is clear that $\gamma_{W, STR}$ should be adjusted.

It is also noteworthy that the partial factor value of 1.6 corresponds well to the partial factor values obtained from the preliminary reliability assessment performed using direct FORM analysis of the wind load models as presented in the previous section and summarised in Table 5.

**INFLUENCE ON DESIGN WIND LOADS**

The influence of an adjustment of the partial factor for wind load ($\gamma_{W}$) on the resulting wind load values provides a direct measure of the practical implications of the reliability reassessment of SANS 10160 - 3: 2011b. However, the extensive revision of the South African strong wind climate and the pending introduction of a revised map for the fundamental free-field wind speed ($v_{b,0}$) should be included in determining changes in the design wind load. Whereas an increase in $\gamma_{W}$ will result in a direct proportional increase in wind load, the underlying gust wind map reported by Kruger et al. (2013b) indicates an overall reduction in characteristic wind speed ($v_{k}$), but with complex geographical features that include regions where $v_{k}$ increases.

This section aims to quantify the influence of these changes on the total design wind load across the country.

The current SANS 10160 wind map and the new drafted map (SANS 2017) are shown in Figure 6. It should be noted that the wind speeds shown on the current wind map include the “gust conversion factor” of 1.4 as stipulated in the code. From inspection it is clear that the new wind map will lead to a significant reduction in wind loads across a large part of the country when compared with the current wind map.

In order to quantify the change in wind load due to the new wind map, it is necessary to convert the gust wind speeds to gust wind pressures. The equation for value of $\gamma_{W, STR} = 1.6$ or higher would result in acceptable reliability performance of the SANS 10160 design functions across this range. This is based on a comparison with the average reliability requirement from the range of values obtained using the four new models. It is clear from the results, however, that for light, wind-sensitive structures with load ratios of $\chi > 0.6$, a partial factor of 1.6 would not be adequate for the average reliability requirement. Further investigation is required in order to develop the most efficient way to treat the reliability performance of these types of structure.

It is also noteworthy that the partial factor value of 1.6 corresponds well to the partial factor values obtained from the preliminary reliability assessment performed using direct FORM analysis of the wind load models as presented in the previous section and summarised in Table 5.
the calculation of design wind loads \((w_d)\) according to the stipulations of SANS 10160-3 is given in Equation 7. As the air density \(\rho\), terrain roughness factors \(c_r\) and pressure coefficients \(c_p\) are constant multiplication factors in this formulation, they are not affected by changing the wind map, and the influence of the ultimate wind loads on structures due to the change in the map may be calculated directly from the change in gust wind pressure.

\[
\frac{1}{2} \rho c_r c_p \]

(7)

The first step in the method was to establish a regional map of the country by overlaying the two maps shown in Figure 6 and defining regions by the resulting overlapping boundaries. Using constant multiplication factors of unity for the air density, terrain roughness factors and pressure coefficients, the gust wind speed values were then converted to design wind pressures. The design wind pressure was calculated for each region using the current wind map \((w_{\text{exist}})\) and the new wind map \((w_{\text{new}})\). The systematic bias \((b_i)\) was then calculated for each region \((i)\) by taking the ratio of the design pressures from the two maps, as shown in Equation 8. By normalising the new wind pressure relative to the current wind pressure, a bias value of less than unity indicates a reduction in total wind load, whereas a bias value of greater than unity implies an increase in total wind load for the region. The regional bias values mapped across South Africa are shown in Figure 7. Finally, the averaged bias across multiple regions \((b_{\text{avg}})\), such as the bias for each province or across the entire country, could be determined by calculating a weighted average of the bias values using the area of each region \((A_i)\), as shown in Equation 9.

\[
b_i = \frac{w_{\text{new}}}{w_{\text{exist}}} \]

(8)

\[
b_{\text{avg}} = \frac{\sum b_i A_i}{\sum A_i} \]

(9)

The area-averaged bias across the country was calculated to be 0.90. The incorporation of the new wind map into the South African wind load standard will therefore result in a total wind load reduction of 10% on average across the entire country. Figure 7 shows that this bias is not evenly distributed across the country. Certain areas, specifically the Free State and large portions of the Northern Cape, Western Cape and Eastern Cape, show a slight increase in the total wind load of approximately 4%. Certain regions in the central Eastern Cape and Western Cape show a significant increase of up to 26%. However, this increase is offset by significant decreases in the average wind load across the remainder of the country, with reductions of between 15% and 35% in the northern parts of the country and a reduction of up to 49% in areas near Beaufort West, due to the anomalous high wind speed region stipulated in the current wind speed map.

In addition to quantifying the influence of the new wind speed map on the total wind load across the country, it is also possible to incorporate the influence of an updated partial factor for wind loads. In the previous section it was shown that the current SANS 10160 wind load partial factor for the STR design function does not meet the required reliability performance, based on a reliability assessment using four reliability models representing the possible range of uncertainties inherent in the SANS wind load formulation. To determine the influence of an updated partial factor, a factored systematic bias \((b_{\text{factored}})\) may be calculated by multiplying the area-averaged bias \((b_{\text{avg}})\) by the ratio of the updated partial factor \((\gamma_{\text{new}})\) to the current partial factor \((\gamma_{\text{w}})\), as shown in Equation 10.

\[
\text{Table 8: Area-averaged bias values of total wind load across South Africa due to the new wind speed map and different partial wind load factors } \gamma_w
\]

\[
\begin{array}{|c|c|c|c|c|}
\hline
\text{Province} & \gamma_w = 1.30 & \gamma_w = 1.45 & \gamma_w = 1.60 & \gamma_w = 1.75 \\
\hline
\text{Eastern Cape} & 0.99 & 1.11 & 1.22 & 1.33 \\
\text{Free State} & 1.04 & 1.16 & 1.28 & 1.40 \\
\text{Gauteng} & 0.84 & 0.94 & 1.04 & 1.14 \\
\text{KwaZulu-Natal} & 0.92 & 1.03 & 1.14 & 1.24 \\
\text{Limpopo} & 0.76 & 0.84 & 0.93 & 1.02 \\
\text{Mpumalanga} & 0.80 & 0.89 & 0.98 & 1.07 \\
\text{North West} & 0.89 & 0.99 & 1.10 & 1.20 \\
\text{Northern Cape} & 0.87 & 0.97 & 1.07 & 1.17 \\
\text{Western Cape} & 0.92 & 1.03 & 1.14 & 1.24 \\
\text{Entire country} & 0.90 & 1.01 & 1.11 & 1.21 \\
\hline
\end{array}
\]
the entire country for different partial factor values is presented in Table 8. It can be seen that an updated partial factor of 1.45 would result in a bias value of 1.01, effectively negating the average reduction in wind loads due to the new wind speed map. Using a partial factor of 1.6 or higher in order to ensure adequate reliability performance of the code, as calculated in the preceding reliability assessment, would result in average wind load increases of 11% or higher.

$$b_{\text{factored}} = \frac{\gamma_{\text{new}} b_{\text{avg}}}{1.3}$$  \hspace{1cm} (10)

When considering $\gamma_w = 1.30$, which is representative of the case where the new wind speed map is used but the wind load partial factor remains unchanged, the bias values for the Gauteng, Limpopo, Mpumalanga, North West and Northern Cape Provinces indicate a significant decrease in total wind load. For the coastal provinces KwaZulu-Natal, and the Eastern and Western Cape, the bias is obtained from aggregation of a range of values, since some regions do experience a significant increase in the total wind load, but the majority of the regions (by area) experience a decrease.

**SUMMARY, CONCLUSIONS AND RECOMMENDATIONS**

The reliability assessment of wind load design needs to consider topics ranging from the random nature of extreme wind storms, through the uncertainties of the load models, to the reliability elements of the load design formulation. A distinction needs to be made between a reliability model ($W$) of the various sources of variability and uncertainty of the underlying load processes and the basic reliability elements of the characteristic wind speed ($v_q$) and load factor ($\gamma_w$) of design procedures. While $W$ remains the best estimate of the aggregate uncertainty of wind loading, $v_q$ and $\gamma_w$ provide a design safety bias to ensure exceedance of the required reliability.

This paper provides a reassessment of reliability models for extreme wind conditions for South Africa and the design procedures implemented in the design standard SANS 10160-3, the sufficiency of the reliability representation in the standard, and the implications of the adjustment of $\gamma_w$ together with ($v_{b,0}$) for wind load on structures.

The reliability implications of two sets of information on wind loading on structures in South Africa, as provided for in SANS 10160-3, are considered in this paper. Mapping of the basic wind speed ($v_{b,0}$) to represent the geographical distribution of the strong wind climate of the country has been proposed by Kruger et al. (2017). Probability models for wind loading that provide for both the time-dependent characteristics of wind storms for the country ($Q_{ref}$) and primary time-independent Davenport wind load components for pressure coefficients ($c_p$) and terrain effects ($c_z$), as specified in SANS 10160-3, are provided by Botha et al. (2018).

The revised wind map for $v_{b,0}$ indicated a general reduction in the wind speed, with significant increases for specific regions. The new models for the primary wind load components indicate that a larger partial load factor for wind ($\gamma_w$) will be required to achieve the target reliability of $\beta_T = 3.0$ applicable to SANS 10160-3. This would result in a systematic increase in wind load on structures for the country as a whole.

The new probability model for wind load ($W_{\text{SANS}}$) results in less conservative bias and dispersion as expressed by the mean and standard deviation (see Table 1) in comparison with the models presented by Kemp et al. (1987) and Milford (1985) on which the present wind load factor $\gamma_w = 1.3$ is based. Figure 2 indicates that better agreement between the new and previous models is obtained when the Milford model is updated ($W_{\text{Up-Mq}}$) to include the present information on the free-field wind pressure ($Q_{ref}$) (see Figure 1 and Table 2). Updated models applied to the assessment of the Eurocode ($W_{\text{Up-GskH}}$, $W_{\text{Up-M}}$) converge into an upper and lower range respectively for the wind load probability model, as shown in Figure 2, and interpreted in terms of first estimates of $\gamma_w$ ranging between values of 1.45 and 1.72, as indicated in Table 5. Figure 3 demonstrates that the range of distributions not only emulates the JCSS model, but also falls within its range of results, albeit close to the upper limit. The result is that the modelling of wind load probability is based on new information, although it is still anchored to the models used for both South Africa and the Eurocode, akin to rational sensitivity analysis. A consensus conclusion is that the present value of $\gamma_w = 1.3$ is clearly insufficient.

The complexity of providing an acceptable and consistent level of reliability through a single partial factor for wind loading is clearly demonstrated by the results given in Figure 5 for a parametric comparison between the required global safety factor ($\gamma_{SFS}$) given by Equation (4) and that provided by a design function (Equation (6)). However, a number of observations regarding the value of $\gamma_w$ may be made. It is clear that the current wind load partial factor of $\gamma_w = 1.3$ provides insufficient values for GSF to achieve the required reliability across the range of conditions represented by Figure 5. A value of $\gamma_w = 1.6$ generally achieves GSF values within the range indicated by the four probability models for wind load, within the mid-range of values for load ratios $\chi$ for typical structures, and for the range of construction materials.

An exceptional situation is for light structures with $\chi > 0.5$ and a low coefficient of variation of resistance $w_{R}$, such as for steel. These structures may be sensitive to both wind loading and achieving insufficient reliability, even for a relatively large value of $\gamma_w$. At the other extreme, structures with large values of $w_{R}$ generally achieve sufficient reliability and may be expected not to be sensitive to wind loading. Provision for such diverse conditions should rather be considered in the materials-based standards, taking cognisance of the probabilistic nature of wind load provisions.

The map for $v_{b,0}$ indicates a general reduction of the characteristic wind speed over large parts of the country, but increases over several smaller regions (see Figure 6), with a geographical average reduction in wind load of 10% (see Figure 7 and Table 8). The most significant observation is that an implementation of $\gamma_w = 1.6$ will reverse the reducing effect of the updated wind map to result in an increase of 11% in the design wind load. A comparison of Figure 7 and Table 8 indicates that regional effects are not effectively captured by breaking down the effects into provinces. The exception is that Gauteng, as both the smallest province and the economic heartland of the country, will be neutrally affected.

The analysis and results presented here follow an engineering process in which information on uncertainties in the standardised design procedures of SANS 10160-3 are compiled, integrated and applied to derive pragmatic design parameters in accordance with the simplified format of characteristic wind speed $v_{b,0}$...
There are significant opportunities to improve the reliability model for wind loading for use in devising more effective reliability-based design procedures. The obvious topic is to update the information on the South African strong wind climate to include the accumulated records in terms of both the recording period and the geographical distribution, allowing for more extensive extreme value modelling. Significant uncertainties in the time-invariant wind load components are indicative of the potential for improving the design standard, for example as reflected by recent efforts related to the Eurocode (see, for example, Steenbergen and Vrouwenvelder 2015).

REFERENCES


Factors influencing the quality of design documentation on South African civil engineering projects

E Akampurira, A Windapo

INTRODUCTION
Construction projects typically go through four phases, namely planning, design, construction and operation and maintenance. Linked to these phases are three crucial roleplayers: the client who requires an engineering solution to a problem, the designer responsible for defining and detailing the solution to the client’s problem, and the contractor who is in charge of the execution of the works as detailed by the designer (Emmitt 2007).

The focus of this study is on the set of design documentation that is provided to the contractor to facilitate the execution of the construction works. This documentation includes engineering drawings, bills of quantities, standard specifications, project specifications, services information, materials report, site-specific geotechnical reports and topographical surveys, among others. These documents are a key product of the design process and aim to provide sufficient detail to enable the contractor to efficiently implement the construction phase of the project (Emmitt 2007; Tilley et al 1999).

Good quality design documentation is of particular importance considering the distinct role the design documents play in communicating the design intent (Emmitt 2007) and enabling the estimation of cost (Davis et al 2009). Wahedi (2016) further points out that during the construction phase, the design documents serve various purposes, including serving as legal documents, coordinating the works and as a management tool for resource allocation. Researchers have also repeatedly identified the critical role that good quality design documentation plays in attaining the construction project objectives of quality, cost and time (Chua et al 1999; Yong & Mustaffa 2011).

However, poor quality design documentation has been identified as an ongoing concern in the construction industry in a number of countries: Australia (Tilley et al 1997; Tilley et al 1999; McLennan & Parminster 2004; Slater & Radford 2012; Philips-Ryder et al 2013), Saudi Arabia (Darwish 2005), Japan (Minato 2003) and the UK (Laryea 2011).

Similarly in South Africa, Malinda (2017) explored the association between project documentation and the project outcomes linked to quality, cost and time. The findings of the research suggested that a positive correlation existed between good quality project documentation and positive project outcomes. The study further identified lack of resources due to low fees, inexperienced consultant personnel and a shortage of skilled staff as the top three out of seven factors suggested as contributing to the poor quality of project documentation.

Furthermore, a number of studies focused on South Africa have identified that poor quality design documentation contributes to inefficiencies in construction projects, including rework (Simpeh et al 2011; CIDB 2011; Emuze 2012), project delays (Baloyi & Bekker 2011; Ramabodu & Verster 2013), cost overruns (Baloyi & Bekker 2011; Ramabodu & Verster 2013) and low productivity (Bierman et al 2016).

Despite this, none of the above studies have specifically focused on or undertaken empirical research on the perceived quality of design documentation and the factors that influence this quality in the South African context. The broader scope of this ongoing research seeks to address this gap. Specifically, the aim of the research described in this paper is to identify the key factors that influence the quality of design documentation in South Africa.

DESIGN DOCUMENTATION QUALITY
The International Standards Organisation defines quality as the degree to which a set

Keywords: design documentation quality, design fees, design review, performance, South Africa
of inherent characteristics fulfill requirements (ISO 2005). Within the context of this research, design documentation is a product of the design process, and the contractor is the user of this product. Therefore the quality of design documentation can be gauged by the degree to which it embeds the desired characteristics (attributes) necessary to meet the requirements of the contractor.

