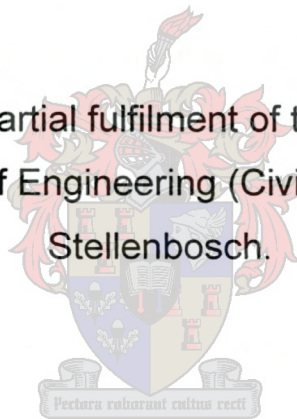# IMPOSED LOADS FOR INACCESSIBLE ROOFS OF LIGHT INDUSTRIAL STEEL BUILDINGS

By

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Thesis presented in partial fulfilment of the requirements for the degree of Master of Engineering (Civil) at the University of Stellenbosch.

**Supervisor**

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## **Declaration**

I, the undersigned, hereby declare that the work contained in this thesis is my own original work and that I have not previously in its entirety or in part submitted it at any university for a degree.

Date:

P.J. de Villiers

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## SYNOPSIS

A critical evaluation of provisions for imposed loads in the South African Loading Code for design of structures, SABS 0160-1989 (SABS), by comparison with other codes was performed earlier. The evaluation revealed the SABS loading code to be generally non-conservative in its provisions for imposed loads for a range of general and specialist occupancy classes. The SABS provision for imposed loads for inaccessible roofs was found to be *substantially non-conservative* in comparison with the other codes. An investigation into the imposed load for inaccessible roofs is subsequently performed in order to establish a scientific rationale through which the codified design values may be measured effectively. Due to the lack of information and the large uncertainties involved in the imposed roof load, stochastic treatment of the loads is implemented. This is in line with the stochastic modelling of loads as implemented in general.

The approach applied is to select a type of building that can be regarded as a generic example of buildings to which these loads apply, and to discretisize the load into the various sub-mechanisms that translate into the imposed roof load. The probabilistic models for the load mechanisms are then quantified, either through physical load surveys, or through conducting an expert survey for those variables which are not observable. The use of expert opinion as a resource for information is not readily accessible in terms of yielding scientifically defendable results. Therefore, the expert survey is performed as a calibrated experiment whereby weights were calculated for the individual experts' opinions and their opinions combined accordingly.

The probabilistic models for the load mechanisms are then translated into load effects by taking into account the physical process resulting in the load effects. By applying these mechanisms in such a way as to maximise the said load effects, equivalent uniformly distributed loads (EUDL's) were calculated for each mechanism. The probabilistic models obtained in terms of the EUDL's pose an easily accessible format through which existing load models and codified provisions can be evaluated. These load models are then utilised to evaluate the SABS provisions in terms of the level of reliability catered for by SABS ultimate limit-state design criteria. It is concluded that the SABS conservatively provides for maintenance loads on the roof, while the reliability for construction loads is non-conservative for large tributary areas and highly non-conservative for small areas. The load models so obtained can further be applied for structural reliability assessment.

# OPSOMMING

'n Kritiese evaluasie van die voorskrifte vir opgelegde belastings in die Suid-Afrikaanse Belastingskode vir die ontwerp van strukture, SABS 0160-1989 (SABS) deur 'n vergelyking met ander kodes is vroeër uitgevoer. Die evaluasie het getoon dat die SABS in die algemeen onkonserwatief is in sy voorsiening vir opgelegde belastings oor 'n bereik van algemene en spesialis okkupasie tipes. Die SABS voorskrif vir opgelegde belastings vir ontoeganklike dakke is hoogs onkonserwatief in vergelyking met die ander kodes. 'n Ondersoek na die opgelegde belasting vir ontoeganklike dakke word gevolglik uitgevoer met die doel om 'n wetenskaplike rasionaal daar te stel waardeur die gekodifiseerde voorskrifte effektief gemeet kan word. As gevolg van die gebrek aan inligting en groot onsekerhede betrokke by die opgelegde dakbelasting word stogastiese modellering geimplimenteer.

Die aanslag wat gevolg is, is om 'n tipe gebou te selekteer wat beskou kan word as verteenwoordigend van die geboue waarvoor hierdie belastings van toepassing is, en om die belasting te diskretiseer in die verskeie lasmeganismes wat die opgelegde dakbelasting voortbring. Die waarskynlikheidsmodelle vir die lasmeganismes word dan gekwantifiseer, óf deur fisiese opnames, óf deur die uitvoering van 'n ekspert-opname vir daardie veranderlikes wat nie waarneembaar is nie. Die gebruik van ekspert opinie as 'n bron van inligting is nie maklik toeganklik in terme daarvan om wetenskaplik verdedigbare resultate te lewer nie. Daarom is die ekspert-opname uitgevoer soos 'n gekalibreerde eksperiment waardeur relatiewe gewigte bereken word vir die individuele eksperts en hul opinies daarvolgens gekombineer word.

Die waarskynlikheidsmodelle vir die lasmeganismes word dan omgeskakel in las-effekte deur in agneming van die fisiese proses wat die las-effek voortbring. Deur die lasmeganismes op só 'n manier toe te pas dat die betrokke las-effekte gemaksimeer word, word ekwivalent uniforme belastings (EUB's) bepaal. Die waarskynlikheids-modelle in terme van EUB's bied 'n maklik toeganklike formaat waardeur bestaande lasmodelle en gekodifiseerde voorskrifte evalueer kan word. Die lasmodelle word gevolglik gebruik om die SABS voorskrifte te evalueer in terme van die vlak van betroubaarheid wat gehandhaaf word deur SABS limiet-staat ontwerp kriteria. Dit is bepaal dat die SABS konserwatief voorsiening maak vir onderhoudslaste op die dak, maar onkonserwatief tot hoogs-onkonserwatief is vir konstruksie laste. Die bepaalde lasmodelle kan verder toegepas word in strukturele betroubaarheids analise.

**Bedankings**

Aan die Universiteit van Stellenbosch, vir die geleentheid asook die finansiële ondersteuning. Aan my studieleier, Prof. JV Retief, vir sy leiding, insig, en tyd. Aan my ouers wie altyd die lig aan die einde van die tonnel kon sien, en aan Annalize, my aanstaande, wie die lig was in die tonnel.

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## CHAPTER 1: INTRODUCTION AND OVERVIEW

In current practice, the SABS 0160-1989 (SABS) provision for imposed loads of inaccessible roofs seems to satisfy the requirements for safety and reliability of structures subject to these loads, with failures being few and far between. However, the SABS provision for imposed loads on inaccessible roofs is based on rule of thumb, and to date no rational evaluation has been performed. An investigation into the imposed load for inaccessible roofs is therefore required in order to establish a scientific rationale through which the codified design values may be measured effectively. Due to the lack of information and the large uncertainties involved in the imposed roof load, stochastic treatment of them is implemented. This is in line with the stochastic modelling of loads as implemented in general. The stochastic treatment of load variables provides the basis through which the level of safety, provided for by SABS design provisions, may be measured quantitatively. Rational treatment contributes both to establishing acceptable levels of reliability, and to improve the economy of the structure.

As an expansion of the introduction to this investigation, the results of a critical evaluation of imposed loads in general, by comparison of SABS provisions with those of other loading codes, are shown in Section 1.1 hereafter. Through this comparative study, it becomes clear that the SABS provision for imposed loads of inaccessible roofs in particular, is *substantially non-conservative* in comparison with the other codes. This, together with the fact that the current SABS provision is not based on any scientific rationale, provides ample motivation for conducting the investigation.

A sensitivity study is subsequently performed in order to determine the significance of imposed roof loads in the design of light industrial steel buildings. This involves identification of failure mechanisms where imposed roof loads have an influence, measurement of the extent of the imposed roof load's effect and characterising the type of buildings for which imposed roof loads plays a governing role in the design. The purpose of the sensitivity study is to familiarise the reader with the codified application of imposed roof loads, as well as to justify that imposed roof loads are sufficiently important in the design of light industrial steel buildings to warrant the investigation.

The remaining part of the investigation is concerned with establishing the probabilistic models which define the imposed loads for inaccessible roofs. Ideally, one would

want to establish a general load model covering the whole spectrum of imposed loads on inaccessible roofs and applicable to all types of buildings. However, such an approach is bound to result in inaccurate approximations of reality and for certain load cases and building types, gross deviations are to be expected. Rather, the approach applied is to select a type of building that can be regarded as a generic example of buildings to which these loads apply, and to discretisize the load into the various sub-mechanisms that translate into the imposed roof load. The load mechanisms are identified through consultation with individuals who, in principle, have knowledge of imposed roof loads. Such individuals are referred to as experts.

The load mechanisms are then quantified, which involves determining the expected value as well as the uncertainty associated with the load mechanisms. To that end it is necessary to obtain information on imposed roof loads. The information is to be obtained through *expert measurement*, that is a survey amongst selected experts with relevant expertise in fields applicable to this investigation. An unmethodological use of expert opinion will not contribute to rational decision making. Therefore, it is imperative that experts are selected scrupulously, with careful consideration of their fields of expertise, and that their opinions are obtained and managed in a scientific manner that yields rationally defendable results. The expert survey is performed as a calibrated experiment whereby weights were calculated for the individual experts' opinions and their opinions combined accordingly. The experts' opinions are utilised to provide data on the *maximum values* of the load mechanisms, which are not observable or measurable through any kind of physical load survey. A physical load survey is conducted to a lesser extent for certain load mechanisms, with the purpose of providing information on the *average values* of the load mechanisms, which is to serve as a means of calibrating the experiment.

The probabilistic models for the load mechanisms resulting from the expert survey represent the uncertainty in the *magnitude* of the load mechanisms. The *spatial variability* of the load mechanisms is accounted for in the process of converting these load mechanisms to equivalent uniformly distributed loads (EUDL's). The approach is to translate the load mechanisms into load effects by taking into account the physical process resulting in the load effects. By applying these mechanisms in such a way as to *maximise* the said load effects, an EUDL may be calculated for each mechanism. Through this method, conservative allowance is made for the spatial variability of the loads. The probabilistic models obtained in terms of the EUDL's

pose an easily accessible format through which existing load models and codified provisions could be evaluated.

The load models subsequently obtained are to be compared to existing codified provisions in order to assess the degree of agreement and to identify where existing provisions are non-conservative. The probabilistic models established for the various load mechanisms may be put forward in a probabilistic model code, providing specifically for the said load mechanisms. A degree of generalisation will also be implemented here, i.e. the obtained load models are to be consolidated and refined so as to represent imposed roof loads in general and not only cater for the specific load mechanisms.

The final step is to perform a reliability analysis on the failure mechanisms identified in the sensitivity study. Current SABS load and resistance factors, together with the computed distribution of the imposed roof load and existing dead load and resistance models are to be utilised in this process. The results from the reliability analysis will be used to evaluate the performance of the SABS 0160-1989 in providing for the imposed load of inaccessible roofs.

## 1.1 An Evaluation of Imposed Loads for Application to Codified Structural Design

Revision of the South African Code of Practise for the General Procedures and Loadings to be Adopted in the Design of Buildings (SABS 0160-1989) provided the opportunity to reassess the provisions made for minimum imposed loads prescribed in the code. There is no evidence that the present provisions are deficient, nor are there substantial recent research results to warrant substantial revision of the relevant prescriptions. Therefore, the review was limited to a thorough comparison with other structural design loading codes.

A critical evaluation of provisions for imposed loads in the South African Loading Code for design of structures, SABS 0160-1989, by comparison with other codes was performed earlier by RETIEF, DUNAISKI and DE VILLIERS (2000). The investigation comprises of a broad and extensive comparison where all aspects of imposed loads are considered, followed by an intensive evaluation of critical aspects identified in the broad survey. The scope of the type of loading that was considered such as floors, roofs, area effects, walls, distributed and concentrated loads, etc., is fairly well developed and therefore does not vary significantly between the various codes. The degree of detail to which occupancy types are provided for varies significantly amongst the codes. Since this factor has various implications, it was evaluated more carefully. Most attention was given to the quantitative values for the minimum imposed loads.

### 1.1.1 Basis for Imposed Load Values

Simplified models of loads such as uniformly distributed loads, which are a function of area, are used to reflect the load effects of the various activities that could occur during the life of the building structure. Conservative assumptions are made to obtain design values. Realistically conservative values can be obtained through the judgement of experienced designers and the performance of structures designed to previous codes. Load surveys are sources of objective information on actual loads. The statistic nature of load surveys, together with incompleteness, require stochastic treatment of load survey data. Reliability modelling provides significant improvement in the rationality of imposed load values. The prudent approach is only to reduce sufficiently conservative loads to the extent that the necessary evidence can substantiate such reduction.

## 1.1.2 Codes Selected for Comparison

A representative set of four loading codes was selected for use as a basis in the evaluation of the SABS Loading Code. The codes, as well as a motivation for their selection, are presented in Table 1. The primary objective of the study was to identify deficiencies in the SABS to be rectified in code revision. The formal adoption of an international suite of structural design codes is an option to be considered for code development for South Africa (South African National Conference on Loading (1998)). Particular emphasis was therefore placed on Eurocode and ASCE 7 as primary contenders as a reference code. A truly international code will be the ideal solution; ISO 2394 is therefore included as a reference, although it is not suitable for current practice.

### Table 1. Codes Used for Evaluation of SABS 0160-1989 Imposed Loads

| Code Prescribing Imposed Loads | Short Title | Motivation |
|---|---|---|
| **SABS 0160-1989** (Revised 1990) *General Procedures and Loadings to be Adopted in the Design of Buildings* | SABS Code | Evaluation of prescription of imposed loads. |
| **BS 6399 Part 1-1996** *Dead and Imposed Loads; Part 3: 1988 Imposed Roof Loads* | BS Code | Served historically as reference to code development for South Africa. |
| **ENV 1991-2-1:1995 Part 2-1** *Actions on Structures* | Eurocode | Results from comprehensive code development process; to replace BS. Important international reference. |
| **AS 1170.1-1989 Part 1:** *Dead and live loads and load combinations* | AS Code | Similarities in conditions between the two countries. |
| **ANSI/ASCE 7-95** *Minimum Design Loads for Buildings and Other Structures* | ASCE Code | Important source of reference for structural design internationally, and for South Africa. |

## 1.1.3 Basis for Comparison of Code Provisions

The comparison of provision for imposed loads in various structural design codes is complicated by the fact that these codes vary significantly in philosophy and approach, layout and structure, the manner in which safety is partiallised throughout

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the code and even the level of reliability. A common basis was therefore required, to which the properties of the various codes could be mapped for comparison.