Some researchers (Tilley et al 1997; Minato 2003) have suggested quality attributes/indicators for design documentation. These attributes and their definitions are summarised in Table 1.

Factors that influence the quality of design documentation

A comprehensive literature review identified a number of studies that have reported on the factors that affect the quality of design documentation (Love & Li 2000; Minato 2003; McLennan & Parminter 2004; Darwish 2005; Love et al 2006; Abdalaziz 2009; Philips-Ryder et al 2013). A previous paper by the present authors (Akampurira & Windapo 2016) highlights and briefly discusses the factors that were identified and included in this study.

A summary of the identified factors is provided in Table 2. Thirty-four factors influencing the quality of design documentation were identified from the literature and grouped into four categories, namely Client [1], Design firm [9], Industry [4] and Design professional [10].

The factors identified in Table 2 were incorporated into a questionnaire used to collect primary data as part of an empirical investigation. The following sections of this paper present the questionnaire development process, data collection and findings from the statistical analysis of the data received.

METHODOLOGY

Questionnaire design

A quantitative approach was adopted involving the use of a web-based questionnaire which incorporated the findings from the literature review and targeted civil engineering consultants. The questionnaire had five sections addressing different aspects of design documentation quality. The questions were a mixture of both open-ended and Likert-type scale questions.

The questionnaire was pilot-tested for clarity, suitability and appropriateness of the questions. Three practicing civil engineers, one contractor and one academic participated in the pilot test. Revisions were made to the wording of certain questions and general formatting to enhance the flow of the questionnaire. The industry-based practitioners proposed three additional factors that could influence the quality of design documentation:

1. Discounting of professional fees to below the recommended fee as per professional body (ECSA) guidelines.
2. Limited opportunity for new entrants into the profession to gain relevant experiential training.
3. Delays in payment of professional fees by clients.

The additional factors reflect issues that are probably unique to the South African construction environment (Lawless 2005; Maritz 2012; Windapo & Cattell 2013; CESAl 2016) and thus not necessarily picked up in the referenced studies that have been conducted predominately outside of South Africa.

Section A of the questionnaire collected demographic information about the respondents. Section D, consisting of 37 factors, sought to solicit the respondents’ perceptions of the extent of influence the identified factors had on the quality of design documentation. The factors were categorised into four constructs, namely Industry, Client, Design Professional and Design Firm related factors. The respondents were requested to rate on a scale of 1 to 5 their assessment of the level of influence (1 = not influential at all, 2 = slightly influential, 3 = somewhat influential, 4 = very influential, 5 = extremely influential). This paper reports on and analyses the responses to these two sections of the questionnaire.

Method of data collection

The target population for data collection was civil engineers working in firms that are members of Consulting Engineers South Africa (CESA). On civil engineering projects, the civil engineers often perform the functions of design, project management and construction supervision. They are therefore well positioned to comment on the various aspects of design documentation. CESA is the leading umbrella organisation for consulting engineers, and its membership database is publicly accessible (CESA 2017). Firms operating in Gauteng, KwaZulu-Natal and Western Cape provinces were selected from the database. The selection of provinces was based on the fact that a significant amount of the construction activity in South Africa takes place in these provinces.

Four hundred and seventy-seven (477) firms were identified from the CESA database. In instances where a company had more than one registered office in a province, only the office registered as the head office was selected for inclusion in the survey. This was necessary to facilitate efficient management of the data collection process. Subsequently 371 firms met the inclusion criteria and were contacted to participate in the survey.

The web-based questionnaire was sent by e-mail to the contact addresses recorded on the CESA database. The cover e-mail briefly described the objectives of the research and provided a URL link to facilitate the online completion of the questionnaire. The request was that the questionnaire should...
### Table 2 Factors influencing the quality of design documentation

<table>
<thead>
<tr>
<th>Indicators/attributes</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Client (CR)</td>
<td></td>
</tr>
<tr>
<td>C1</td>
<td></td>
</tr>
<tr>
<td>Unrealistic client expectations regarding the time required for the design</td>
<td>✓</td>
</tr>
<tr>
<td>C2</td>
<td>✓</td>
</tr>
<tr>
<td>The quality of the project brief provided</td>
<td>✓</td>
</tr>
<tr>
<td>C3</td>
<td>✓</td>
</tr>
<tr>
<td>No specific person responsible for design coordination and providing information</td>
<td>✓</td>
</tr>
<tr>
<td>C4</td>
<td>✓</td>
</tr>
<tr>
<td>Client’s lack of relevant project experience</td>
<td>✓</td>
</tr>
<tr>
<td>C5</td>
<td>✓</td>
</tr>
<tr>
<td>Numerous changes to client requirements</td>
<td>✓</td>
</tr>
<tr>
<td>C6</td>
<td>✓</td>
</tr>
<tr>
<td>Insufficient and missing information</td>
<td>✓</td>
</tr>
<tr>
<td>C7</td>
<td>✓</td>
</tr>
<tr>
<td>Provision of wrong information by the client</td>
<td>✓</td>
</tr>
<tr>
<td>C8</td>
<td>✓</td>
</tr>
<tr>
<td>Failure to review the design documentation</td>
<td>✓</td>
</tr>
<tr>
<td>C9</td>
<td>✓</td>
</tr>
<tr>
<td>Provision of conflicting information</td>
<td>✓</td>
</tr>
<tr>
<td>C10</td>
<td>✓</td>
</tr>
<tr>
<td>Unrealistic client expectations with respect to time required for construction</td>
<td>✓</td>
</tr>
<tr>
<td>C11</td>
<td>✓</td>
</tr>
<tr>
<td>Client’s insistence to commence construction before completion of the detailed design phase</td>
<td>✓</td>
</tr>
<tr>
<td>Design firm (DF)</td>
<td></td>
</tr>
<tr>
<td>D1</td>
<td>✓</td>
</tr>
<tr>
<td>Lack of quality control practices and procedures</td>
<td>✓</td>
</tr>
<tr>
<td>D2</td>
<td>✓</td>
</tr>
<tr>
<td>Failure to adopt quality management systems, e.g. ISO 9001</td>
<td>✓</td>
</tr>
<tr>
<td>D3</td>
<td>✓</td>
</tr>
<tr>
<td>Failure to provide relevant training to staff</td>
<td>✓</td>
</tr>
<tr>
<td>D4</td>
<td>✓</td>
</tr>
<tr>
<td>Inadequate design review processes</td>
<td>✓</td>
</tr>
<tr>
<td>D5</td>
<td>✓</td>
</tr>
<tr>
<td>Work overload on designers due to low staff levels</td>
<td>✓</td>
</tr>
<tr>
<td>D6</td>
<td>✓</td>
</tr>
<tr>
<td>Poor allocation of time in consideration of available workload</td>
<td>✓</td>
</tr>
<tr>
<td>D7</td>
<td>✓</td>
</tr>
<tr>
<td>Lack of relevant software</td>
<td>✓</td>
</tr>
<tr>
<td>D8</td>
<td>✓</td>
</tr>
<tr>
<td>High staff turnover</td>
<td>✓</td>
</tr>
<tr>
<td>D9</td>
<td>✓</td>
</tr>
<tr>
<td>Inadequate supervision of junior design staff</td>
<td>✓</td>
</tr>
<tr>
<td>Industry (IR)</td>
<td></td>
</tr>
<tr>
<td>E1</td>
<td>✓</td>
</tr>
<tr>
<td>Low design fees</td>
<td>✓</td>
</tr>
<tr>
<td>E2</td>
<td>✓</td>
</tr>
<tr>
<td>Selection of design firms based on lowest price tendered</td>
<td>✓</td>
</tr>
<tr>
<td>E3</td>
<td>✓</td>
</tr>
<tr>
<td>Shortage of civil engineering skills</td>
<td>✓</td>
</tr>
<tr>
<td>E4</td>
<td>✓</td>
</tr>
<tr>
<td>Low emphasis on professional standards</td>
<td>✓</td>
</tr>
<tr>
<td>Design professional (DR)</td>
<td></td>
</tr>
<tr>
<td>F1</td>
<td>✓</td>
</tr>
<tr>
<td>The designer is inexperienced</td>
<td>✓</td>
</tr>
<tr>
<td>F2</td>
<td>✓</td>
</tr>
<tr>
<td>Lack of coordination between different design disciplines</td>
<td>✓</td>
</tr>
<tr>
<td>F3</td>
<td>✓</td>
</tr>
<tr>
<td>Limited time available for checking and coordinating all design documentation</td>
<td>✓</td>
</tr>
<tr>
<td>F4</td>
<td>✓</td>
</tr>
<tr>
<td>Improper use of design software</td>
<td>✓</td>
</tr>
<tr>
<td>F5</td>
<td>✓</td>
</tr>
<tr>
<td>Reuse of design documents and details from previous projects without adequate review</td>
<td>✓</td>
</tr>
<tr>
<td>F6</td>
<td>✓</td>
</tr>
<tr>
<td>Designer’s unfamiliarity with construction techniques and materials</td>
<td>✓</td>
</tr>
<tr>
<td>F7</td>
<td>✓</td>
</tr>
<tr>
<td>The heavy workload on the designer</td>
<td>✓</td>
</tr>
<tr>
<td>F8</td>
<td>✓</td>
</tr>
<tr>
<td>Poor communication between multi-disciplinary teams</td>
<td>✓</td>
</tr>
<tr>
<td>F9</td>
<td>✓</td>
</tr>
<tr>
<td>Failure to understand the client brief</td>
<td>✓</td>
</tr>
<tr>
<td>F10</td>
<td>✓</td>
</tr>
<tr>
<td>Lack of experience on similar projects</td>
<td>✓</td>
</tr>
</tbody>
</table>
be circulated among the civil engineering practitioners in the office.

Where possible, and to increase the number of responses to the questionnaire, the researcher made personal contact with individuals in the firm to encourage them to circulate the questionnaires among their colleagues. In addition, reminder e-mails were periodically sent to all respondents during the data collection period (November 2016 – January 2017).

The use of e-mails to deliver an online questionnaire presented certain challenges, which included bounced e-mails, undelivered e-mails, the inability to confirm if an e-mail had been delivered to the intended recipient’s inbox, delivery of e-mails to junk mail folders and automatic filters/blocks in the recipient’s e-mail settings. Bowen et al. (2010) point out that these occurrences make it difficult to determine the response rates to online surveys. 129 questionnaires were returned, of which 91 had been completed. The completed questionnaires form the basis of the findings reported in this paper.

### Analysis of the data

The survey data were analysed using the statistical analysis program SPSS 24 to obtain descriptive statistics. The reliability of the scale was tested for internal consistency using the Cronbach’s coefficient alpha \( \alpha \) (where \( 0 \leq \alpha \leq 1 \)). Internal consistency refers to the extent to which all the items on a scale measure the same construct (Tavakol & Dennick 2011; Field 2013).

#### Mean score

The overall ranking of the influencing factors as perceived by the respondents was determined using the mean score (MS) of the Likert scale items (Chan et al 2003). The mean score is calculated using the formula

\[
MS = \frac{\sum (f \times s)}{N}
\]

where
- \( f \) = frequency of response to each rating (1–5) for each factor
- \( s \) = score given to each factor by the respondents, ranging from 5 for extremely influential to 1 for not influential at all
- \( N \) = number of responses to that factor.

#### Kendall’s coefficient of concordance

Kendall’s coefficient of concordance \( W \) (\( 0 \leq W \leq 1 \)) was used to evaluate the degree of agreement among the respondents on the ranking of the factors. A value of 0 indicates no trend of agreement among the respondents, while 1 shows total agreement or unanimity. The level of significance (p-value) associated with the test was used to establish whether the degree of agreement among the respondents on the ranking was consistent or not at a 5% significance level. The null hypothesis that there is no significant agreement among the respondents within the same group can be rejected when \( W \) is significant (\( p < 0.05 \)) (Siegel & Castellan 1988; Field 2013).

### SURVEY RESULTS

The following sections present the results of the data collected through an empirical survey. The analysis reported upon in this paper is based on 91 completed questionnaires received from civil engineers.

#### Characteristics of the respondents

**Role in the organisation**

The respondents’ roles in the organisations were diverse and are reflected in Table 3. It can be seen from Table 3 that a significant number of the respondents (46.2%) were design engineers. The respondents, by the nature of their role, would be intimately involved in the production and management of the design documents necessary for construction.

**Professional registration status**

Table 3 presents a summary of the professional registration status of the respondents. Professional registration is an indication of having achieved an acceptable degree of competence as assessed by other professionals in the industry (ECSA 2017). 75.8% of the respondents were professionally registered with the relevant professional bodies in the industry, specifically the Engineering Council of South Africa (ECSA) or the South African Council for Project and Construction Management Professions (SACPCMP). A percentage of the respondents, namely 9.9%, were registered with both professional bodies.

**Years of experience**

Figure 1 presents the categorised years of experience of the respondents. It shows...
that the respondents’ years of experience in the construction industry ranged from 2 to 40 years, with the average being 17.4 years. 81.3% of the respondents had more than 5 years’ experience.

As a consequence, the information provided by the respondents was deemed reliable considering their roles, years of experience and professional registration status in the construction industry.

Nature of projects
The respondents based their responses to the factors that influence the quality of design documentations on a variety of civil engineering construction projects. The majority of these were completed predominately within the previous five years (92.3%). This demonstrates that the projects referred to were carried out in the recent past, and this reduced the errors that could have arisen from a recall bias (Neuman 2014). These projects ranged from roads, urban civil infrastructure (e.g. stormwater, foul sewer) to mining works. Similarly, the construction project costs varied from R1 million to R2 billion. This illustrates that the responses were based on a variety of construction projects typical of the civil engineering construction industry in South Africa.

Ranking of factors that influence the quality of design documentation
The overall ranking of the factors that influenced the quality of design documentation as perceived by the respondents is presented in Table 4. This ranking is based on the calculated mean score. Each of the items on the scale attained a mean score of $M \geq 3$. “3” on the scale represented “somewhat influential.” This implies that respondents perceived that all the identified factors influenced the quality of design documentation, albeit to varying degrees.

Cronbach’s coefficient $\alpha$ was used to assess the internal consistency of the measurement scale. Cronbach’s coefficient $\alpha \geq 0.7$ is an indicator of a reliable scale (Field 2013). The Cronbach’s coefficient obtained for this survey was $\alpha = 0.933$, which indicated that the measurement scale was internally consistent at the 5% significance level.

Table 4 shows that the key factors influencing the quality of design documentation on civil engineering projects are low design fees, selection of design firms according to the lowest price tendered, limited time available for checking and

| Table 4 Overall ranking of factors that influence the quality of design documentation |
|---------------------------------|-----|--------|--------|
| Factor                          | Mean | Rank   | Category   |
| Low design fees                 | 4.494 | 1   | Industry   |
| Selection of design firms based on the lowest price offered | 4.44 | 2 | Industry |
| Limited time available for checking and coordinating all design documentation | 4.378 | 3 | Design professional |
| Discounting of professional fees to below the recommended fee as per ECSA guidelines | 4.352 | 4 | Industry |
| Poor communication among multi-disciplinary teams | 4.319 | 5 | Design professional |
| Unrealistic client expectations regarding time required for the design | 4.297 | 6 | Client |
| Lack of coordination between different design disciplines | 4.256 | 7 | Design professional |
| Numerous changes to client requirements | 4.256 | 8 | Client |
| The level of quality of the project brief provided by the client | 4.231 | 9 | Client |
| Inadequate design review processes | 4.176 | 10 | Design firm |
| Unrealistic client expectations with respect to time required for construction | 4.154 | 11 | Client |
| Designer’s failure to understand the client brief | 4.121 | 12 | Design professional |
| Client’s insistence on commencing construction before completion of the detailed design phase | 4.110 | 13 | Client |
| Designer is inexperienced | 4.099 | 14 | Design professional |
| The heavy workload on the designer | 4.044 | 15 | Design professional |
| Re-use of design documents and details from previous projects without effective review by the designer | 4.033 | 16 | Design professional |
| Lack of quality control practices and procedures in the generation of design documentation | 4.022 | 17 | Design firm |
| Poor allocation of time, taking into consideration the current workload | 4.011 | 18 | Design firm |
| Low emphasis on professional standards | 3.967 | 19 | Industry |
| Improper use of design software | 3.967 | 20 | Design professional |
| No specific person on client team responsible for coordinating and providing the relevant information to the design team | 3.956 | 21 | Client |
| Inadequate supervision of junior design staff | 3.955 | 22 | Design firm |
| Shortage of engineering skills within the civil engineering industry | 3.945 | 23 | Industry |
| Designer’s lack of experience on similar projects | 3.934 | 24 | Design professional |
| Insufficient and missing input information from the client | 3.934 | 28 | Client |
| Designer’s unfamiliarity with construction techniques and materials | 3.934 | 25 | Design professional |
| Provision of conflicting information by the client | 3.911 | 26 | Client |
| Provision of wrong information by the client | 3.901 | 27 | Client |
| Client’s lack of relevant project experience | 3.868 | 29 | Client |
| Low staff levels leading to work overload on designers | 3.824 | 30 | Design firm |
| Failure to provide relevant training to staff | 3.747 | 31 | Design firm |
| Client’s failure to review the design documentation | 3.697 | 32 | Client |
| Limited opportunity for new entrants into the profession to gain relevant experiential training | 3.549 | 33 | Industry |
| High staff turnover | 3.516 | 34 | Design firm |
| Delays in payment of professional fees | 3.500 | 35 | Client |
| Failure to adopt quality management systems, e.g. ISO 9001 | 3.253 | 36 | Design firm |
| Lack of relevant software to undertake the production of the design documentation | 3.099 | 37 | Design firm |

Kendall’s Coefficient of Concordance $w$ 0.127
Level of significance 0.000

Mean scores: 5 = extremely influential, 4 = very influential, 3 = somewhat influential, 2 = slightly influential, 1 = not influential at all
coordinating all design documentation, discounting of professional fees to below the recommended fee as per ECSA guidelines and poor communication among multi-disciplinary teams.

Kendall’s coefficient of concordance W for the overall ranking was W = 0.127, p < 0.05, therefore the null hypothesis that there is no significant agreement among the respondents within the same group can be rejected. This suggests that there was an acceptable degree of agreement among respondents regarding the ranking of the factors.

**DISCUSSION OF SURVEY RESULTS**

The respondents identified low design fee as the most important factor influencing the quality of design documentation in the South African construction industry. This is in agreement with Malinda (2017), who noted that lack of resources as a result of low fees contributed to the poor quality of the project documentation. This finding is also in line with the results reported in studies undertaken in Japan (Minato 2003) and Australia (Tilley et al. 1999; Slater & Radford 2012). It also supports the assertion by Bubshait et al. (1998) that there is a strong correlation between low design fee and deficiencies in the design documentation. Project fees influence the allocation of resources, e.g. time and personnel dedicated to the project design process, and invariably the quality of the product of this process.

The emphasis on lowest price tendered as a selection criterion for professional service providers has been identified as a challenge in the South African construction industry (Weidemann 2014; CESA 2016). The results of this study reflect this concern in view of the high ranking assigned to the factor selection of design firms based on the lowest price tendered (ranked 2). Slater (2012) identified a similar challenge in the Australian sector, and as a mitigating measure, suggested that the quality and capabilities of the consultants should be included in the weighting during the selection process. The South African government has recently issued new procurement guidelines that seek to encourage the incorporation of quality criteria in addition to the existing price and preference criteria in the selection of professional service providers (SAICE 2016).

This study found that discounting of professional fees to below the recommended ECSA fee guidelines was ranked among the key factors that influence the quality of design documentation. The fee guidelines provide a benchmark and an approach to determining the appropriate professional remuneration that is required to provide a quality service (CESA 2016). Research in South Africa indicates that the discounting of fees is prevalent in the industry and is driven by the procurement prescripts, which emphasise competition on the basis of price (Okonkwo 2014; Weidemann 2014; CESA 2016).

The late payment of professional fees has been identified as a significant concern in the South African construction industry (Maritz 2012; CESA 2016). In this study delay in payment of professional fees was ranked low (ranked 35 out of 37). This low ranking could be attributed to the fact that often at the design stage, the professional has an agreed fee that guides his allocation of resources. The late payment of fees manifests itself later on in the lifespan of the project and effectively only affects the cash flow of the business and not necessarily the resource allocations to the design task. This inference requires further investigation.

Respondents assigned a low degree of influence to the factor “Failure to adopt quality management systems, e.g. ISO 9001” (ranked 36). However, in interpreting this result, it is noted that all the respondents of this survey worked in CESA-registered firms where a quality management system is a prerequisite for membership. Further research might consider inclusion of companies that do not have a quality management system.

The respondents’ relatively high ranking of the factor “Inadequate design review processes” at 10 was unexpected considering that the respondents’ firms all had a quality management system in place. The design review process is an important aspect of the QMS and aims to provide an avenue for checking that the client requirements have been met (ISO 2005). While this ranking does not imply the absence of design review practices, it does suggest an area of weakness. Further research could be undertaken to establish how design reviews are conducted by engineering consulting firms in South Africa.

Shortage of engineering skills in the civil engineering industry was ranked relatively low at 23. By comparison, some industry reports in South Africa (Lawless 2005; Windapo & Cattell 2013; Malinda 2017) have highlighted skills shortage as a challenge to the sector as a whole, which is especially evident in the client bodies and government departments. It is acknowledged that these studies did not focus specifically on the quality of design documentation; nevertheless their findings reflect the challenges in the industry. The low ranking might be explained by the fact that the survey did not involve respondents from the client bodies. Furthermore, it is probable that many of the respondents of this survey were not intimately involved with the staff recruitment process and thus were unable to relate to this factor as a direct challenge.

**SUMMARY AND CONCLUSION**

This research explored the relative importance of factors within the design process that influence the quality of design documentation in South Africa. Thirty-four factors were identified from a comprehensive literature review and formulated into a questionnaire. An additional three factors were proposed during the pilot testing phase of the questionnaire. The consolidated thirty-seven factors were incorporated into a final web-based questionnaire administered to South African civil engineering consultants practising in CESA member firms.

Analysis of the data collected shows that the following factors, presented in order of importance, have the greatest influence on the quality of civil engineering design documents: low design fees; selection of design firms by lowest price tendered; limited time available for checking and coordinating all design documentation; discounting of professional fees to below the recommended fee as per ECSA guidelines; and poor communication among multi-disciplinary teams. The following factors are considered to be the least influential: limited opportunity for new entrants into the profession to gain relevant experiential training; high staff turnover; delays in payment of professional fees; failure to adopt quality management systems, e.g. ISO 9001; and lack of appropriate software to produce the design documentation.

It is, however, necessary to recognise the limitations of this research. The CESA member firms’ data bases were used to identify the survey respondents. These member firms are required to meet certain criteria (e.g. implementation of a quality management system) which may
ACKNOWLEDGEMENTS

The authors gratefully acknowledge the time and input of all the respondents to the questionnaire. The authors would also like to express their appreciation for the support received from Kantey and Templer consulting engineers during the course of these studies.

REFERENCES


Malinda, M 2017. Quality of project documentation as a major risk source in infrastructure projects in South Africa. MEng (Civil Engineering) Dissertation. Stellenbosch University.


SAICE (South African Institution of Civil Engineering) 2016. Focus on: National Treasury standard for infrastructure procurement and delivery management. Midrand: SAICE.


A numerical investigation on hydro-mechanical behaviour of a high centreline tailings dam

M Naeini, A Akhtarpour*

The high rate of failure in tailings dams reflects the inadequacy of conventional stability and seepage analyses for evaluating their performance, especially during construction. In this article, the hydro-mechanical behaviour of a high centreline tailings dam during staged construction is evaluated by coupled stress-pore pressure analyses. The effects of factors such as foundation permeability, raising rate, anisotropic permeability of slimes and changes in the slope of dykes on the structural response are investigated. In these analyses, a combination of realistic conditions, including saturated-unsaturated seepage of materials, consolidation and loading due to staged construction are simulated using SIGMA/W software. The results of this study indicate that foundation permeability and anisotropic permeability of slimes have a significant influence on development of pore pressure in impoundment, while changing the downstream slope of dykes causes substantial displacements.

INTRODUCTION

Proper disposal of mining waste is one of the biggest concerns of major mines in the world. In many mines (especially in copper and gold mines), more than 99% of the processed materials are turned into residual wastes including tailings (Villavicencio et al 2013), which generally contain a considerable amount of water and environmental pollutants. Therefore safe and sustainable tailings disposal is crucial for protecting water resources and the environment against contamination hazards. One common method of tailings management is the construction of tailings dams (with coarse-grained tailings) in the form of raised embankments with upstream, downstream or centreline designs. The objective of such structures is to ensure sufficient and safe storage of tailings during and after mining operations (Hamade & Mitri 2013). However, failure of two to five tailings dams per year on average (Davies 2002; Bowker & Chambers 2015) shows that conventional seepage and stability analyses may not encompass all the behavioural aspects of these structures. Consequently hydro-mechanical analyses aimed at determining the actual behaviour of tailings dams during staged construction and making sure of their stability and safe operation are of vital (Saad & Mitri 2010a).

Besides the dynamic studies, which are beyond the scope of this paper, the latest numerical studies carried out on tailings dams can be divided into three groups. The first group includes seepage analyses performed with the objective of locating the phreatic surface, managing the water resources or providing inputs for stability analyses. See, for example, the studies of Rykaart et al (2001) and Yuan and Lei (2015). The second group comprises conventional stability analyses that utilise numerical or limit equilibrium methods to determine the safety factor under static and/or pseudo-static conditions. See, for example, the studies by Ozcan et al (2013), Wei et al (2016) and Zhang et al (2016). Owing to the complex behaviour of tailings dams during staged construction, the third group of numerical studies focuses on hydro-mechanical analyses. For example, Psarropoulos and Tsompanakis (2008) studied the behaviour and stability of three upstream, downstream and centreline tailings dams during staged construction, and found that from the stability and displacement perspective, the downstream method has the best and the upstream method has the worst performance, while the centreline method provides intermediate results. Saad and Mitri (2010a) performed transient coupled non-linear analyses on an upstream tailings dam built of oil

Keywords: tailings dam, centreline method, coupled hydro-mechanical analysis, staged construction

M NAEINI holds a BSc in Civil Engineering and an MSc in Geotechnical Engineering from Ferdowsi University of Mashhad in Iran. As a part of his dissertation, he investigated the numerical behaviour of tailings dams. His research interests include laboratory and numerical investigations on the geotechnical properties of soils and tailings.

PROF DR ALI AKHTARPOUR* holds a BSc in Civil Engineering and an MSc and PhD in Geotechnical Engineering from Amirkabir University of Technology in Iran. He is an assistant Professor at Ferdowsi University of Mashhad in Iran. His research interests include geotechnical earthquake engineering and numerical investigations into geotechnical problems with regard to infrastructure such as dams, foundations and excavations.

MAHDI NAEINI holds a BSc in Civil Engineering and an MSc in Geotechnical Engineering from Ferdowsi University of Mashhad in Iran. His research interests include laboratory and numerical investigations on the geotechnical properties of soils and tailings.

Contact details:
Ferdowsi University of Mashhad
Azad Sq
Mashhad
Khorasan Razavi
Iran
T +98 51 3662 7977
E mhd.naeini@gmail.com

* Corresponding author
sand, coal wash and gold tailings in staged construction. This study reported that the highest pore water pressure (PWP) in the impoundment and horizontal displacement in the dam body were observed in the model whose materials had the lowest permeability. It was also reported that total and differential settlement in the embankment dykes must be checked to prevent erosion and overtopping (Saad & Mitri 2010b). Hamade and Mitri (2013) developed a new approach for evaluating the performance of the tailings dam by incorporating its hydro-mechanical behaviour as well as uncertainties in material characteristics. Ormann et al (2013) utilised a fully coupled analysis to place rock dykes at the downstream side of the Aitik tailings dam so that with each raise in elevation the safety factor remained higher than 1.5. In a three-dimensional hydro-mechanical analysis of an upstream tailings dam, Hu et al (2015) reported that, given the tailings permeability variations during the staged construction process, the long-term behaviour of the dam is best determined through coupled analysis.

To the authors’ knowledge, most numerical studies of tailing dams have been conducted with the upstream method and/or a height of less than 40–60 m. However, in most countries, such as Chile, the upstream method is forbidden outright (Villavicencio et al 2013), and in large mines where a high volume of tailings needs to be disposed the use of high tailings dams is inevitable. In this article, the coupled hydro-mechanical (stress-pore pressure) behaviour of a high centreline tailings dam is evaluated by coupled stress-pore pressure analysis, and the effects of important factors such as foundation permeability, raising rate, anisotropic permeability of slimes and dykes slopes on structural response are examined. In these analyses the realistic conditions of tailings dams including saturated-unsaturated seepage of materials, consolidation and loading due to staged construction are simulated with the SIGMA/W finite element software (Krahn 2012). The results of these analyses provide useful information on the performance of this type of tailings dam under different conditions.

THE STUDIED MINE AND TAILINGS DAM

The Sungun porphyry copper mine is located 125 km north-east of Tabriz in East Azerbaijan Province of north-western Iran at 43° 46´ E and 38° 42´ N (Figure 1). Having a proved reserve of about 410 million tons of copper ore with an average grade of 0.67% (Ghasemi et al 2012), this mine is the largest open copper pit in Iran, and it is expected to produce at least 380 million tons of tailings in about 30 years. As a result, extensive studies have been done on the safe and sustainable disposal of tailings in compliance with environmental laws in the basin of Zarnekaabchay Stream located 6 km south-east of the mine. One of the main options for the construction of the Sungun tailings dam was the use of cycloned-sand tailings for elevation raise with the centreline method, which is addressed in this study.

Figure 2 shows the idealised geometry of the dam at the maximum cross-section along with its main components. The main components of this dam are as follows:

1. **Foundation:** To simplify the behavioural analysis and make it applicable to other high centreline tailings dams, the foundation is assumed to have a smooth bottom and a thickness of 10 m (based on field investigations).
2. **Starter dam:** This structure is made of waste rock and an inclined clay core rising to a final height of 70 m. In reality there is a filter layer, which protects the core against piping failure; however this layer was not modelled because of its low thickness (1 m).
3. **Sand dykes:** This region is modelled with underflow tailings and a downstream slope of 4H:1V, an upstream slope of 2H:1V, a crest width of 10 m, and a height of up to 170 m. Considering the design of hydro-cyclones and separation of most of the coarse-grained materials from tailings slurry, impoundment materials
are assumed to consist entirely of fine-grained tailings (slimes). This section is modelled with a length of 490 m in a slope of 0.5%, and its freeboard is assumed to be equal to 5 m at the end of construction. It should be noted that the cycloned-sand/slimes interface does not include any low permeability core/barrier system.