The approach followed was first to use the most extensive range of factors such as load and occupancy type, as the basis for a comprehensive comparison. This ensured that all the factors related to imposed loads considered in all the codes were substantially reflected in the comparison. The factors considered in the study were uniformly and concentrated floor and roof loads and area load reductions, imposed loads on walls and balustrades, all considered against an extensive complement of occupancies. Critical aspects in need for more detailed evaluation were then addressed in an intensive survey, where SABS provisions for imposed loads were measured against the other codes. Particular attention was given to provisions of the two potential codes of reference, ASCE 7 and ENV 1991-2-1.

### 1.1.4 Extensive Comparison of All Codes

All aspects of imposed loads that need to be considered for structural design were taken into account in the extensive comparison of the various codes. In spite of substantial differences in approach and detail of treatment of relevant aspects, a reasonable match could be compiled for comparison. A rather detailed exercise was required, which is fully reported by DE VILLIERS, RETIEF and DUNAISKI (2000). Aspects that were considered are summarised in Table 2.

**Table 2. Scope of Extensive Comparison of Codes**

| General aspects of code provisions that were compared | • Pattern loading, permanency of imposed loads, dynamic forces, impact, movable partitions for floor loads<br>• Imposed floor load reduction<br>• Minimum concentrated floor load intensities<br>• Minimum imposed roof loads: classification and load intensities; load reduction; curved roofs; additional loads on roof trusses.<br>• Forces on parapet walls, balustrades and railings. |
|---|---|
| Minimum imposed floor loads as a function of occupancy | • Building and floor occupancy classification used as a basis for imposed load values<br>• Comparison of imposed load values prescribed by the various codes, for all possible occupancy types<br>• Treatment of special components of facilities such as stairs, landings, corridors, hallways, cantilever balconies<br>• Treatment of special occupancies such as industrial and storage areas |

### General Properties of Imposed Floor Loads

A systematic comparison was made of the methods and values in which the properties of imposed loads, as summarised in Table 2, are specified in the various codes. A summary of the most important aspects is presented in Table 3.

**Table 3. General Properties of Imposed Floor Loads**

| SABS 0160-1989 | ASCE 7-95 | ENV 1991-2-1 | BS 6399-1-1996 | AS 1170.1-1989 |
|---|---|---|---|---|
| Pattern Loading | | | | |
| Applied to produce most severe effects. | Applied to a portion of structure if effects are more severe. | Applied to most unfavourable tributary zone per storey. | No provisions. | No provisions. |
| Movable Partitions | | | | |
| Load intensities are prescribed. Distinction between different weights of partitions. | Provision where specified imposed load is smaller than 3.8 kN/m². No prescribed load intensities. | No provisions. Partitions are taken as part of the nominal dead load. | Same as SABS. Less conservative prescribed minimum. No distinction is made for different weights of partitions. | Same as SABS. Less conservative prescribed minimum. No distinction is made for different weights of partitions |
| Reduction of Prescribed Minimum Imposed Load Intensities | | | | |
| Reduction is only dependent on the tributary area of the member. For assembly, storage, manufacturing and garaging areas the maximum reduction is 30% where the area is greater than 80m². For other occupancies the maximum reduction is 50% where the area is greater than 20m². | Reduction is dependent on the influence area, member type and number of floors supported by the member. No reduction applies to assembly areas. For one floor the maximum reduction is 50%. For more than one floor the maximum reduction is 60%. | Reduction is dependent on tributary area, member type and number of floors, and does not apply to storage, heavy industrial or garaging areas. For areas greater than 20m² reductions of 30, 40 and 50% applies to light storage, assembly and shopping, and other areas respectively. | Reduction is dependent on tributary area, member type and number of floors, and does not apply to storage, heavy industrial or garaging areas. For vertical members, the maximum reduction is 50%. For horizontal members, the maximum reduction is 25%. | Reduction is only dependent on the tributary area of the member and does not apply to storage, industrial or garaging areas, or two one-way spanning slabs. For imposed loads greater than 5 kN/m² the maximum reduction is 20% and for imposed loads smaller than 5 kN/m² the maximum reduction is 50%. |

### Minimum Concentrated Imposed Floor Load Intensities

Provision for minimum concentrated imposed floor loads in the SABS code generally compared well with that of the other codes, and no deficiencies could be identified.

### Minimum Uniformly Distributed Imposed Floor Load Intensities

Minimum values for uniformly distributed imposed floor loads for the various occupancy types form a vital component of a loading code. A proper comparison between the various codes requires a comprehensive list of occupancy types as a basis. This was done by using the AS code, with its eight building types, with an average of 15 floor uses each, as a reference. Where necessary the list was extended to include all occupancies provided for in all the codes. Thus, the imposed

load values of the various codes could be compared over the full spectrum of floor occupancies.

The comparison gave persuasive evidence that imposed load values prescribed by the SABS are systematically lower than those of the other codes. This is true for some parts of residential areas and areas such as offices, parking areas, storage and others. In a number of instances the differences were substantial, for example stairs and corridors, classrooms and retail areas. However it was also clear that the extensive comparison magnified this point significantly.

### *General Properties of Imposed Roof Loads*

Aspects of imposed roof loads generally provided for in the various codes are the classification of roofs, load intensities and their reduction for different types of roof, curved roofs, additional loads on roof trusses, accidental loads during maintenance and provision for snow load if not provided for separately. Load intensity is the most important component of the provisions. The comparison is summarised in Table 4.

### *Forces on Parapet Walls, Balustrades and Railings*

All of the loading codes classify parapet walls, balustrades and railings into a number of categories according to the type of area they serve. Imposed load values are then assigned to the various categories. The total number of categories for the various loading codes provides an indication of the level of detail to which loads are specified: four to six categories are generally used, except for the BS code which uses 14 categories. There is substantial agreement between SABS and the other codes.

**Table 4. Comparison of Imposed Roof Load Intensities**

| SABS 0160-1989 | ASCE 7-95 | ENV 1991-2-1 | BS 6399-1-1996 | AS 1170.1-1989 |
|---|---|---|---|---|
| *Accessible roofs:* Load values are prescribed. If the roof is to be used as a floor, the prescribed floor load intensity applies.<br><br>*Inaccessible roofs:* Load values are prescribed. A reduction with increase in tributary area is allowed. | *Special-Purpose roofs:* Load values are prescribed. The values are more conservative than those of the SABS.<br><br>*Ordinary, flat or curved roofs:* Load values are prescribed. Reduction is based on tributary area and roof slope. Values are substantially more conservative than SABS. | *Accessible roofs:* Use floor load values.<br><br>*Special-Purpose roofs:* Load values to be determined for the particular case.<br><br>*Inaccessible roofs:* Prescribed load values are dependent on roof slope only. For roof slopes smaller than 20°, the values are substantially more conservative than SABS. | *Accessible roofs:* Values are prescribed. Provision for snow loads. Values are less conservative than SABS.<br><br>*Inaccessible roofs:* A distinction is made between general buildings and small buildings. Load values dependent on roof slope are prescribed. The values are more conservative than SABS. | *Accessible roofs:* Values are prescribed. Reduction based on tributary area. Differentiate between houses and other buildings. Values are more conservative than SABS.<br><br>*Inaccessible roofs:* Load values are prescribed. Reduction with tributary area. Values close to the SABS over the range of tributary areas. |

## 1.1.5 Intensive Comparison of Other Loading Codes to SABS

The broad and extensive comparison of the various codes was followed by an intensive evaluation of the SABS code, considering areas of discrepancies and potential deficiencies. Attention was primarily given to imposed floor load values and the corresponding definition and classification of occupancies. The classes of building or floor zones used in SABS 0160 was used as the basis for the comparison; a few occupancies not specified in the SABS code were added to the set. This comparison not only provides focus on the critical aspects of the SABS code that were identified through the extensive evaluation; it also provides a more balanced evaluation of the relative rating of the SABS code. The complete intensive comparison is reported in RETIEF (2000).