**CONSTITUTIVE LAW AND MATERIALS PROPERTIES**

The geotechnical properties assumed for the materials in the hydro-mechanical analyses are given in Table 1. All the materials are assumed to behave according to the elastic perfectly plastic model. SIGMA/W uses the Mohr-Coulomb yield criterion as the yield function (Krahn 2012):

\[
F = \sqrt{2} \sin \left( \frac{\pi}{3} \right) - \frac{\sqrt{2}}{3} \cos \left( \frac{\pi}{3} \right) \sin \phi - \frac{I_2}{3} \sin \phi' - c \cos \phi' 
\]

(1)

where \( I_2 \) is the second deviatoric stress invariant, \( \theta \) is the lode angle, and \( I_1 \) is the first stress invariant. The plastic potential function \( G \) employed in the software is similar to the yield function, except that in Equation (1) the dilation angle, \( \psi \), is used instead of the effective friction angle, \( \phi' \) (Krahn 2012). The suction influence on the strength can be obtained from Equation (2) (Krahn 2012):

\[
C = c' + (\sigma_a - \sigma_w) \tan \phi' \left( \frac{\theta - \theta_s}{\theta_s - \theta_r} \right) 
\]

(2)

where \( c' \) is the effective cohesion, \( \theta_s \) and \( \theta_r \) are the saturated and residual volumetric water contents, \( \theta \) is the volumetric water content, and \( (\sigma_a - \sigma_w) \) is the matric suction.

The non-linear elastic modulus of sand tailings and the foundation are calculated with Equation (3) (Byrne et al. 1987):

\[
E = kP \left( \frac{\sigma_v'}{\sigma_a} \right)^n = K \sigma_v' \nu 
\]

(3)

where \( \sigma_v' \) is the vertical effective stress, \( P_a \) is the atmospheric pressure, and \( K \) and \( n \) are the coefficient and power. Based on the results of Rowe cell consolidation tests, the average elastic modulus of slimes is calculated to be 7 000 kPa, which is within the range of assumptions or measurements reported by Liang and Elias (2010), Jaouhar et al. (2011), L. Bolduc (2012) and Ferdosi et al. (2015b). Based on the assumptions of L-Bolduc and Aubertin (2014) and Ferdosi et al. (2015a), the elastic modulus of waste rock is assumed to be 50 000 kPa.

To determine the failure parameters, the effective friction angle of sand tailings is obtained from anisotropically consolidated undrained (CK0,U) triaxial shear tests conducted over a range of densities and cell pressures. In addition, given the small difference between \( \phi' \) values of sand tailings and the slimes produced in hard-rock mines (Bussière 2007), the effective friction angle of slimes is assumed to be 35°, which is within the range of results reported by Volpe (1979) and Vick (1990) for copper slimes. In view of the discussions by Ferdosi (2014) and based on the assumption of Ferdosi et al. (2015a), \( \phi' \) of waste rock is assumed to be 45°. Moreover, \( \phi' \) and \( c' \) of the core and the foundation materials are determined based on the results of consolidated drained (CD) triaxial shear tests.

Based on the results of constant head permeability tests, the average vertical permeability of sand tailings with a void ratio of 0.82 is determined to be \( k_v = 4.9 \times 10^{-6} \) m/s. The permeability of slimes is obtained by two methods: (a) measuring directly at the end of every stress increment in a Rowe cell consolidation test, and (b) using the coefficient of consolidation \( (c_v) \), which is within the approximate range of \( 10^{-7} \sim 10^{-8} \) m/s in the stress of 1–640 kPa. The results of consolidation tests show that the effective vertical stress function (void ratio) – saturated permeability of tailings can be well expressed by an exponential function in the form of Equation (4):

\[
\frac{k_v}{k_{v0}} = (\sigma_v')^{-0.363} 
\]

(4)

<table>
<thead>
<tr>
<th>Material</th>
<th>USCS classification</th>
<th>( J_{\text{wet}} ) (kN/m²)</th>
<th>( J_{\text{sat}} ) (kN/m²)</th>
<th>Poisson’s ratio (v)</th>
<th>E (kPa)</th>
<th>( c' ) (kPa)</th>
<th>( \phi' ) (°)</th>
<th>Poisson’s ratio (v)</th>
<th>( k_y ) (m/s)</th>
<th>( k_y / k_{v0} )</th>
<th>Water retention curve</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand tailings</td>
<td>SM</td>
<td>18.5</td>
<td>19.40</td>
<td>0.28</td>
<td>( *K = 628 )</td>
<td>0</td>
<td>38</td>
<td>5</td>
<td>4.9 \times 10^{-6}</td>
<td>0.8</td>
<td>Van Genuchten (1980)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \alpha = 2.408 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \beta = 1.76 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \theta_s = 0.45 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \theta_r = 0.04 )</td>
</tr>
<tr>
<td>Slimes</td>
<td>ML</td>
<td>–</td>
<td>19.2</td>
<td>0.30</td>
<td>7 000</td>
<td>0</td>
<td>35</td>
<td>0</td>
<td>1.4 \times 10^{-7}</td>
<td>to ( 8.7 \times 10^{-9} )</td>
<td>0.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( D_{\alpha} = 0.0015 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( D_{\alpha} = 0.03 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \beta_s = 0.48 )</td>
</tr>
<tr>
<td>Waste rock</td>
<td>GW</td>
<td>22</td>
<td>–</td>
<td>0.23</td>
<td>50 000</td>
<td>0</td>
<td>45</td>
<td>1</td>
<td>1.6 \times 10^{-3}</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( D_{\alpha} = 4.7 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( D_{\alpha} = 26.2 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \beta_s = 0.38 )</td>
</tr>
<tr>
<td>Compacted clay (core)</td>
<td>CL</td>
<td>18.3</td>
<td>19.7</td>
<td>0.3</td>
<td>20 000</td>
<td>15</td>
<td>32</td>
<td>0</td>
<td>6.8 \times 10^{-9}</td>
<td>0.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Van Genuchten (1980)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \alpha = 14.29 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \beta = 1.14 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \theta_s = 0.41 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \theta_r = 0.2 )</td>
</tr>
<tr>
<td>Foundation</td>
<td>CL</td>
<td>–</td>
<td>20.7</td>
<td>0.3</td>
<td>1 040</td>
<td>0</td>
<td>45</td>
<td>1</td>
<td>6.0 \times 10^{-6}</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Van Genuchten (1980)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \alpha = 29.64 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \beta = 1.94 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \theta_s = 0.44 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \theta_r = 0.08 )</td>
</tr>
</tbody>
</table>

* Parameters of Equation (3)
where $k_{wi}$ is the permeability coefficient at each stress increment, $k_{oi}$ is the initial permeability coefficient and $\sigma'_i$ is in kPa. Additionally, given the layered nature of tailings (Sarsby 2013), the anisotropy ratio ($k_x/k_y$) is assumed to be 0.8 for sand tailings (based on the discussions of Klohn (1979); Saad & Miti (2010a, 2010b)) and 0.1 for slimes (based on the discussions of Vick (1990); Abadjiev (1976); Vermeulen (2001); Saad & Miti (2010a, 2010b)).

Based on the test results of L-Bolduc and Aubertin (2014), permeability of waste rock and slimes (based on the discussions and results of Klohn (1979); Abadjiev (1976); Vermeulen (2001)) is assumed to be $1.6 \times 10^{-3}$ m/s, which is in the range of results of other researchers (L-Bolduc & Aubertin 2014).

To obtain the unsaturated permeability parameters, the water retention curve of sand tailings is determined by laboratory tests, and is then curve-fitted using the Equation (5) (Van Genuchten 1980):

$$\theta_w = \theta_o + \frac{\theta_s - \theta_r}{1 + (\psi/\alpha)^m}$$

where $\theta_w$ is the volumetric water content at different suction, $\psi$ is the negative pore water pressure, and $a$ (in kPa), $n$, and $m$ are the curve fitting parameters. The parameter $m$ is assumed to be equal to $1/n$ (Krahn 2012). For the core material, parameters $a$ and $n$ are determined using the equations suggested by Tinjum et al (1997) for compacted clays, and the retention curve of the foundation material is assumed to match the default curve provided in SIGMA/W for clays containing sand and gravel. In addition, the retention curve of slimes and waste rock is estimated using the modified Kovacs model. This model was first introduced by Aubertin et al (1998) for tailings from hard-rock mines, and was later developed for other soils (Aubertin et al 2003). The unsaturated permeability function of all the materials is determined based on their retention curves and on the equation provided by Van Genuchten (1980).

**NUMERICAL ANALYSIS**

**Coupled analysis**

SIGMA/W is a computer program for solving two-dimensional soil consolidation problems by using fully coupled analysis (Krahn 2012). In this analysis, each node of element grid (mesh) is assigned three equations, two of which concern the displacements (horizontal and vertical) while the third expresses the flow. Variations of displacement and pore water pressure can be determined by solving these three equations simultaneously.

**Constitutive equation of the soil structure**

Equation (6) shows the strain-stress development of an unsaturated soil in two-dimensional space (Fredlund & Rahardjo 1993):

$$\begin{bmatrix}
\Delta(\sigma_x - u_w) \\
\Delta(\sigma_y - u_w) \\
\Delta(\sigma_z - u_w)
\end{bmatrix}
= 
\frac{E(1 - v)}{(1 + v)(1 - 2v)}
\begin{bmatrix}
\Delta \varepsilon_x \\
\Delta \varepsilon_y \\
\Delta \varepsilon_z
\end{bmatrix}
$$

where $\varepsilon$ is the normal strain, $\gamma$ is the shear strain, $\sigma$ is the normal stress, $r$ is the shear stress, $u_w$ is the pore air pressure, $u_w$ is the pore water pressure, $E$ is the elastic modulus of the soil structure, $v$ is the Poisson’s ratio, and $H$ is the modulus of unsaturated soil for soil structure in regard to matric suction ($u_a - u_w$), which can be obtained by using the procedure described by Chen et al (2014).

**Flow equation for water phase**

Two-dimensional water flow in saturated-unsaturated soil is solved with Darcy’s law (Fredlund & Rahardjo 1993):

$$ \frac{k_x \partial^2 u_w}{\gamma_w \partial x^2} + \frac{k_y \partial^2 u_w}{\gamma_w \partial y^2} + \frac{\partial u_w}{\partial t} = 8 \beta$$

where $k_x$ and $k_y$ are the permeability on the horizontal and vertical directions, $u_w$ is the seepage velocity, $\theta_w$ is the volumetric water content, $\gamma_w$ is the unit weight of water, and $t$ is time. The volumetric water content of elastic materials is determined by Equation (8) (Darkshanamurthy et al 1984):

$$\theta_w = \frac{\beta}{3} \frac{E_x + \omega u_w}{3}$$

where

$$\beta = \frac{E}{H (1 - 2v)} = \frac{3K_B}{H}$$

$$\omega = \frac{1}{R} - \frac{3\beta}{H}$$

where $K_B$ is the bulk modulus and $R$ is a modulus relating the change in volumetric water content to the change in matric suction, and equals the inverse of the slope of the soil-water characteristic curve (Krahn 2012).

**Staged construction and boundary conditions of the base model**

Figure 3 shows the regions and boundary conditions of the base model in the last stage of construction. In general, staged construction of the dam is modelled in three phases.

1. **In situ analysis:** To determine the initial stresses in the foundation, the volume of each element is multiplied by its corresponding unit weight, and the resultant force is exerted vertically on the nodal points. SIGMA/W determines the horizontal stresses using the coefficient of earth pressure at rest, $k_0 \omega/(1 - v')$ (Krahn 2012). In the next phases, coupled analysis is conducted to determine the pore water pressure developed in the foundation due to
staged construction of the starter dam, impoundment and dykes.

2. **Starter dam:** In view of the modelling approach, investigations and results provided by Naylor et al. (1988), Zomorodian et al. (2006) and Elia et al. (2011), the fully coupled analysis of staged construction of the starter dam is performed by assuming seven 10 m thick layers. Furthermore, the total construction time of this structure is assumed to be two years.

3. **Dykes and impoundment:** Once the starter dam has been modelled, slimes are disposed of into impoundment and sand dykes constructed downstream. The fully coupled analysis of dykes and impoundment is performed by assuming 33 and 32 layers 5 m thick respectively. The total duration of this operation is assumed to be 30 years, with an annual elevation rise of 5.67 m. Concerning the boundary conditions, in the in situ analysis the initial water surface is defined to be at the level of the ground surface in order to produce hydrostatic water pressure in the foundation. In addition to displacement boundaries, for which the method of assignment in all the analyses is illustrated in Figure 3, the coupled analysis also requires a hydraulic boundary condition. For this purpose, when a new layer is added to the core and slimes, the boundary condition of zero pressure \( p = 0 \) is assigned to the upper boundary of this layer. With this approach, variations of water level are modelled with the rise in the elevation of materials, and the surface of the new layer can be drained. With the addition of the weight of each new layer, the resulting increases in the pore water pressure in the previous layers and the loading-induced consolidation are calculated simultaneously using the coupled analysis.

### Meshing

In view of the inclined geometry of the core, mesh is developed in the form of a rectangular grid of quads, which is well suited for four-sided regions according to the software manual (Krahn 2012). Other regions have an unstructured quad and triangle mesh, which has been recommended for various geometries (Krahn 2012). Additionally, triangular and rectangular elements are assigned with 3 and 9 Gauss points respectively to enhance the accuracy.

Based on the results of mesh convergence analyses, dykes, impoundment and foundation are modelled with 5-m elements, and layers of starter dam are modelled with 10-m elements. This mesh consists of 6 246 nodes and 6 064 elements in total.

### Other models

The previous section explained the geometry, material properties and method of analysis for the base model. Using this base model as the basis of work, the hydro-mechanical behaviour of the dam under the conditions described in Table 2 is studied. As this table shows, in Models 1 and 2 the effect of foundation permeability, in Models 3 and 4 the effect of raising rate, in Models 5 to 8 the effect of anisotropic permeability of slimes, in Models 9 and 10 the effect of changes in downstream slope, and in Model 11 the effect of the presence of permeable bedrock on the structural response are investigated. Models 5 and 8 are very unlikely to occur, and they are only analysed because of their conceptual utility. Moreover, apart from the differences listed in this table, all the models use the same material properties, modelling technique and analysis conditions previously explained for the base model.

### Results and Discussion

Based on the discussions of Saad and Mitri (2010a, b), the hydro-mechanical behaviour of the tailings dam during staged construction, in terms of pore water pressure developed in the impoundment and horizontal displacement of the dam body, was evaluated. All outputs were obtained for the base model as well as for the other models in Table 2, and the results of these models are compared. Two cross-sections, A and B (see Figure 4), were selected for detailed study of the pore water pressure in the impoundment and the horizontal displacement in the dam body respectively. The position of these two cross-sections

---

**Figure 4** Position of cross-sections A and B used for presenting the results of the analyses.

---

### Table 2 Models used to study the effects of different factors on the hydro-mechanical behaviour of the high centreline tailings dam

<table>
<thead>
<tr>
<th>Model no</th>
<th>Description</th>
<th>Parameter</th>
<th>Model no</th>
<th>Description</th>
<th>Parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Increasing foundation permeability</td>
<td>( k_h = 1 \times 10^{-4} ) (m/s)</td>
<td>7</td>
<td>Increasing anisotropic permeability of slimes</td>
<td>( k_v/k_h = 0.05 )</td>
</tr>
<tr>
<td>2</td>
<td>Decreasing foundation permeability</td>
<td>( k_h = 1 \times 10^{-7} ) (m/s)</td>
<td>8</td>
<td>Increasing anisotropic permeability of slimes</td>
<td>( k_v/k_h = 1.0 )</td>
</tr>
<tr>
<td>3</td>
<td>Increasing raising rate</td>
<td>25 Years</td>
<td>9</td>
<td>Increasing downstream slope</td>
<td>1H:3V</td>
</tr>
<tr>
<td>4</td>
<td>Decreasing raising rate</td>
<td>35 Years</td>
<td>10</td>
<td>Decreasing downstream slope</td>
<td>1H:5V</td>
</tr>
<tr>
<td>5</td>
<td>Decreasing anisotropic permeability of slimes</td>
<td>( k_v/k_h = 0.01 )</td>
<td>11</td>
<td>Permeable bedrock</td>
<td>Drainage at base</td>
</tr>
<tr>
<td>6</td>
<td>Increasing anisotropic permeability of slimes</td>
<td>( k_v/k_h = 0.2 )</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
allows them to provide useful information on the performance of the impoundment and the dam body.

Figure 5 shows the pore water pressure and the effective confining pressure in cross-section A at the end of construction in Models 1–8 and the base model, and Figure 6 shows the horizontal displacement in cross-section B at the end of construction in all the models (excluding Model 11).

The base model
Figure 7 shows the contour lines of the pore water pressure developed in the impoundment at the end of construction in the base model. The peak PWP has developed in the middle of the impoundment at the approximate range of (0.3–0.5) H. This is due to the permeability value of the foundation and the possibility of drainage at the impoundment surface, which cause the PWP to dissipate more quickly in the upper and lower layers than in the middle layers. This can also be seen in cross-section A in Figure 5(a). The trend of PWP change in this cross-section is similar to that reported by Gassner and Fourie (1998) for the case where the bottom of the tailings was allowed to drain. At heights above 80 m, the pore water pressure in cross-section A exceeds the confining pressure, and the materials have an undrained-like behaviour (see Figure 5). In addition, at all heights, the PWP in cross-section A is less than the hydrostatic pressure (Figure 5a).

Figure 8 shows the contour lines of the horizontal displacement of the dam at the end of construction. The maximum displacement in the dykes (1.8–2.0 m) is observed in the upper section and downstream of the starter dam in the range of (0.35–0.6) H. This range is similar to the results of the analysis conducted by Törnqvist and Lindquist (2016) for a high tailings dam with similar geometry.

Effect of foundation permeability
The results of field tests conducted by drilling boreholes in the foundation indicate that the coefficient of permeability is in the range of $10^{-7}$–$10^{-3}$ m/s. Given that the obtained field data has no specific distribution, the effect of permeability of the foundation on the hydro-mechanical behaviour was investigated by assuming a fixed value for this parameter.

Based on the contour lines of the pore water pressure in the impoundment of Models 1 and 2 (Figure 9), several results can be inferred. First, there is little difference between the results of the base model and Model 1, both of which have a foundation with “low” permeability, and the maximum pore water pressure in Model 1 is about 50 kPa lower than that in the base model. This is while the permeability of the foundation has increased a hundredfold. In Model 2, where the foundation permeability is in the category of “very low”, the peak pore water pressure in the impoundment area has reached more than 1 600 kPa. In addition, in this model the bulb of maximum pore water pressure area has shifted closer to the foundation, because of the much lower potential for drainage of excess pore water through the foundation. This is similar to the results obtained by Saad.
and Mitri (2010a, 2010b) and Ormann et al. (2013) for upstream tailings dams with low permeable foundations, where the bulb of maximum pore water pressure has been reported to be in the range of approximately 20 to 30% of the impoundment floor.

According to Figure 5, there is very little difference between the base model and Model 1 in terms of pore water pressure and effective confining pressure in cross-section A. This result indicates that, as long as the foundation has a “low” permeability, the pore water pressure of the impoundment is not very sensitive to changes in the permeability coefficient of the foundation. However, in Model 2, in addition to pore water pressure being greater than the base model and closer to the hydrostatic pressure, there is a much wider difference between the effective confining pressure and the pore water pressure. In other words, in cross-section A, a large area of the impoundment (from a height of approximately 45 m from the floor) has an undrained-like behaviour. Nevertheless, in the two previous models, this condition is observed at heights above 80 m. The trend of pore water pressure change in this model and at this cross-section is similar to the results reported by Gassner and Fourie (1998) for the scenario where there was no drainage in the model floor.

Comparing the horizontal displacement in cross-section B of Models 1 and 2 with that of the base models, Figure 6 shows little difference between the results of Model 1 and the base model, as both have a maximum horizontal displacement of about 1.96 m; however, horizontal displacement of Model 2 in cross-section B reaches about 2.2 m. Therefore, as the permeability of the foundation decreases to a “very low” category, the PWP developed in the impoundment and the resulting seepage force applied to the body increase, which in turn result in increased horizontal displacement in the dam body. The above results show that more detailed studies are needed on the permeability parameters of the foundation of tailings dams.

Effect of raising rate
Duration of construction is an important factor in the stability of upstream tailings dams and this issue for this method of construction has been discussed in investigations such as those by Sarsby (2013) and Saad (2013), although Vick (1990) has not specified any limit for the raising rate of centreline tailings dams, and has only recommended a certain height limit for each individual elevation raise.

Figure 10 shows the contour lines of the pore water pressure in Models 3 and 4. It can be observed that the maximum pore water pressure (up to 800 kPa) has been developed in Model 3, but that in Model 4 the pore water pressure developed
in the impoundment has decreased with the decrease in the raising rate down to a peak value of 650 kPa. Hence, in general the pore water pressure increases with an increase in the raising rate because faster construction means materials have less time for drainage. Obviously, at the end of construction, the dam built at a faster rate needs a longer time for the pore water pressure to dissipate.

In cross-section A (Figure 5), the pore water pressure of Model 3 is closer to the hydrostatic pressure than it is in Model 4 and the base model, but the PWP value and the effective confining pressure of all three models are almost the same, and at a height of above 80 m materials of all three models exhibit an undrained-like behaviour. In cross-sections closer than A to the dam body, all three models show less difference between PWP and $\sigma'_x$; therefore, although the maximum pore water pressure of the impoundment increases with the raising rate, the overall performance of the impoundment near the dam body does not depend much on this parameter. This causes the horizontal displacement in cross-section B of Models 3 and 4 to be almost identical to that which is observed in the base model (see Figure 6). On the whole, an increase (decrease) in the raising rate increases (decreases) the maximum x-displacement in this cross-section by roughly 10%. This amount of change is not sufficiently large to cause significant changes in the dam performance.

Effect of anisotropic permeability of slimes

It has been reported that seepage pattern is influenced more by the anisotropic permeability of tailings than by the definite value of this coefficient (Sarsby 2013). For this reason, some studies, such as those of Klohn (1979), Vick (1990) and Sarsby (2013), have investigated the effect of this parameter on the position of the phreatic surface under steady seepage conditions. However, the technical literature provides only sparse information regarding the effects of anisotropy in the permeability of slimes on the development of pore water pressure and overall performance of tailings dams during staged construction. Thus Models 5 to 8 are used to evaluate the performance of the dam. The resulting contour lines obtained for the pore water pressure in the impoundment are shown in Figure 11:

(a) Maximum PWP of the impoundment has decreased from 1 400 kPa in Model 5 to 550 kPa in Model 8. This means that as $k_v/k_h$ increases, the pore water pressure developed in the impoundment decreases because dissipation of excess pore water pressure along the vertical direction and flow of water towards the impoundment surface or the foundation become easier.

(b) As $k_v/k_h$ increases, the bulb of maximum pore water pressure shifts towards the foundation, perhaps because the flow of water towards the impoundment surface and the dissipation of excess pore water pressure along this path require less energy than does the flow towards the foundation. Thus water

Figure 9 Contour lines of pore water pressure developed in the impoundment area at the end of construction in (a) Model 1 and (b) Model 2

Figure 10 Contour lines of pore water pressure developed in the impoundment area at the end of construction in (a) Model 3 and (b) Model 4
pressure dissipation in the upper sections of the impoundment takes less time than it does in the lower sections, and the bulb of maximum pore water pressure shifts closer to the foundation.

(c) As $k_v/k_h$ increases, so does the pore water pressure developed in the foundation. In Model 8, the PWP developed in the foundation even exceeds the maximum PWP developed in the impoundment. In this model, as the developed PWP decreases, the effective vertical stress in the impoundment increases and the weight of materials exhibits a greater effect on the foundation.

(d) Comparison between the results of the base model and Model 6 with maximum PWP values of 750 and 550 kPa respectively shows that the slightest change in the anisotropy ratio triggers significant changes in the maximum PWP and its distribution in the impoundment, hence the overall performance of the impoundment is strongly affected by this parameter.

Figure 5 shows a clear increase in the pore water pressure and decrease in the effective confining pressure with the decrease in $k_v/k_h$. Generally, as this coefficient decreases, the gap between the effective confining pressure and the pore water pressure grows, and a larger portion of the impoundment in cross-section A exhibits an undrained-like behaviour. In this situation, the profile of pore water pressure shifts closer to the hydrostatic pressure, and in some parts of Model 5 it even exceeds the hydrostatic pressure. The other notable point is the negligible effect of changing the anisotropy ratio from 0.5 to 1 (in Models 7 and 8) on the results in cross-section A.

Examination of Figure 6 for Models 5–8 shows that in general, as the $k_v/k_h$ of tailings in the impoundment increases, horizontal displacement of the body decreases.

The other notable results are as follows:

a. In Model 5 the greatest horizontal displacement in cross-section B is 2.4 m, but in other models it is within the range of 1.6 to 1.8.

b. Models 7 and 8 have similar displacements. These two models also have the same value of PWP developed in cross-section A at the end of the construction. Therefore in the range of $k_v/k_h = 0.5$ to $k_v/k_h = 1.0$, the effect of anisotropy in the permeability of slimes on the x-displacements of the studied dam can be ignored.

c. Comparison of Model 6 with the base model shows that, although the slightest change in permeability leads to significant changes in the pore water pressure, its effect on the horizontal displacement in the dam body is somewhat negligible.

d. According to Figure 12, in Model 5 displacements in the impoundment are greater than those in the body and reach 2.8 m. In this model, given that the movement of excess pore water pressure is mostly horizontal, slimes are subjected to great horizontal seepage force, which leads to large horizontal displacements in the impoundment.

Effect of changing the downstream slope of the dykes

Based on the studies of Rico et al (2008) and Bowker and Chambers (2015), the downstream slope of the dykes is one of the most important factors controlling the stability of tailings dams, and slope instability has led to dam failure on many occasions.
Here Models 9 and 10 are used to evaluate the effect of changes in the downstream slope of the dykes on the horizontal displacement of the dam body. Figure 13 shows the contour lines of the horizontal displacement of the body at the end of construction in these two models. In Model 9, the maximum horizontal displacement in the body is 2.6 m, but in Model 10 this value has decreased by about 80 cm. Therefore, in general, decreasing the slope of the dam body triggers a substantial decrease in the horizontal displacements.

In Figure 6, a significant effect of a change in the downstream slope of the dam on the horizontal displacement can be observed. In cross-section B, the minimum and maximum displacements are observed in Models 9 and 10 respectively, and the results of other analysed models are all within the range bounded by the profiles of these two models. Comparison between the results of Models 9 and 10 with those of the base model shows that an increase in the slope changes the displacement more noticeably than does a decrease.

**Effect of permeable bedrock**

Some borehole samples obtained from the bedrock show extensive weathering down to depths of 10–25 m. Therefore in Model 11, the base of the foundation is assumed to have a drained boundary condition and the resulting effect on the pore water pressure of the impoundment is studied (Figure 14a). The maximum PWP in the impoundment of Model 11 is 50 kPa less than that in the base model, so there is actually little difference between the pore water pressures developed in these two models. In this model, with the construction of primary layers of the impoundment, the PWP is drained towards the bedrock. However, since the rate of elevation raise of the impoundment is faster than the time needed for complete drainage, the movement of water towards the bedrock occurs at a slower rate, and the PWP becomes almost similar to that of the base model. Consequently, the seepage force exerted on the dam body is almost the same as that in the base model, and both models have almost similar horizontal displacements in cross-section B (see Figure 14b).

Thus it can be seen that the great height of the impoundment in comparison to the foundation, assuming a drained boundary condition at the lower boundary of the model, has little effect on the distribution and value of pore water pressure and horizontal displacement.

**CONCLUSIONS**

The results of this study can be summarised as follows:

1. Given the permeability value of the foundation in the base model, maximum pore water pressure was observed in the middle of the impoundment in the approximate range of (0.2–0.5) H. Meanwhile, peak horizontal displacement in the dykes was observed in the range of (0.35–0.6) H.

2. A decrease in permeability of the foundation has a significant effect on the distribution of PWP in the impoundment, an effect that is reflected in the increased value of the bulb of maximum pore water pressure and a shift in its position toward the bottom layers of the impoundment. This leads to an increase in the seepage force exerted on the dam body and triggers more horizontal displacements.

3. Although an increased raising rate increases the pore water pressure developed in the impoundment, its effect on horizontal displacement in the body of the analysed tailings dam is negligible.

4. Anisotropy in the permeability of slimes has a significant effect on the pore water pressure developed in the dam impoundment, as it was observed that an increase in the coefficient \( k_v/k_h \) decreased the maximum PWP in the impoundment. It is important to note that even a slight change in the anisotropy ratio leads to considerable changes in the distribution of the PWP and its maximum value. Therefore it is vitally important to pay due attention to this parameter, especially in dams in which pore water pressure has a significant effect on the structural stability (e.g. upstream tailings dams). However, from the perspective of displacement in the analysed tailings dam, when \( k_v/k_h \) is in the range of 0.5–1, this coefficient...
shows an insignificant effect on the horizontal displacement of the dam body and can therefore be disregarded.

5. Changing the downstream slope of the dam wall causes significant changes in its horizontal displacements. In fact, among the analysed parameters, this parameter has the greatest effect on the horizontal displacement. This result shows the importance of paying due attention to the outer slope in addition to other factors related to dam stability in staged construction.

6. Given the great height of the dam, the permeable bedrock has little effect on the horizontal displacement and pore water pressure of the dam.

7. It can be generally stated that, as the pore water pressure in the impoundment increases, so do the seepage forces and the hydraulic gradient exerted horizontally on the dam wall. This leads to greater horizontal displacements in the dam. This relation is validated by comparing Models 2, 3 and 5 with the base model.

8. From a qualitative perspective, the overall performance of the analysed tailings dam under different conditions is in good agreement with the results obtained by other researchers. This is particularly true for the pore water pressure developed in the dam impoundment. Therefore factors affecting the structural response of this dam are applicable to other tailings dams.

**ACKNOWLEDGEMENT**

The authors would like to thank Toos Ab Consulting Engineers Company for providing valuable information and data.

**REFERENCES**


The influence of health and safety practices on health and safety performance outcomes in small and medium enterprise projects in the South African construction industry

J N Agumba, T C Haupt

Health and safety (H&S) management is imperative for construction projects owing to the high level of fatalities and accident rates experienced. However, very few studies have used mixed-method research to examine H&S practices that are tailored towards small and medium construction enterprises (SMEs) to establish whether they influenced overall H&S performance. A Delphi study involving 16 H&S experts produced a refined H&S conceptual model comprising five H&S practices and one H&S performance outcome which were tested on 216 construction SMEs. Exploratory factor analysis and confirmatory factor analysis confirmed that the H&S practices and performance were valid, reliable and acceptable variables. Structural equation modelling (SEM) produced a good-fit model. Upper management commitment and involvement in H&S influenced overall H&S performance indirectly through the mediating variables of project supervision and H&S resources and training. These three H&S practices are essential in influencing H&S performance at project level of construction SMEs, and are viewed as catalysts for H&S culture. However, to ensure that H&S performance by construction SMEs is improved, upper management personnel should be committed and involved in H&S at project level.

INTRODUCTION
The H&S of workers is part and parcel of human security (International Labour Organization (ILO 2003)). The South African Occupational Health and Safety Act 181 of 1993 states that every worker has a right to a healthy and safe working environment.