## 1.1.5.1 Comparison of Floor Load Intensities using the SABS 0160-1989 Table 4 as Basis

The load categories used in SABS 0160 Table 4 to group all occupancy classes with a common imposed load intensity were subdivided to provide compatibility with occupancies and imposed load values of other codes. The approach applied was to maintain the overall structure of the extended SABS Table 4, but to separate occupancy classes identified from differences with other codes, whilst maintaining a minimum set of sub-categories. A summary of the comparison of the distributed

imposed load values of the SABS code to that of the other codes is given in Table 5 (values are in terms of kN/m²). A representative and synoptic description of occupancies, grouped together in SABS load categories, is provided. Average values of the ratio of values of each of the other codes to that of the SABS code is also tabulated. Average ratios are given per occupancy class and per code.

It is quite apparent that SABS imposed load values are, excepting a few instances, significantly and systematically lower than that of the other codes. Although weighted average ratios are required to provide a proper reflection of the comparative imposed load values, inspection shows that values are of the order 16% to 18% lower for SABS. The exception is the BS code, which prescribes on average only marginally higher imposed loads.

### Table 5. Comparison of SABS Imposed Load Values to Other Codes

| OCCUPANCY | SABS | ASCE | ENV | BS | AS | Ratio |
|---|---|---|---|---|---|---|
| Dwelling house/unit | | 1.4 | | 1.5 | 1.5 | 1.07 |
| Bedrooms, wards, dormitories, etc in hospitals, hotels | 1.5 | | 2 | 2 | 2 | 1.32 |
| Corridors, lobbies, landings to dwelling house | | 1.9 | | 1.5 | 3 | 1.42 |
| Stairs to dwelling house | | | 3 | | | 2.00 |
| Classrooms, lecture theatres | | 1.9 | 3 | 3 | 3 | 1.36 |
| Operating theatres, x-ray rooms | 2 | 2.9 | 3 | 2 | 3 | 1.36 |
| Reading rooms in libraries | | 2.9 | 3 | 2.5 | 2.5 | 1.36 |
| Garages, parking areas: < 25 kN gross weight | 2 | 2.4 | 2 | 2.5 | 3 | 1.24 |
| Offices for general use | 2.5 | 2.4 | 3 | 2.5 | 3 | 1.09 |
| Offices with data processing equipment | 3 | | | 3.5 | | 1.06 |
| Cafes, restaurants, dining rooms, lounges | | 4.8 | 3 | | 2 | 1.09 |
| Kitchens, laundries in hotels, offices, educational etc | | | 2 | 3 | 4 | 1.00 |
| Communal bathrooms, toilets in hotels, offices, etc | 3 | 1.9 | 2 | 2 | 2 | 0.66 |
| Entertainment areas | | 3.6 | | 3 | | 1.10 |
| Light industrial | | 6 | | 2.5 | 4 | 1.39 |
| Assembly areas; fixed seating in residential buildings | | 2.9 | 4 | 4 | 3 | 0.87 |
| Assembly halls, theatres, sport complex; fixed seats | 4 | 2.9 | 4 | 4 | 4 | 0.93 |
| Grandstands with fixed seating | | 4.8 | | 5 | 5 | 1.23 |
| Retail shops, department stores: sales and display | | 4.8 | 5 | 4 | 5 | 1.18 |
| -upper floors | 4 | 3.8 | | | | 1.10 |
| Light laboratories, banking halls | | | | 3 | 3 | 0.75 |
| Assembly halls, sport complex; without fixed seats; stairs, corridors, landings of grandstands; public assembly areas, cantilever balconies | 5 | 4.8 | 5 | 5 | 5 | 1.00 |
| Stages to assembly halls, theatres | | 7.2 | | 7.5 | 7.7 | 1.36 |
| Filing and storage: offices, hotels, institutions | | | 6 | 5 | 5 | 1.07 |
| Stack rooms: books, stationary | 5 | | | 2.4/m | 4/m | |
| Shelved areas to libraries | | 7.2 | 6 | 4 | 3.3/m | 1.15 |
| Exhibition halls | | | 5 | 4 | | 0.9 |
| Ratio of values relative to SABS – Average | | 1.18 | 1.17 | 1.05 | 1.18 | 1.16 |
| – Standard deviation | | 0.34 | 0.30 | 0.21 | 0.33 | 0.26 |

## 1.1.5.2 Evaluation of SABS 0160-1989 Uniformly Distributed Imposed Load Intensities

An evaluation of the distributed load intensities of SABS relative to the values of the other codes is summarised in Table 6 for the critical occupancy classes. The comparison is primarily made against ASCE 7 and Eurocode as reference codes. The availability of reliability based load models as provided in the Probabilistic Model Code (JCSS 2000) is also listed. Load models can be applied in the adjudication process, whereby the compatibility of the SABS values to a selected reference code is justified through verification with the load model for that particular load.

Important occupancy classes for which uniform imposed floor loads need to be reconsidered for SABS are those which cover large areas in certain buildings: bedrooms, wards and alike for residential buildings; offices; retail areas; garages and parking areas. Although increased loads for general occupancies will influence structural cost over large areas, these loads are also related to wide public exposure to structural performance. Satisfactory motivation will have to be provided for not being compatible with at least one of the reference codes.

Specialist areas to be reconsidered are lecture and operating theatres, reading rooms, restaurants, entertainment areas, stages, commercial storage and shelved areas. Auxiliary areas such as stairs, corridors and lobbies need to be carefully evaluated to provide for the effects of crowding and vehicles.

A discrepancy exists in the way in which the SABS and the other codes assign imposed load values to storage and industrial areas. The SABS and the Eurocode assigns specific minimum load intensities to these areas and states that the load should be determined for the specific case in practise. The AS, BS and ASCE codes presents specific examples of such floor occupancies to which imposed load values are assigned (exceeding the absolute minimum given by the SABS) and in the case of storage areas the assigned load values are dependent on the height of the stacked materials. For relatively low storage heights of stacked materials the SABS is already non-conservative in comparison with the BS and AS.