Federated Employer Mutual Assurance (2017) reported that by June 2017, 24 fatal accidents had occurred in the South African construction industry, including construction SMEs. Although the average fatality rate had decreased in the UK, the Health and Safety Executive (HSE 2016) reported that in 2015 there were 43 fatalities in construction. In Hong Kong, construction recorded the highest number of work-related fatalities and accident rates (Labour Department 2016). In the US, 904 incidents were recorded in construction, which included construction transportation; fires and explosion; falls, trips and slips; workers exposed to harmful substances or environment and workers coming into contact with objects and equipment (Bureau of Labour Statistics (BLS 2016)). The construction industry is still plagued by poor H&S performance. However, these work-related fatalities, injuries and incidents are preventable.

These statistics suggest that the construction industry is dangerous and hazardous, and this damages its reputation. These fatality, injury and incident rates have a negative cost effect on the economy as employees could be forced to take early retirement due to permanent disability, which makes them an economic burden due to their unemployability and pension payments. Furthermore, medical expenses incurred is likely to put a strain on the economy.

The poor H&S performance in the construction industry in South Africa is exacerbated by limited commitment of upper management personnel to H&S issues. As a result, the productivity and performance of the construction projects is negatively affected. These facts have raised the need to investigate if there are any variables that can be used to predict H&S performance.

As a result, exploratory factor analysis and structural equation modelling (SEM) techniques were used to test the influence of health and safety practices on health and safety performance outcomes in SMEs. The study was based on an exploratory factor analysis of H&S practices and a confirmatory factor analysis of a H&S performance outcome. Then, SEM was used to produce a model of variables that influence health and safety performance outcomes in South African construction SMEs.
SMEs to comply with basic construction requirements and their casual attitude towards H&S. SMEs do not properly maintain their tools and equipment, their personnel do not use their personal protective equipment, and they view H&S interventions as a luxury (cildb 2004). This is not unique to South Africa. Unnikrishnan et al (2014) found that in India safety management practices are inadequately implemented in the work environment of SMEs. According to the cildb (2009), SMEs lack formal occupational health and safety management systems (OHSMSs). According to Benjaoran and Bhokha (2010), most construction projects do not establish such a system on site. SMEs lack H&S training and competence (cildb 2009), and these shortcomings could jeopardise the improvement of H&S at the project level of construction SMEs.

The poor H&S performance of the construction industry in South Africa provided the impetus for promulgating the Construction Regulations of 2003 (Smallwood & Haupt 2005) which were revised in 2014. For compliance with the regulations, Azimah et al (2009) argue that H&S management should be addressed. Unnikrishnan et al (2014) emphasise that H&S management practices should be improved to comply with H&S standards, which will result in better productivity.

The underperformance by construction SMEs is caused by their financial constraints and their lack of managerial and technical skills (Department of Public Works (DPW), 1999; Agumba et al 2005; Martin 2010). This hinders the implementation of H&S practices, leading to poor H&S culture. It is imperative to identify H&S practices that are suitable, viable and manageable for construction SMEs.

H&S performance measures are classified as lagging and leading indicators (Toellner 2001). Leading indicators can either be subjective or objective (Grabowski et al 2007). On the other hand, lagging indicators are measured by the number of accidents and workers’ compensation statistics (Mohamed 2002). However, they can still be measured subjectively (Chinda & Mohamed 2008). The use of leading indicators instead of lagging indicators is increasingly advocated (Hinze 2005) to measure H&S performance. Flin et al (2000) state that H&S climate measurement is a leading indicator which measures the H&S culture of an organisation. However, for an optimum H&S culture to manifest, H&S practices should be implemented and practised effectively.

Previous studies have not reached consensus regarding the H&S practices that could improve the H&S performance of construction SMEs and also evaluate their H&S culture (Mearns et al 2003; Teo & Ling 2006; Fernández-Muñiz et al 2007; Chinda & Mohamed 2008; Molenaar et al 2009; Cheng et al 2012). According to Grabowski et al (2010) and Hinze et al (2013), this is a leading H&S indicator.

The present study conceptualised a model linking H&S practices and overall H&S performance. The theoretical model was used to determine the relationship between H&S practices and H&S performance. Two specific research questions were addressed:

- Which H&S practices are valid and reliable for managing H&S in construction SMEs?
- What is the influence of the H&S practices on the H&S performance on construction SME projects?

**MODEL CONSTRUCTS AND HYPOTHESES**

The conceptual model shown in Figure 1 suggests that H&S performance is directly determined by one independent factor, namely upper management commitment and involvement in H&S. This factor indirectly determines H&S performance via the intermediary factors of employee involvement and empowerment in H&S and an occupational health and safety management system (OHSMS) defined by eight practices.

**Upper management commitment and involvement in H&S**

The importance of management commitment and involvement in H&S is fundamental to an organisation’s H&S culture (O‘Toole 2002; Arboleda et al 2003; Choudhry et al 2008; Khair et al 2012). Several studies on H&S performance improvement (Mohammed 2002; Abudayyeh et al 2006; Fernández-Muñiz et al 2007; Aksorn & Hadisuwarno 2008; Teo et al 2008; Azimah et al 2009) established that management commitment and involvement in H&S was critical in terms of H&S management systems. It influences H&S performance by reducing accidents, disease, worker absenteeism and injuries. Fernández-Muñiz et al (2007) established that management commitment positively influences employee involvement in H&S.

Langford et al (2000) found that when management practices H&S, employees are willing to participate in H&S. The hypotheses proposed were:

H1. Upper management commitment and involvement in H&S positively influence employee involvement and empowerment in H&S

H2. Upper management commitment and involvement in H&S positively influence H&S outcomes

Fernández-Muñiz et al (2007) established that management commitment positively influences the safety management system, which comprises policy, incentives, preventative and emergency planning, control, training and communication. Surienty et al (2010) established that management commitment in H&S is positively associated with occupational H&S management implementation. Chinda and Mohamed (2008) found that leadership influenced H&S culture. When upper management is committed and involved in the organisation’s H&S, OHSMS implementation will be successful.

The hypotheses proposed were:

H3a Upper management commitment and involvement in H&S positively influence appointment of H&S staff

H3b Upper management commitment and involvement in H&S positively influence effective formal and informal written communication

H3c Upper management commitment and involvement in H&S positively influence effective formal and informal verbal communication

H3d Upper management commitment and involvement in H&S positively influence effective H&S resources

H3e Upper management commitment and involvement in H&S positively influence effective H&S project planning

H3f Upper management commitment and involvement in H&S positively influence effective project supervision

H3g Upper management commitment and involvement in H&S positively influence effective H&S training

H3h Upper management commitment and involvement in H&S influence effective H&S policy

**Occupational health and safety management system**

The potential impact of H&S practices such as management commitment and involvement and workforce involvement and empowerment in H&S are fundamental...
drivers of H&S performance improvement. The involvement of management and the workforce needs to be supported by a comprehensive OHSMS to bring about the improvement of H&S performance. The literature review identified the following essential H&S practices defining OHSMS:

**Appointing or hiring of H&S staff**
According to Vredenburgh (2002), the promotion of H&S culture can be influenced when workers are predisposed to display H&S-conscious attitudes in their work. Grabowski et al. (2010), Hinze (2005) and Sawacha et al. (1999) indicate that hiring H&S staff will improve H&S performance. The Construction Regulations 2014 advocate that H&S personnel be hired. Grabowski et al. (2010) state that hiring quality personnel in H&S positions influences H&S performance. The hypotheses proposed were:

- **H4a** Appointment of H&S staff positively influences employee involvement and empowerment in H&S
- **H5a** Appointment of H&S staff positively influences H&S performance outcome.

**Formal and informal written communication**
Cooper (1998) indicates the importance of communication in influencing H&S performance in the form of formal and informal written communication, which is the transfer of information to employees about possible risks in the workplace and the correct way to combat them. The Health and Safety Executive (HSE 2008) highlights the need for written information regarding H&S procedures and the correct way to perform tasks, which would reinforce what has been verbally communicated. Azimah et al. (2009) indicate that consistent communication of H&S legislation and regulations is vital to achieve good H&S performance. Dingsdag et al. (2008b) argue that workers’ perceptions of H&S culture are reinforced by good H&S communication. Fernández-Muñiz et al. (2007) found that communication as part of the OHSMS influences H&S performance and employee involvement in H&S. Grabowski et al. (2010) also established that communication influences H&S performance. The hypotheses proposed were:

- **H4b** Formal and informal written communication positively influences employee involvement and empowerment
- **H5b** Formal and informal written communication positively influences H&S performance outcome.

**Formal and informal verbal communication**
The Health and Safety Executive (2008) highlights the need for H&S information to be verbally communicated to workers before changes are made to the way their work activities are executed. Furthermore, Fernández-Muñiz et al. (2007) and Kheni et al. (2006) established the importance of verbal communication in improving H&S performance. Fernández-Muñiz et al. (2007) found that communication as part of the OHSMS influences H&S performance and employee involvement in H&S. The hypotheses proposed were:

- **H4c** Effective formal and informal verbal communication positively influences employee involvement and empowerment in H&S
- **H5c** Effective formal and informal verbal communication positively influences H&S performance outcome.

**Health and safety resources**
Health and safety resources enable H&S performance of the project to be achieved (Abudayeh et al. 2006; Rajendran & Gambatese 2009). Choudhry et al. (2007) established that the availability of resources was a good predictor of H&S performance. Fernández-Muñiz et al. (2007) indicated that H&S resources influence employee involvement and H&S performance. Rajendran and Gambatese (2009) showed that the availability of H&S resources influences the reduction of injuries of workers in construction projects. Two hypotheses were proposed:

- **H4d** H&S resources positively influence employee involvement and empowerment in H&S
- **H5d** H&S resources positively influence H&S performance outcomes

**Project planning of H&S**
Project planning of H&S involves evaluating risks and establishing necessary H&S measures to avoid accidents, which include planning for emergencies (Fernández-Muñiz et al. 2007). Arocena and Nuñez (2010) indicate that H&S planning is an element of OHSMS for SMEs and, when implemented, reduces accidents. Fernández-Muñiz et al. (2007) found that planning influences employee involvement and H&S performance. The hypotheses proposed were:

- **H4e** Effective project planning of H&S positively influences employee involvement and empowerment in H&S
- **H5e** Effective project planning of H&S positively influences H&S performance outcomes.

**Project supervision**
Project supervision verifies the extent to which goals have been met, as well as compliance with internal norms or work procedures (Fernández-Muñiz et al. 2007). Teo et al. (2008) and Fang et al. (2004) indicate that supervision influences H&S performance. Fernández-Muñiz et al. (2007) found that control influences employee involvement and H&S performance. The hypotheses proposed were:

- **H4f** Effective project supervision positively influences employee involvement and empowerment in H&S
- **H5f** Effective project supervision positively influences H&S performance outcomes.

**Training in health and safety**
Langford et al. (2002) found that training of operatives and H&S supervisors ensures H&S awareness and improved performance. Zeng et al. (2008) point out that some accidents such as falling from a height and being hit by falling materials on construction sites can easily be prevented by implementing training programmes for employees. The influence of H&S training to improve H&S performance has been addressed by many researchers (Sawacha et al. 1999; Tam et al. 2004; Kheni et al. 2006; Choudhry et al. 2007; Fernández-Muñiz et al. 2007; Aksorn & Hadisukumo 2008; Dingsdag et al. 2008b). Fernández-Muñiz et al. (2007) also established that training in H&S influences employee involvement. The hypotheses proposed were:

- **H4g** Effective training in H&S positively influences employee involvement and empowerment in H&S
- **H5g** Effective training in H&S positively influences H&S performance outcomes.

**Health and safety policy**
Health and safety policy are the rules and procedures that employees’ management should adhere to in the workplace and are the bedrock of the OHSMS. According to Cox and Cheyne (2000), the major factor that influences H&S is the extent to which workers perceive the H&S rules and procedures as being implemented and promoted in the organisation. Ng et al. (2005) and Fernández-Muñiz et al. (2007) established the importance of H&S policy for improving
Building trust and cooperation, and it is required decisions in the organisation and held accountable for making the abilities. Azimah et al. (2008) and Chinda and Mohamed (2008), who used subjective measures.

**RESEARCH METHODOLOGY**

A mixed-method research paradigm was adopted for this study, which entailed an exploratory design. The Delphi method was used to attain consensus on H&S measures that are ideal for improving H&S performance in construction SME projects. This method enabled the development of a refined questionnaire survey which was distributed to SMEs. The mixed method ensured rigour of the methodology.

The Delphi questionnaire consisted of 64 H&S measures, categorised in ten H&S practices. Four rounds of Delphi survey were used to reach consensus on the H&S practices proposed for H&S performance improvement. Twenty H&S experts were purposively sampled, of whom 16 were involved in all the Delphi iterations. The experts rated the H&S measures according to their importance and effect on improving H&S performance at the project level of SMEs. The retained H&S measures had to attain a median value of between 9.00 and 10.00 for importance. Their impact value had to be between 90% and 100%. The two scales were used to ensure stringency in retaining a measure and to avoid bias by using only one measure to determine consensus. Thirty-one H&S measures were retained which were considered to improve H&S at the project level of SMEs. The retained H&S measures comprised the final questionnaire presented to construction SMEs in South Africa. The 31 H&S measures defined five H&S practices. H&S performance was defined by nine H&S measures. The SME respondents were required to indicate their level of agreement with the use of the H&S measures in their projects, rated on a five-point Likert scale, where 1 = strongly disagree, 2 = disagree, 3 = neutral, 4 = agree, and 5 = strongly agree.

Other parts of the questionnaire were designed to profile the participants in terms of their position in the organisation, gender, race, experience in the construction industry and qualifications. The questionnaire also profiled the organisation according to the type of business and geographic location. The questionnaire was piloted with eight personnel members from eight construction SMEs who were...
knowledgeable about H&S measures practised at the project level. The final version of the questionnaire was presented to 1 450 conveniently sampled SMEs. The data was collected using e-mail, and a drop-and-collect method was adopted, which resulted in 228 questionnaires being returned. This represented a 15.7% response rate, which concurs with the findings of Kongtip et al (2008). Only 216 questionnaires were deemed valid for analysis. According to Kline (2005), over 200 usable questionnaires for statistical analysis using structural equation modelling are sufficient. Similarly, Pallant (2013) indicates that over 200 usable questionnaires are adequate to undertake robust statistical analysis. The software used was the Statistical Package for Social Science (SPSS) version 20 for the descriptive statistical analysis of the data. The frequencies, mean scores and standard deviation, and the factor analysability of the H&S practices and performance were computed. Similarly, exploratory factor analysis (EFA) was used to determine the unidimensionality and reliability of the H&S practices and performance. Reliability was tested using Cronbach’s alpha with a cut-off value of 0.70 as recommended by Hair et al (2006). Maximum Likelihood with Promax Rotation was selected as the extraction and rotation methods in the EFA. Confirmatory factor analysis (CFA) was used to determine the acceptability of the H&S practices and performance. The Tucker Lewis Index (TLI), which should be greater than 0.90; the root mean square error of approximation (RMSEA) and standardised root mean squared residuals (SRMR) less than 0.08; p-value less than 0.05; and normed chi-squared (χ²/df) less than 5 were applied. The structural equation model (SEM) using Mplus version 6.1 was used to test the influence of the H&S practices and performance. The data of the model was found to be acceptable after the fit indices of the model had been checked.