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**Table 6. Comparison of Selected Imposed Load Intensities**

| SABS | | Occupancy class | Evaluation and Recommendations | Load model available |
| Cat | Value KN/m² | | | |
|---|---|---|---|---|
| 1 | 1.5 | (a) All rooms in a dwelling unit and dwelling house | | Residence |
| | | (b) Bedrooms, wards, dormitories, private bathrooms and toilets in educational buildings, hospitals, hotels and other institutional occupancies | Category 1(b) represents large areas with exposure to the public. An increase to 2 kN/m² will achieve compatibility with reference codes. | Hotel guest room, Patient room, Lobby |
| | | (c) Corridors, stairs, lobbies, landings to a dwelling house | | |
| | | (d) Stairs | Compatibility with Eurocode – 3 kN/m² | |
| 2 | 2 | (a) Classrooms, lecture theatres | Use load model to adjudicate between ASCE (2 kN/m²) & Eurocode (3 kN/m²) | School room |
| | | (b) Operating theatres, x-ray rooms | An increase to 3 kN/m² will achieve consensus with other codes | - |
| | | (c) Reading rooms in libraries | | Libraries |
| 3 | 2 | Garages and parking areas for vehicles of gross weight < 25 kN | Reference to ASCE requires increase | |
| 4A | 2.5 | Offices for general use | Reference to Eurocode requires increase | Office |
| 5 | 3 | Entertainment | Reference to ASCE requires increase | |
| | | Light industrial | Reference to ASCE requires substantial increase. Refine definition. Use model. | Light industrial |
| 6 | 4 | Grandstands with fixed individual seating | Confirm merit of reduced load for seating by SABS as compared to other codes | |
| 7 | 4 | Sales and display areas in retail shops and department stores | Lower than both ASCE & Eurocode. Use load model to evaluate change | Merchant / Retail |
| 9 | 5 | Shelved areas to libraries | Substantially lower than Eurocode and particularly ASCE values | Storage |
| 10 | 3 | Corridors, stairs, lobbies, aisles, hallways, landings to buildings other than Category 1 | SABS: same as adjacent room, but ≥ 3 kN/m². Other codes consider crowd loads and wheeled vehicles, with much higher loads | Concentration of people |

Although differences between the SABS imposed load values and those of the ASCE and Eurocode respectively did not match, no clear preference for a reference code could be determined. This is confirmed by the statistics of load ratios listed in Table 5. Compatibility with either ASCE or Eurocode would require an increase for seven occupancy classes each, and ten increases to satisfy both codes.

### 1.1.5.3 Occupancy Classification System of the Loading Codes

The occupancy classification system of SABS was compared to the reference codes with respect to comprehensiveness, clarity and convenience. The ASCE code has the most comprehensive list of 63 occupancies arranged alphabetically; Eurocode has a sparse set of 21 occupancies, arranged according to intensity values, somewhat similar to the arrangement of the 35 SABS occupancy classes. The layout of SABS 0160 Table 4 can be retained even if values are adjusted and the number of classes is increased.

**1.1.6 Conclusions from the Comparative Study of Imposed Loads**

Minimum imposed load values prescribed by SABS 0160-1989 are lower than those of a representative set of codes or as compared to the two possible reference codes, ASCE-7 and Eurocode. This is the case for a range of general and specialist occupancy classes. The compatibility of the SABS code to at least one of the potential reference codes should be improved. Deviations should be justified sufficiently.

**1.2 Imposed Loads for Inaccessible Roofs**

The SABS provision for imposed loads for inaccessible roofs was found to be substantially non-conservative in comparison with the other codes. This is confirmed through Table 4 in Section 1.1.4, where it is stated that the Eurocode, ASCE and BS codes prescribe more conservative load values. The prescribed minimum value of the SABS is subsequently evaluated in more detail through comparison with that of the other codes. Figure 1 shows the comparison of the magnitude of the imposed load for inaccessible roofs prescribed by the various loading codes over the range of tributary areas.

The imposed roof load is dependent on the tributary area in the following way: A uniformly distributed load applied over a large area is more conservative than applying the same load over a smaller area. The reason being that the ratio of expected-total-maximum-load to area-size decreases as the area increases. Thus, one would expect that the prescribed loads of the various codes decrease as the area increases in order to maintain a constant level of reliability over the range of tributary areas.

The imposed roof load is also dependent on the roof slope through the following principle: As the roof slope increases, the roof becomes less "accessible" and for a certain maximum roof slope the roof would be totally inaccessible (for all practical purposes) in terms of people or materials on the roof. This is reflected through certain loading codes prescribing an imposed roof load value of zero for roof slopes larger than a certain value (45° for the BS code). Therefore smaller roof loads would pertain to steeper roof slopes.

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The Eurocode and BS code do not take into account the dependency on the tributary area whilst the SABS and AS codes do not account for the dependency on the roof slope. The ASCE code is the only code that incorporates both the tributary area and the roof slope.
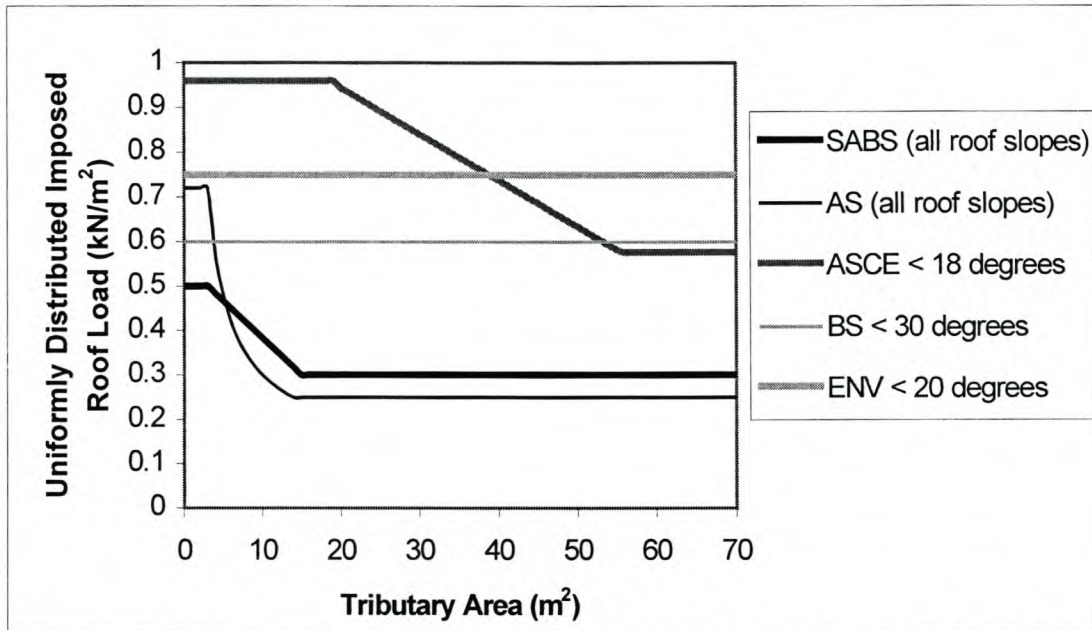


**Figure 1. Imposed Roof Loads dependent on Tributary Area and Roof Slope**

Only the load values for the lower bounds of roof slopes prescribed by the ASCE, BS and ENV codes are included in the comparison shown in Figure 1. The reason is that the lower roof slopes pertain to the larger prescribed load values and it is the larger values which are relevant when evaluating the SABS load value. The smallest roof slope used to define a category is that of the ASCE, which is equal to 18°. By far the majority of roof slopes encountered in South Africa is smaller than 18°, particularly for light industrial steel buildings, and therefore the comparison in Figure 1 is valid and representative.

The following conclusions are made from the comparison of the magnitude of the imposed roof load shown in Figure 1:

- For large tributary roof areas (in excess of 50m²) the SABS is substantially less conservative than the Eurocode, ASCE and BS codes, and only marginally more conservative than the AS code. The structural members supporting roof areas in excess of 50m² would typically be the frames of low-rise industrial steel buildings, which would also normally have inaccessible roofs. For these members the

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SABS prescribed imposed load value is factor 2 smaller than those of the ASCE and BS codes and factor 2.5 smaller than that of the Eurocode.

- For smaller tributary roof areas (areas smaller than 15m$^2$) the SABS is substantially less conservative than the ASCE code (factor 2), and to a lesser extent (but still significantly) non-conservative in comparison with the Eurocode and BS code. Roof areas smaller than 15m$^2$ would typically apply to the purlins of low-rise industrial steel buildings. For roof areas smaller than 5m$^2$, the AS code also prescribes a larger imposed roof load, although members supporting such small roof areas are not very common.

The larger load values of the Eurocode, ASCE and BS codes cannot be attributed to snow load provisions since the snow load is provided for separately in these codes. It could be argued that other loading codes do not place particular attention on the imposed load for inaccessible roofs, due to the imposed load combination rarely being the critical load combination in these countries, with the additional dead loads (in the form of insulation materials) and the snow load playing the governing role. The other codes may therefore just as well prescribe a higher imposed roof load value due to the economical implications of this being suppressed by the higher dead and snow loads. However, harmonisation of loading codes to an international reference code is necessary, and the fact remains that the magnitude of the SABS prescribed imposed roof load is *disconcertingly less conservative* than particularly those of the Eurocode and ASCE code which are the two main contenders for an international reference code.

## 1.3 Motivation for the Investigation

It would not suffice to merely increase the magnitude of the SABS imposed roof load to be on par with those of the other codes without any rational basis for doing this. It is also argued that due to the nature of the loads for inaccessible roofs, survey data or other scientific information on them would not be readily obtainable. A comprehensive literature investigation was performed in search of information on imposed roof loads or any load survey data on the subject but none was found. This is in contrast to imposed floor loads where load models are available to aid in the adjudication process as is seen in Table 6, Section 1.1.5.

An investigation into the imposed load for inaccessible roofs is therefore required in order to establish a scientific rationale through which the codified design values may be measured effectively. Due to the lack of information and the large uncertainties involved in the imposed roof load, stochastic treatment of them is required. Conceptually, probabilistic modelling can be used to derive values to satisfy prescribed levels of reliability and to evaluate existing levels of reliability provided for by the SABS code. Through this method it can be confirmed whether the SABS code provisions for imposed loads for inaccessible roofs are satisfactory in reliability terms.

The type of building selected to serve as the basis for the investigation is a low-rise industrial steel building (alternatively known as a light industrial steel building). The light industrial steel building is considered to be the most common case of where inaccessible roofs apply and is subsequently regarded as sufficiently representative to be used in the investigation.

## 1.4 Objective of the Investigation

The subject of this report is defined as an investigation into imposed loads for inaccessible roofs of light industrial steel buildings with the purpose of providing answers to the following questions:

- How significant is the imposed roof load in the design of industrial buildings and in what respect or failure modes does it play a governing role?

- What are the actual mechanisms involved in producing the imposed roof load and in what way can it be modelled in order to realistically resemble imposed roof loads in practice?

- How do the prescribed load models found in design- and probabilistic model codes compare with the actual behaviour of the imposed load for inaccessible roofs found in practice and in which areas do these models under-perform when describing the imposed roof load?

- Does the prescribed imposed roof load value of the SABS 0160-1989 in combination with codified load and resistance factors achieve the desired level of reliability for light industrial steel buildings?

## 1.5 Scope of the Investigation

The scope of the work involved in providing answers to the questions raised in the previous section is set forth as follows:

- A sensitivity study is to be conducted in order to determine the significance of imposed roof loads in the design of light industrial steel buildings. This involves identification of failure mechanisms where imposed roof loads have an influence, measurement of the extent of the imposed roof load's effect and characterising the type of buildings for which imposed roof loads plays a governing role in the design.

- The next step to be implemented is the gathering of information on the imposed roof load. The nature of the information is the type of mechanisms involved in producing the loads, the way in which these mechanisms translate into roof loads, and the uncertainty associated with the magnitude and application of them. The information is to be obtained through expert measurement, that is a survey amongst selected experts with relevant expertise in fields applicable to this investigation. The source of the information therefore lies in the experience of the experts taking part in the survey and will be expressed through expert opinion. Expert opinion as a resource for information is not readily accessible in terms of yielding scientifically defendable results. It is clear that an unmethodological use of expert opinion will not contribute to rational decision making. Through the course of this report the motivation for using expert opinion as the source of information will be justified. In short the reason is that, owing to the nature of the load mechanism translating into the imposed roof load, there is no other alternative but to draw on the knowledge of experts. Evidently it is imperative that such experts be selected scrupulously, with carefully consideration of their fields of expertise, and that their opinions be obtained and managed in a scientific manner that yields rationally defendable results.

- Following from the gathering of information, the imposed roof load is to be modelled probabilistically, which in essence entails quantifying the uncertainty associated with the imposed roof load. This involves processing of the data obtained from the expert survey and establishing of probabilistic models that will realistically simulate the likelihood of occurrence of the range of possible imposed roof load values for the different load mechanisms. Conservative assumptions

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are to be made and the load models will be based on principles that would result in larger rather than smaller load values.

- The load models subsequently obtained are to be compared to existing codified provisions in order to assess the degree of agreement and to identify where existing provisions are non-conservative. The probabilistic models established for the various load mechanisms may be put forward in a probabilistic model code, providing specifically for the said load mechanisms. A degree of generalisation will also be implemented here, i.e. the obtained load models are to be consolidated and rationalised so as to represent imposed roof loads in general and not only cater for the specific load mechanisms.

- The final step is to perform a reliability analysis on the failure mechanisms identified in the sensitivity study. Current SABS load and resistance factors, together with the computed distribution of the imposed roof load and existing dead load and resistance models are to be utilised in this process. The results from the reliability analysis will be used to evaluate the performance of the SABS 0160-1989 in providing for the imposed load of inaccessible roofs.

## CHAPTER 2: SENSITIVITY STUDY TO DETERMINE THE IMPORTANCE OF THE IMPOSED ROOF LOAD

A sensitivity study is conducted to determine the extent to which the imposed roof load has an influence on the design of the main structural members of light industrial steel buildings. If it is concluded with some degree of certainty that the imposed roof load does not determine the sizes of the main members in most cases, then it could be argued that this investigation is not warranted. Therefore the sensitivity study is a vital part of the thesis as it determines whether or not to proceed with the investigation.

The predominant load mechanisms for light industrial steel buildings involved in determining the sizes of the structural members are the deadload, imposed load and the wind load. Since the dead load is permanently present on the structure, that leaves the imposed load and wind load as the two main contenders. So, for any structural member, either the imposed load or the wind load is involved in determining its size. The aforementioned reasoning is subjected to the codified provision for combination of loads. The SABS 0160-1989 prescribes the following load combinations for ultimate limit states design:

$$Q_n = 1.5D_n \tag{1a}$$
$$Q_n = 1.2D_n + 1.6L_n \tag{1b}$$
$$Q_n = 0.9D_n + 1.3W_n \tag{1c}$$
$$Q_n = 1.2D_n + 1.3W_n \tag{1d}$$

where $Q_n$ = the total nominal load in
$D_n$ = the nominal dead load in
$L_n$ = the nominal imposed load
$W_n$ = the nominal wind load in

Comparing the two gravitational load combinations, that is Combinations (1a) and (1b), it is found that Combination (1a) will only become dominant for $D_n$ > 1.6 kN/m² when $L_n$ = 0.3 kN/m² (as is prescribed by the SABS for large tributary roof areas). Dead loads in excess of 1.6 kN/m² very seldom apply to light industrial steel buildings and it is therefore concluded that Combination (1b) will always be dominant for such buildings. Combination (1a) plays a role in the design of buildings with a more significant self-weight such as concrete structures and is subsequently discarded for the purpose of this investigation.