RESULTS AND DISCUSSION

Demographic descriptive statistics

The results indicated that 29.0% of the respondents were business owners, 67.0% had titles such as H&S representatives and site agents, and 86.0% were male. Africans comprised 61.0% of the respondents, 30.0% were white and 6.0% were Asian/Indian and coloureds. The majority of the

<table>
<thead>
<tr>
<th>Table 1 Result of Delphi round 4</th>
<th>Importance</th>
<th>Median impact (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H&amp;S practices and measures</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Appointment of H&amp;S staff</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Employing at least one qualified manager with H&amp;S training on multiple projects</td>
<td>8.00</td>
<td>75.00</td>
</tr>
<tr>
<td>At least one staff member with H&amp;S training is employed on each project</td>
<td>8.00</td>
<td>80.00</td>
</tr>
<tr>
<td>Employing at least one H&amp;S representative on each project</td>
<td>7.50</td>
<td>70.00</td>
</tr>
<tr>
<td>Formal and informal written communication</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Provision of written information about H&amp;S procedures</td>
<td>8.50</td>
<td>85.00</td>
</tr>
<tr>
<td>Provision of written information about the correct way to perform tasks</td>
<td>8.00</td>
<td>80.00</td>
</tr>
<tr>
<td>Written circular/brochure that informs workers about the risks associated with their work</td>
<td>7.00</td>
<td>80.00</td>
</tr>
<tr>
<td>Written circular/brochure that informs workers about preventive measures to reduce risk</td>
<td>7.00</td>
<td>80.00</td>
</tr>
<tr>
<td>Formal and informal verbal communication</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Provide clear verbal instructions to both literate and non-literate employees about H&amp;S</td>
<td>9.50</td>
<td>80.00</td>
</tr>
<tr>
<td>H&amp;S information verbally communicated to workers before changes are made to the way their work activities are executed</td>
<td>9.00</td>
<td>80.00</td>
</tr>
<tr>
<td>Organise regular meetings to verbally inform workers about the risks associated with their work</td>
<td>9.00</td>
<td>85.00</td>
</tr>
<tr>
<td>Organise regular meetings to verbally inform workers about preventive H&amp;S measures of risky work</td>
<td>9.00</td>
<td>90.00</td>
</tr>
<tr>
<td>H&amp;S resources</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Provision of personal protective equipment (PPE)</td>
<td>9.00</td>
<td>80.00</td>
</tr>
<tr>
<td>Training in H&amp;S by attending seminars/workshops</td>
<td>8.50</td>
<td>80.00</td>
</tr>
<tr>
<td>Material safety data sheets provided for all hazardous materials on site</td>
<td>9.00</td>
<td>80.00</td>
</tr>
<tr>
<td>Employing technically skilled employees with H&amp;S training</td>
<td>9.00</td>
<td>80.00</td>
</tr>
<tr>
<td>Adequate information brochures on H&amp;S</td>
<td>8.00</td>
<td>80.00</td>
</tr>
<tr>
<td>Provision of a budget for H&amp;S</td>
<td>10.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Provision of correct tools, equipment and plant to execute construction</td>
<td>9.50</td>
<td>90.00</td>
</tr>
<tr>
<td>Provision of good welfare facilities such as showers, canteens, toilets</td>
<td>10.00</td>
<td>85.00</td>
</tr>
<tr>
<td>Project planning of H&amp;S</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ergonomics are considered when deciding the method of construction</td>
<td>9.00</td>
<td>80.00</td>
</tr>
<tr>
<td>Re-engineering is considered to reduce hazards</td>
<td>9.00</td>
<td>85.00</td>
</tr>
<tr>
<td>When head office decides on the method of construction H&amp;S is included in the decision-making process</td>
<td>9.00</td>
<td>85.00</td>
</tr>
<tr>
<td>Each project has a site-specific H&amp;S plan</td>
<td>9.00</td>
<td>85.00</td>
</tr>
<tr>
<td>Layout of the site considers H&amp;S aspects</td>
<td>9.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Use of hazard-identification procedures</td>
<td>9.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Use of risk-assessment procedures</td>
<td>9.00</td>
<td>85.00</td>
</tr>
<tr>
<td>Constructability of project is reviewed</td>
<td>9.00</td>
<td>80.00</td>
</tr>
<tr>
<td>Scheduling for H&amp;S</td>
<td>9.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Project supervision</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Proper supervision by staff trained in H&amp;S</td>
<td>9.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Identification of hazards by at least one staff member trained in H&amp;S</td>
<td>9.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Results of inspection discussed at H&amp;S meeting</td>
<td>9.00</td>
<td>85.00</td>
</tr>
<tr>
<td>H&amp;S inspections done at least daily</td>
<td>9.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Local authorities and H&amp;S enforcement agencies visit sites for inspection</td>
<td>9.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Ad hoc informal H&amp;S inspections of work-place</td>
<td>9.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Regular H&amp;S audits of projects</td>
<td>9.00</td>
<td>90.00</td>
</tr>
</tbody>
</table>

Legend: Measures in black reached consensus and were retained, whereas the red measures did not reach consensus and were not retained.

Continued on page 66
Table 1 continued … Results of Delphi round 4

<table>
<thead>
<tr>
<th>H&amp;S practices and measures</th>
<th>Importance median</th>
<th>Median impact (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Training in H&amp;S</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Workers undergo induction on H&amp;S before commencing work on a particular site</td>
<td>9.50</td>
<td>90.00</td>
</tr>
<tr>
<td>Workers trained in proper care of PPEs</td>
<td>9.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Workers trained in proper use of PPEs</td>
<td>9.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Workers are regularly trained in H&amp;S</td>
<td>9.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Instruction manuals or safe work procedures are used to aid in preventive action</td>
<td>9.00</td>
<td>80.00</td>
</tr>
<tr>
<td><strong>Employer helps employees to train in house (study leave, grants)</strong></td>
<td>8.00</td>
<td>80.00</td>
</tr>
<tr>
<td>Workers are given time off for training</td>
<td>8.00</td>
<td>80.00</td>
</tr>
<tr>
<td><strong>Upper management commitment and involvement in H&amp;S</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Managers encourage and support worker participation, commitment and involvement in H&amp;S activities</td>
<td>10.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Managers encourage and support training of employees in H&amp;S</td>
<td>9.50</td>
<td>90.00</td>
</tr>
<tr>
<td>Managers communicate regularly with workers about H&amp;S</td>
<td>10.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Managers actively monitor the H&amp;S performance of their projects and workers</td>
<td>10.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Managers take responsibility for H&amp;S</td>
<td>10.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Managers actively and visibly lead in H&amp;S matters</td>
<td>10.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Managers regularly visit workplaces to check work conditions or communicate with workers about H&amp;S</td>
<td>10.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Managers encourage and arrange meetings with employees and other managers to discuss H&amp;S matters</td>
<td>10.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Managers conduct toolbox talks themselves</td>
<td>9.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Managers ensure that the H&amp;S budget is adequate</td>
<td>9.50</td>
<td>90.00</td>
</tr>
<tr>
<td>Managers recognise and reward outstanding H&amp;S performance of workers</td>
<td>9.00</td>
<td>90.00</td>
</tr>
<tr>
<td><strong>H&amp;S policy</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Proper implementation of safety management system</td>
<td>9.00</td>
<td>85.00</td>
</tr>
<tr>
<td>Company has H&amp;S policy</td>
<td>8.00</td>
<td>80.00</td>
</tr>
<tr>
<td>Written in-house H&amp;S rules and regulations for all workers reflecting management concern for safety, principles of action and objectives of achievement</td>
<td>8.00</td>
<td>80.00</td>
</tr>
<tr>
<td>The firm coordinates its H&amp;S policies with other human resource policies to ensure the wellbeing of workers</td>
<td>8.50</td>
<td>90.00</td>
</tr>
<tr>
<td><strong>Worker/employee involvement and empowerment in H&amp;S</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Workers are involved in production of H&amp;S policy</td>
<td>9.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Workers provide written suggestions on H&amp;S</td>
<td>8.50</td>
<td>85.00</td>
</tr>
<tr>
<td>Workers are kept informed of provisions of H&amp;S plan</td>
<td>9.00</td>
<td>85.00</td>
</tr>
<tr>
<td>Workers are involved in H&amp;S inspections</td>
<td>9.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Workers are consulted when H&amp;S plan is compiled</td>
<td>9.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Workers are involved in development of H&amp;S rules and safe work procedures</td>
<td>9.00</td>
<td>90.00</td>
</tr>
<tr>
<td>Workers have the explicit right to refuse to work in potentially unsafe unhealthy conditions</td>
<td>9.50</td>
<td>90.00</td>
</tr>
<tr>
<td><strong>Legend:</strong> Measures in black reached consensus and were retained, whereas the red measures did not reach consensus and were not retained</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

37% of the SMEs were operating as subcontractors, 35.6% as general contractors, and 4.2% were civil contractors, suggesting that SMEs are involved in various kinds of construction activities. The majority (90.7%) of the respondents’ conducted business in Gauteng Province.

Delphi results and discussion

Table 1 indicates that 31 H&S measures were retained from the 64 H&S measures identified in the literature. The H&S measures that were not retained did not attain the recommended median value of 9.00, which was considered to be very important, and a 90% cut-off value was considered to have a major impact. These 31 H&S measures were then categorised into five H&S practices. These practices represented the refined conceptual H&S model after Round 4 of the Delphi survey. This is illustrated in Figure 2.

Three of the original H&S practices conceptualised in Figure 1 were retained with the measures hypothesised to define them, namely upper management commitment and involvement in H&S with 11 measures, worker/employee involvement, and empowerment in H&S with five measures, and project supervision with six measures. However, two H&S practices were renamed. The first one was project H&S planning and communication with four measures. This combination concurred with the National Examination Board in Occupational Safety and Health (NEBOSH). The combination of these H&S measures to re-name the H&S practices was aimed at achieving the recommended number of measures, namely three, per construct. This was to enable robust statistical analysis to be done using SPSS (Pallant 2013) and SEM (Kline 2005).

The second adopted name was H&S resource and training with five measures. This combined practice is in line with the findings of Choudhry et al. (2007). The combination was acceptable because two measures were retained in the H&S resource practice, and three were retained in the H&S training practice. The refined H&S conceptual model was then tested statistically.

Exploratory factor analysis

Prior to the assessment of the structural model, exploratory factor analysis (EFA) and confirmatory factor analysis (CFA) were carried out. The EFA determined the validity and reliability of the five H&S practices and the H&S performance outcome. Two measurement properties were used, namely convergent validity and internal reliability. The results are tabulated in Table 2.

The five independent variables of upper management commitment and involvement in H&S, employee involvement and empowerment in H&S, project H&S

respondents, 80.0%, indicated that they had over six years of experience in the construction industry. 28.2% of the respondents had a matric qualification, 12.0% had basic schooling with some having no qualifications, and the remaining 58.3% had post-secondary school qualifications.
planning and communication, project supervision, and H&S resources and training were reliable. Their individual Cronbach’s alpha coefficients were > 0.70, indicating acceptable internal reliability as recommended by Hair et al (2006). The Kaiser-Meyer-Olkin (KMO) of each variable was greater than the recommended value of 0.60 and the Bartlett’s Test of Sphericity was $p < 0.000$. The results are in line with the suggested cut-off values according to Pallant (2013), which suggest that factor analysis could be conducted with the data.

Factor analysis revealed that the measures of upper management commitment and involvement in H&S, employee involvement and empowerment in H&S, project H&S planning and communication, project supervision, and H&S resources and training loaded together on the respective variables. The factor loadings for all the measures were greater than the recommended value of 0.40 as suggested by Field (2005) and Hair et al (2006), therefore achieving convergent validity. The eigenvalue for each of the constructs suggested that they were unidimensional. It can be stated that sufficient evidence of validity and reliability was provided for these H&S practices which enabled CFA to be undertaken. The results are supported by the findings from previous studies by Findley et al (2004), Fernández-Muñiz et al (2007), Choudhry et al (2007) and Agumba and Haupt (2008).

The dependent variable of H&S performance was reliable. The Cronbach’s alpha was greater than 0.70 at 0.907, indicating acceptable internal reliability as suggested by Hair et al (2006). A KMO of 0.905 with Bartlett’s Test of Sphericity of $p < 0.000$ was obtained. These were in line with the recommendation of Pallant (2013). These results suggested that factor analysis could be conducted with the data. All nine measures of H&S performance outcome converged together on this factor. The factor loadings were greater than 0.668 as shown in Table 2, which were greater than the recommended value of 0.40 as suggested by Field (2005) and Hair et al (2006). The eigenvalue of 5.267 explained 58.521% of the variance in the data. Therefore sufficient evidence of validity was achieved. The results indicate that the factor was unidimensional.

### Table 2 H&S practices and H&S performance outcome

<table>
<thead>
<tr>
<th>H&amp;S measures</th>
<th>Cronbach level after deletion</th>
<th>Factor loading</th>
<th>Rank</th>
</tr>
</thead>
<tbody>
<tr>
<td>I/we communicate regularly with workers about H&amp;S</td>
<td>0.847</td>
<td>0.786</td>
<td>1</td>
</tr>
<tr>
<td>I/we actively monitor the H&amp;S performance of the projects and workers</td>
<td>0.844</td>
<td>0.778</td>
<td>2</td>
</tr>
<tr>
<td>I/we encourage discussions on H&amp;S with employees</td>
<td>0.849</td>
<td>0.728</td>
<td>3</td>
</tr>
<tr>
<td>I/we regularly visit workplaces to check work conditions or communicate with workers about H&amp;S</td>
<td>0.850</td>
<td>0.717</td>
<td>4</td>
</tr>
<tr>
<td>I/we actively and visibly lead in H&amp;S matters by e.g. walk-through of the site</td>
<td>0.855</td>
<td>0.672</td>
<td>5</td>
</tr>
<tr>
<td>I/we take responsibility for H&amp;S, e.g. stopping dangerous work on site, etc</td>
<td>0.854</td>
<td>0.667</td>
<td>6</td>
</tr>
<tr>
<td>I/we ensure that the H&amp;S equipment is bought, e.g. hardhats, overalls, etc</td>
<td>0.857</td>
<td>0.618</td>
<td>7</td>
</tr>
<tr>
<td>I/we regularly conduct toolbox talks with the workers</td>
<td>0.857</td>
<td>0.604</td>
<td>8</td>
</tr>
<tr>
<td>I/we provide workers with H&amp;S training when there is less work in the project</td>
<td>0.865</td>
<td>0.491</td>
<td>9</td>
</tr>
<tr>
<td>I/we reward workers who make an extra effort to do work in a safe manner</td>
<td>0.873</td>
<td>0.465</td>
<td>10</td>
</tr>
<tr>
<td>I/we encourage and support worker participation, commitment and involvement in H&amp;S activities</td>
<td>0.867</td>
<td>0.452</td>
<td>11</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>H&amp;S measures</th>
<th>Cronbach level after deletion</th>
<th>Factor loading</th>
<th>Rank</th>
</tr>
</thead>
<tbody>
<tr>
<td>Our workers are involved in the production of H&amp;S policy</td>
<td>0.778</td>
<td>0.863</td>
<td>1</td>
</tr>
<tr>
<td>Our workers help in developing H&amp;S rules and safe work procedures</td>
<td>0.776</td>
<td>0.839</td>
<td>2</td>
</tr>
<tr>
<td>Our workers are consulted when the H&amp;S plan is compiled</td>
<td>0.791</td>
<td>0.814</td>
<td>3</td>
</tr>
<tr>
<td>Our workers are involved in H&amp;S inspections</td>
<td>0.832</td>
<td>0.598</td>
<td>4</td>
</tr>
<tr>
<td>Our workers can refuse to work in potentially unsafe unhealthy conditions</td>
<td>0.857</td>
<td>0.458</td>
<td>5</td>
</tr>
</tbody>
</table>

Continued on page 66
Assessment of the measurement model

The EFA statistics were valid and reliable, which enabled the measurement model to be analysed using CFA. The results in Table 3 indicate that four of the five H&S practices tested did not fit in some of the statistical indices proposed, which led to re-specification. The re-specified H&S practices were management commitment and involvement, project supervision, project H&S planning and communication, and H&S resources and training. Four of the H&S practices’ p-values were not acceptable. This was due to the large amount of data analysed which tends to produce significant results. Pallant (2013) argues that the p-value cannot be used as a solitary measure to determine the acceptable fit of a construct. The p-values of H&S performance and employee involvement and empowerment were acceptable.