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For the purpose of this sensitivity study, load combination (1d) is discarded on the basis that the wind load and the dead load on the roof will in most cases be in opposite directions. Combination (1c), having the smaller dead load partial factor, would therefore result in larger uplift forces than Combination (1d). Therefore, Combination (1c) would produce larger positive moments at the eaves of the windward columns and larger negative moments on the roof element. Combination (1c) is subsequently regarded is sufficiently representative of the wind load effects.

So, the two load combinations emerging as representative of the gravitational and wind load effects are Combinations (1b) and (1c) respectively. They will subsequently be referred to as the gravitational (D+L) and wind load (D+W) combinations. The gravitational and wind load combinations are to be compared in terms of the magnitudes of the load effects which they induce under certain circumstances. The term "circumstances" includes the building geometry, relative stiffnesses of members, and the location of the building terrain as this determines the extent to which it is subjected to wind forces. The philosophy in this approach is as follows: If one can conclude that, under circumstances commonly found in practice, the gravitational load combination plays a significant role in the design, then the investigation into imposed roof loads is warranted. Through this process the types of light industrial steel buildings which are inclined to be governed by the gravitational load combination will also be characterised.

## 2.1 Circumstance Parameters

A generic example of a typical light industrial steel building as used for the analysis, is shown in Figure 2. The load effects being considered are those most critical in the design process. They are:

- The positive and negative moments in the column at the eaves.
- The positive and negative moments in the roof element at the ridge of the roof.
- The negative moments in the purlins at midspan and at the supports due to the uplift and gravitational loads respectively. The extent to which the gravitational load combination determines the size of the purlins is discussed in Section 2.2.5.

Although the maximum moment on the roof is located at a slight offset from the ridge of the roof, using the moment at the ridge will not detriment this comparative study in any significant way.

For comparison of the gravitational and wind load combinations the effect of axial forces in the columns is disregarded. Due to the relatively small weight of the structure and the small imposed loads it is assumed that axial forces do not have a significant influence, and bending is considered to be the primary load effect that determines the sizes of the columns. The gravitational load combination subjects the columns to axial compression and bending, which renders the columns to being designed as beam-columns. The wind load combination will predominantly place the columns in axial tension and bending. For the same bending moment, a column under axial compression is more critical than one in tension, and therefore by not considering axial forces in the columns one would underestimate the degree to which the gravitational load combination governs in design by a small margin.
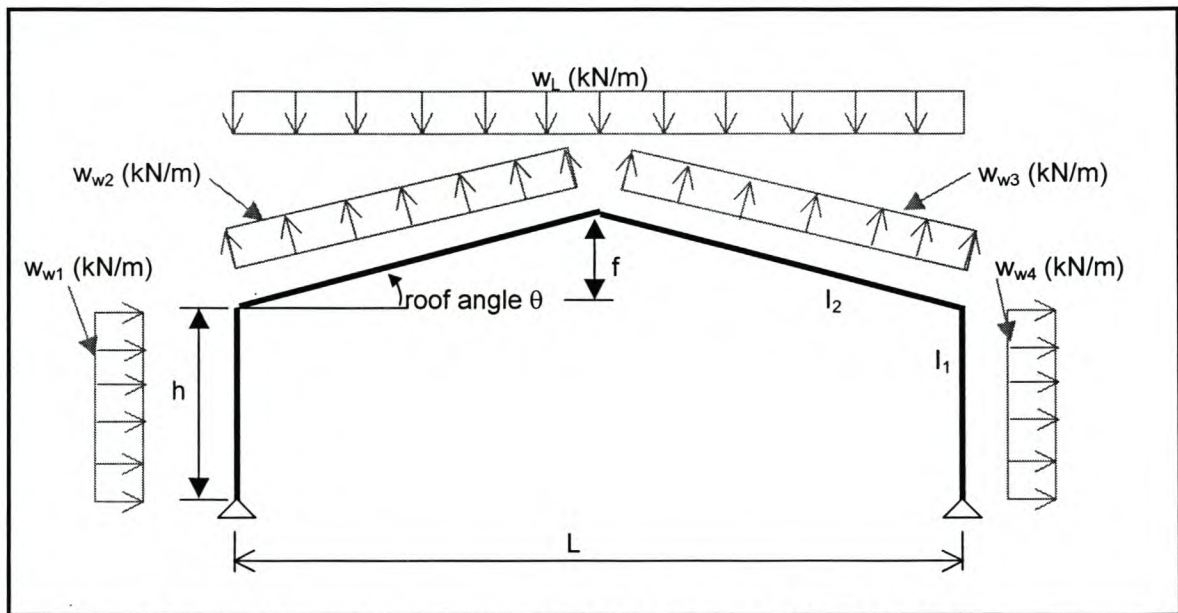


**Figure 2. Generic Example of a Light Industrial Steel Building**

Evidently from Figure 2, no rotational restraint is provided at the foundations. It is accepted that this is generally the most common modelling technique and results in a cost-effective way of construction.

Under the gravitational loading $Q_n = 1.2D_n + 1.6L_n$, the magnitudes of the load effects are defined through Equations (2a) & (2b). The rigid frame formulae used in Equations (2a) & (2b) are obtained from the STEEL DESIGNERS MANUAL.

$$M_{eaves} = \frac{-w_L L^2 (3 + 5m)}{16N} \tag{2a}$$

$$M_{roof} = \frac{w_L L^2}{8} + mM_{eaves} \tag{2b}$$

where
$M_{eaves}$ = the moment at column eaves

$M_{roof}$ = the moment at the ridge of the roof

$w_L, L, h, \theta, I_2, I_1$ = as defined in Figure 2

$N$ = $2(k+1) + m + m(1 + 2m)$

$k$ = $\dfrac{2I_2 h \cos\theta}{I_1 L}$

$m$ = $1 + \phi$

$\phi$ = $\dfrac{L \tan\theta}{2h}$

As is evident from Equations (2a) & (2b), the load effects are dependent on a number of variables, the so-called *circumstance parameters*. Under the gravitational loading the circumstance parameters are:

- The height h of the building.
- The span L of the building.
- The roof angle $\theta$.
- The magnitudes of the nominal dead and imposed loads $D_n$ and $L_n$.
- The stiffness of the roof element to stiffness of the column ratio $I_2/I_1$.

The circumstance parameters shown above are the least number of variables to which Equations (2a) & (2b) can be reduced.