The fit indices for management commitment and involvement, employee involvement and empowerment, project supervision, project planning and communication fitted after re-specification, apart from the p-values. The p-values were significant. However, the other indices fitted, indicating that the H&S practices had a good fit. To support this finding the normed chi-square was less than the recommend value of 5. The CFI and TLI were greater than the recommended value of 0.90. The RMSEA and SRMR were less than the recommended value of 0.088. This result is in agreement with the findings of Fernández-Muñiz et al (2007).

The fit indices for H&S resources and training fitted after the re-specification of the practice, apart from the TLI. The
p-value indicated a non-significant result of less than 0.05. The normed chi-square was less than 5, indicating good fitting practice. The CFI was greater than 0.90, whereas the TLI was less than 0.088, indicating weak fitting practice. The RMSEA indicated a close fit with a value of 0.088, and the SRMR indicated a good fit with a value of less than 0.08.

The fit indices for H&S performance outcome fitted after the re-specification of the practice. The p-value indicated a non-significant result of less than 0.05. The normed chi-square was less than 5, indicating good fitting practice. The CFI and TLI were greater than 0.90, and the TLI indicated good fitting practice. The RMSEA and SRMR indicated a good fit with a value of less than 0.08. The results of the measurement models suggest that the refined conceptual model of H&S can be used to determine a perfect fit model of H&S for construction SMEs.

Assessment of the structural model
Before the hypotheses were tested, the goodness-of-fit indices were examined to establish whether the data fitted the hypothesised model perfectly. The results in Table 4 indicate the goodness-of-fit indices for the structural model in Figure 3. The chi-square was significant with a p-value of less than 0.05, indicating that the null hypothesis of the model not fitting could be rejected. The normed chi-square ratio $x^2/df$ was 1.77, which was below the recommended value of 3.00. The RMSEA was 0.06, which was below 0.08, and the SRMR was 0.074, which was less than the recommended value of 0.08. The CFI was 0.849 and the TLI was 0.837. These values were less than the acceptable cut-off value of 0.90. Although the data did not fit the model perfectly, it could be described as having achieved a close fit. This enabled the results of the relationships of the constructs to be interpreted.

Figure 3 shows that six relationships could not be rejected and six were rejected. The finding indicates that upper management commitment and involvement in H&S influenced project H&S planning and communication, project supervision, and H&S resources and training. A notable finding was that upper management commitment and involvement did not directly influence employee involvement and empowerment and H&S performance as per the hypothesised model. This was contrary to the findings of Fernández-Muñiz et al. (2007) and Azimah et al. (2009) respectively. However, it can be argued that upper management commitment and involvement in H&S influenced employee involvement and empowerment indirectly through the mediating variable of project supervision. However, employee involvement and empowerment in H&S did not influence H&S performance as hypothesised.

### Table 3: Confirmatory factor analysis

<table>
<thead>
<tr>
<th>H&amp;S practices</th>
<th>No of metrics</th>
<th>$x^2$</th>
<th>Df</th>
<th>$x^2/df$</th>
<th>p-value</th>
<th>RMSEA</th>
<th>CFI</th>
<th>TLI</th>
<th>SRMR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Management commitment &amp; involvement</td>
<td>11</td>
<td>58.980</td>
<td>43</td>
<td>1.37</td>
<td>0.053</td>
<td>0.041</td>
<td>0.965</td>
<td>0.956</td>
<td>0.043</td>
</tr>
<tr>
<td>Employee involvement &amp; empowerment</td>
<td>5</td>
<td>9.00</td>
<td>5</td>
<td>1.80</td>
<td>0.1091</td>
<td>0.061</td>
<td>0.982</td>
<td>0.964</td>
<td>0.033</td>
</tr>
<tr>
<td>Project supervision</td>
<td>4</td>
<td>12.506</td>
<td>8</td>
<td>1.563</td>
<td>0.1300</td>
<td>0.051</td>
<td>0.982</td>
<td>0.966</td>
<td>0.033</td>
</tr>
<tr>
<td>Project H&amp;S planning &amp; communication</td>
<td>6</td>
<td>2.227</td>
<td>1</td>
<td>2.227</td>
<td>0.1356</td>
<td>0.075</td>
<td>0.939</td>
<td>0.961</td>
<td>0.011</td>
</tr>
<tr>
<td>H&amp;S resources &amp; training</td>
<td>5</td>
<td>10.699</td>
<td>4</td>
<td>2.68</td>
<td>0.0302</td>
<td>0.088</td>
<td>0.941</td>
<td>0.853</td>
<td>0.040</td>
</tr>
<tr>
<td>H&amp;S lagging indicator/outcome</td>
<td>9</td>
<td>55.379</td>
<td>25</td>
<td>2.22</td>
<td>0.0004</td>
<td>0.075</td>
<td>0.938</td>
<td>0.910</td>
<td>0.049</td>
</tr>
</tbody>
</table>

### Table 4: Goodness-of-fit values for the structural model

<table>
<thead>
<tr>
<th>Model</th>
<th>$x^2$</th>
<th>Df</th>
<th>$x^2/df$</th>
<th>p-value</th>
<th>RMSEA</th>
<th>CFI</th>
<th>TLI</th>
<th>SRMR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hypothesised model</td>
<td>1276.46</td>
<td>722</td>
<td>1.77</td>
<td>0.0000</td>
<td>0.060</td>
<td>0.849</td>
<td>0.837</td>
<td>0.074</td>
</tr>
</tbody>
</table>
Upper management commitment and involvement in H&S influenced H&S performance indirectly through the mediating variables of project supervision and H&S training and resources. Arguably, if upper management personnel are committed and involved in H&S, H&S performance will improve. This finding suggests that upper management involvement and commitment are critical for H&S performance success. The findings further established that employee involvement and empowerment in H&S were only influenced by project supervision. However, it was not influenced by upper management commitment and involvement in H&S, project H&S planning and communication, and H&S resources and training as originally hypothesised.

Furthermore, project H&S planning and communication did not influence H&S performance as hypothesised. These results are contrary to the findings of Fernández-Muñiz et al. (2007). However, it can be argued that Fernández-Muñiz et al. (2007) tested the H&S practices as part of the OHSMS. They suggest that the OHSMS is influenced by management commitment, which in turn influences employee involvement and H&S performance outcomes. However, testing the hypothesised H&S practices in the OHSMS individually in this study gave a better thrust to the H&S practices that are directly influenced by upper management involvement and commitment in H&S and eventually impact H&S performance.

CONCLUSION
The present study elicited information from H&S stakeholders in the construction industry regarding current H&S practices in SMEs and contributed to the existing literature by providing empirical evidence addressing the research questions. Results from the exploratory factor analysis indicated that the five H&S practices, namely upper management commitment and involvement in H&S, employee involvement and empowerment in H&S, project supervision, project H&S planning, communication in H&S and H&S resources and training, and the H&S performance outcomes were valid and reliable measures of H&S in SMEs. Attaining the desired levels of validity and reliability enabled the confirmatory factor analysis to be undertaken. The results indicated that the five H&S practices and H&S performance achieved an acceptable fit model to enable the structural model to be analysed. The results of the structural equation modelling indicated that upper management commitment and involvement in H&S influenced H&S performance indirectly through the mediating variables of project supervision and H&S training and resources. Each relationship was positive and significant.

The findings inform SME contractors involved in building and civil engineering that upper management personnel should be committed and involved in H&S as they are the catalyst to ensure improvement in H&S performance. They will ensure that project supervision is undertaken to eliminate any hazards involved in the project. Furthermore, they will ensure that H&S resources and training are provided to their employees. In addition, the significant relationships in the tested model which influenced the H&S performance outcome can be deemed to be the minimum H&S practices that construction SMEs in civil and building projects should adopt. This will ensure that they have a small set of H&S practices to implement to improve their H&S performance outcomes, thereby ensuring that H&S culture is improved.

This finding is important, especially in a country where H&S is not given the necessary attention. It is therefore suggested that upper management commitment and involvement in H&S, project supervision and H&S resources and training should constitute the construction H&S performance improvement model (CHSPIM). Finally, the findings of this study have contributed to the H&S body of knowledge in the South African construction industry.

RECOMMENDATIONS TO THE INDUSTRY
The managerial and practical implications of this study that should assist civil and building contractor SMEs to improve their H&S performance are three non-negotiable H&S practices. These H&S practices are upper management commitment and involvement in H&S, project supervision, and H&S resources and training. In addition, the proposed H&S model will enable H&S culture to be developed at project level for construction SMEs.

Furthermore, it can be argued that South African construction SMEs require only a few manageable H&S practices that would assist them in managing their H&S activities effectively. This is because construction SMEs do not have the resources that large construction firms have to enable them to implement a large number of H&S practices. Furthermore, these H&S practices would act as early warning signs that an accident will occur if they are not implemented at all or not implemented properly.

In addition, construction SMEs may find the developed questionnaire useful for audits of their H&S performance and benchmarking their H&S practices against those of their competitors. The practices can eventually be used to identify H&S areas that are problematic in the workplace and that require special and immediate attention.

Finally, the H&S practices can be used to improve the minimal compliance with the requirements of the 2014 Construction Regulations so as to reduce site closures by the Department of Labour in South Africa due to non-compliance.

LIMITATIONS OF THE STUDY
Despite the findings of this study and the recommendations for future research, several limitations were identified, namely:
- The majority of the respondents conducted their business in Gauteng Province, which prevents the findings from being generalised to all the provinces of South Africa.
- The use of the self-reporting in the questionnaire for the independent, intervening and dependent variable is vulnerable to bias reporting, which is acknowledged in this study.

FURTHER STUDY
It is recommended that further research be done. An employee survey is advocated, as the current study focused on upper management personnel and those knowledgeable about the current H&S practices in their organisation. It is proposed that these practices be validated using structural equation modelling to determine their relationship to and influence on H&S performance, i.e. reduction in accidents, injuries and damage to property, and to improve the motivation of the workforce. Finally, the use of field observations is also recommended for future study to overcome the bias that a self-reporting questionnaire is perceived to have.

ACKNOWLEDGEMENT
The research funding support of the National Research Foundation (NRF) is
REFERENCES
Arboleda, A, Morrow, P C, Crum, M R & Shelley, M 2008. The
Federated Employer Mutual Assurance 2017. FEM’s accident stats as at June 2017. Available at: http://www.fem.co.za/Layer_SL/FEM_Home/FEM_Accident_Stats/FEM_Accident_Stats.htm (accessed on 9 August 2017).
HSE (Health and Safety Executive) 2008. Successful health and safety. Richmond, UK: HSE.
Labour Department (Occupational Safety and Health Branch) 2016. Occupational Safety and Health Statistics Bulletin (Hong Kong), 16: 1–8.
The Journal of the South African Institution of Civil Engineering is published quarterly in March, June, September and December. Articles submitted for publication are reviewed by a panel of referees under the guidance of the SAICE Journal Editorial Panel. The journal publishes research papers covering all the disciplines of civil engineering (structural, geotechnical, railway, coastal/marine, water, construction, environmental, municipal, transportation) and associated topics that are relevant to the civil engineering profession, and that preferably have relevance to civil engineering in southern Africa and the African continent. When preparing articles for publication, authors should please take note of the following and comply with the guidelines as set out:

CLASSIFICATION OF ARTICLES CONSIDERED FOR PUBLICATION

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

POLICY REGARDING LANGUAGE AND ORIGINALITY OF SUBMITTED ARTICLES

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

SUBMISSION PROCEDURES AND REQUIRED FORMAT

- Online submission: Manuscripts must be uploaded as PDF files (http://journal.saic.e.org.za). Individual file sizes may not exceed 4 MB. Should you experience problems uploading your paper, please contact the editor (verelene@saice.org.za).
- Format: Manuscripts should be prepared in MS Word and presented in double line spacing, single column layout with 25 mm wide margins. Line numbers must be applied to the whole document. All pages should bear the authors’ names and be numbered at the bottom of the page. With the exception of tables and figures (see below) the document should be typed in Times New Roman, 12 pt font. Contributions should be accompanied by an abstract of not more than 200 words.
- First page: The first page of the manuscript should include the title of the paper, the number of words of the main text (i.e. excluding figures, tables and the list of references), the initials and surnames of the authors, professional status (if applicable), SAICE affiliation (Member, Fellow, Visitor, etc), telephone numbers (landline and mobile), and e-mail and postal addresses. The name of the corresponding author should be underlined. Five keywords should be suggested.
- Figures, tables, photos and illustrations: These should preferably be submitted in colour, as the journal is a full-colour publication. Their positions should be clearly marked in the text as follows: [Insert Figure 1]. Figures, tables, photos, illustrations and equations should be numbered consecutively and should appear in the text directly after they have been referred to for the first time. Illustrations must be accompanied by appropriate captions. Captions for tables should appear above the table. All other captions should appear below the illustration (figures, graphs, photos). Only those figures and photographs essential to the understanding of the text should be included. All illustrations should be referred to in the text. Figures should be produced using computer graphics. Hand-drafted figures will not be accepted. Lettering on figures should be equivalent to a Times New Roman 9 pt font or slightly larger (up to 12 pt) if desired. Lettering smaller than 9 pt is not acceptable. Tables should be typed in Times New Roman 9 pt font. They should not duplicate information already given in the text, nor contain material that would be better presented graphically. Tabular matter should be as simple as possible, with brief column headings and a minimum number of columns.
- Mathematical expressions and presentation of symbols: Equations should be presented in a clear form which can easily be read by non-mathematicians. Each equation should appear on a separate line and should be numbered consecutively. Symbols should preferably reflect those used in Microsoft Word Equation Editor or MathType, or should be typed using the Times New Roman symbol set.
- Variables in equations (e.g. y, z, etc) as well as lower case Greek letters should be presented in italics. Numbers (digits), upper case Greek letters, symbols of metric measurement units (m for metres, s for seconds, etc) and mathemati-cal/trigonometrical functions (such as sin, cos and tan) are not written in italics, but in upright type (Roman). Variables and symbols used in the body of the text should match the format used in the equations, i.e. upright or italics, whichever is applicable.
- Metric measurement abbreviations/units should conform to international usage – the SI system of units should be used.
- Decimal commas may be used, but decimal points are preferred.
- Symbols should preferably be defined in the text, but if this is not possible, a list of notations may be provided for inclusion at the end of the paper.
- Headings: Sections and paragraphs should not be numbered. The following hierarchy of headings should be followed:

**HEADING OF MAIN SECTION**

- **Heading of subsection**
  - **Heading of sub-sub-section**

- **References:** References should follow the Harvard system. The format of text citations should be as follows: “Jones (1999) discovered that…” or “recent results (Brown & Carter 1985; Green et al 1999) indicated that…” References cited in the text should be listed in alphabetical order at the end of the paper. References by the same author should be in chronological order. The following are examples of a journal article, a book and a conference paper:

Papers published previously in the *Journal of the South African Institution of Civil Engineering* should be cited if applicable.

- **Footnotes, trade names, acronyms, abbreviations:** These should be avoided. If acronyms are used, they should be defined when they first appear in the text. Do not use full stops after abbreviations or acronyms.

- **Return of amended papers:** Papers requiring amendments will be accepted up to six months after the referee reports had been sent to authors, after which the paper will be withdrawn from the system.

**FINAL ARTICLE**

- **Copyright:** On acceptance of the paper or technical note, copyright must be transferred by the author/s to the South African Institution of Civil Engineering on the form that will be provided by the Institution.

- **Photos of authors:** The final corrected version of the paper should be accompanied by recent, high-resolution head and shoulders colour photographs and a profile not exceeding 100 words for each of the authors.

- **Proofs:** First proofs of papers will be sent to authors in PDF format for verification before publication. No major re-writes will be allowed, only essential minor corrections.