Under the wind load combination $Q_n = 0.9D_n + 1.3W_n$ the magnitudes of the load effects are defined through Equations (3a-c). The rigid frame formulae used in Equations (3a-c) are obtained from the STEEL DESIGNERS MANUAL.

$$M_{eaves+} = \frac{1}{8N}\left[2\left(w_{w4} - w_{w1}\right)h^2\left(B+C\right) + k + \frac{\left(\left(w_{w2} + w_{w3}\right) - w_L\right)L^2\left(3 + 5m\right)}{4}\right]$$
$$- \frac{\left(w_{w2} - w_{w3}\right)Lh\tan\theta}{4} + \frac{w_{w1}h^2}{2} \tag{3a}$$

$$M_{eaves-} = \frac{1}{8N}\left[2\left(w_{w4} - w_{w1}\right)h^2\left(B+C\right) + k + \frac{\left(\left(w_{w2} + w_{w3}\right) - w_L\right)L^2\left(3 + 5m\right)}{4}\right]$$
$$- \frac{\left(w_{w2} - w_{w3}\right)Lh\tan\theta}{4} - \frac{w_{w4}h^2}{2} \tag{3b}$$

$$M_{roof} = -\frac{V_A L}{2} + \frac{2M_{eaves+} - w_{w1}h^2}{h}\left(h + \frac{L\tan\theta}{2}\right) - w_{w1}h\left(\frac{h + L\tan\theta}{2}\right)$$
$$+ \frac{w_{w2}L^2}{8\cos^2\theta} - \frac{w_L L^2}{8} \tag{3c}$$

where  $M_{eaves+}$      = the moment at the eaves of the windward column

      $M_{eaves-}$      = the moment at the eaves of the leeward column

      $M_{roof}$      = the moment at the ridge of the roof

      $w_{w1}, w_{w2}, w_{w3}, w_{w4}$      = as defined in Figure 2

      $L, h, \theta, l_2, l_1$      = as defined in Figure 2

      $k, \phi, m, N$      = as defined for Equations (2a) & (2b)

      $V_A$      =

$$\frac{\dfrac{L\left(3w_{w2} + w_{w3}\right)}{8} - \dfrac{w_L L}{2} + \dfrac{\left(w_{w1} + w_{w4}\right)h^2}{2L}}{\phantom{x}}$$
$$- \frac{\left(w_{w2} - w_{w3}\right)Lh\tan\theta\left(1 + m\right)}{4L}$$

      B      $= 2\left(k+1\right) + m$

      C      $= 1 + 2m$

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Under the wind load combination the circumstance parameters are:

- The height h of the building.
- The span L of the building.
- The roof angle $\theta$.
- The magnitude of $D_n$.
- The stiffness of roof beam to stiffness of column ratio $I_2/I_1$.
- The quantity of $W_n$ which is in turn dependent on the following variables:
  - Basic regional wind velocity. Refer SABS 0160-1989 Clause 5.5.2.2.
  - The class of the structure. Refer SABS 0160-1989 Clause 5.5.2.6.
  - The terrain category. Refer SABS 0160-1989 Clause 5.5.2.4.
  - The height of the building site above sea level. Refer SABS 0160-1989 Clause 5.5.3.1.
  - The external and internal pressure coefficient $C_{pe}$ and $C_{pi}$ for the walls and roof. Refer to SABS 0160-1989 Clause 5.5.1.

For the purpose of this study, the internal pressure coefficients are ignored. The SABS 0160-1989 prescribes internal pressure coefficients $C_{pi}$'s ranging from 0 to +0.8, depending on the area-ratio of the openings in the windward wall to that of the leeward wall. Selection of a representative $C_{pi}$ - value, would make the building very specific, and it is assumed that the most representative situation occurs when the four walls are more or less equally permeable and $C_{pi} = 0$. Note that the inclusion of positive $C_{pi}$ - values would increase the extent to which the roof beam is determined by the uplift forces of the wind load combination.

The gravitational and wind load combinations are now compared to determine which one produces the largest load effects for different building geometry's, different relative stiffnesses and different dead loads, imposed loads and wind loads. This is done through a parametric study where one parameter is varied and the others kept constant. The interval of the parameter (that is being varied) is subsequently recorded where the one load combination dominates the other in terms of the said load effects.

Since there are many variables this parametric study is rather iterative and therefore the use of a spreadsheet programme is beneficial in computing the load effects for each circumstance. A spreadsheet programme SENSTUDY was developed to measure the influence of the different circumstance parameters on the load effects

under consideration. To verify whether the modelling process was done correctly, certain cases were checked using the computer programme PROKON STRUCTURAL ANALYSES. SENSTUDY is presented in Appendix A and a verification of the model through comparison with the PROKON analysis is shown in Appendix B.

The study showed that certain circumstance parameters did not have a significant influence on the range of buildings where the gravitational load governs. The reason being either that the range of the given parameter as applicable to light industrial buildings is not wide enough to be influential, or that the formulae used (Equations (2a) through (3c)) are simply insensitive to variations in these parameters. An example of the latter is an increase in say the roof angle would alter the negative moment at the column eaves under the gravitational load condition with the same amount as it would alter the negative moment under the wind load condition.

The following circumstance parameters are identified as non-influential within their respective ranges:

- The roof angle $\theta$. The comparison of the two load combinations is insensitive for variations between 3 and 15°, which is the normal range of roof angles for light industrial buildings. The representative roof angle is taken as 10°.

- Basic regional wind velocity V. V as proposed by SABS 0160-1989: Clause 5.5.2.2 is 40m/s for 80% of the area of South Africa, and therefore other wind velocities are discarded.

- The class of the structure. Referring to SABS 0160-1989: Table 5, there is no significant change in the wind speed multiplier $k_z$ depending on the class of the structure for building heights up to 15m, which can be considered the upper limit of building heights for light industrial steel buildings. The building is taken as class B.

- The altitude of the building above sea level. Referring to SABS 0160-1989: Clause 5.5.3.1, $k_p$ chosen as 0.53 at 1000m above sea level is representative.

- The external pressure coefficient $C_{pe}$ for the walls and roof. Referring to SABS 0160-1989 Tables 6, $C_{pe}$ for the walls remains constant at +0.7 for the windward

side of the building. $C_{pe}$ for the leeward wall varies from -0.2 to -0.25 and is subsequently taken as -0.225. Referring to SABS 0160-1989 Table 7, the $C_{pe}$ - values for the roof are taken at a roof angle of 10° and are equal to -1.2 and -0.4 for the windward and leeward sides of the roof respectively. In applying Tables 6 & 7 of the SABS 0160-1989 it is assumed that h/L is smaller than 0.5, which is the most common range of h/L-ratios for light industrial steel buildings.

The basis on which the moments at the column eaves $M_{eaves}$ and on the roof $M_{roof}$ are compared for the two load combinations is in terms of $M/L^2$ – that is the ratio of the moment to the square of the building span L. The reason being that $M/L^2$ is the simplest common term to which both the formulae for the moments under gravitational and wind loading can be reduced to. This is done by substituting h = (h/L) × L in Equations (2a)-(3c). Therefore, the circumstance parameters h and L are consolidated into a single parameter h/L, defining a specific shape of the building. By inspection it can be seen that $L^2$ is now a common factor for all of Equations (2a)-(3c) when h/L is introduced in this manner. Refer to SENSTUDY in Appendix A for a presentation of the formulae used to compare the moments. The formulae found in SENSTUDY are derived from Equations (2a) - (3c) through the introduction of h/L.

It is found that the comparison is most sensitive to the value of h/L, with $M_{eaves}$ and $M_{roof}$ being dominated by the gravitational load combination for buildings with small h/L-ratios. Figures 3 & 4 illustrate how the different circumstance parameters influence the comparison of $M_{eaves}$ resulting from the two load combinations.

As is evident from Figures 3 & 4, the wind load combination dominates for larger values of h/L. Note that for Terrain Category 3 (Figure 4), the range of h/L-ratios where the gravitational load combination dominates the wind load combination in terms of the largest absolute value of $M_{eaves}$ is much larger than for Terrain Category 2 (Figure 3). The h/L-ratio where the wind load combination starts to dominate the gravitational load combination in terms of the given load effect is subsequently referred to as the *transition ratio*, and is shown in Figures 3 & 4 where the curves intersect.
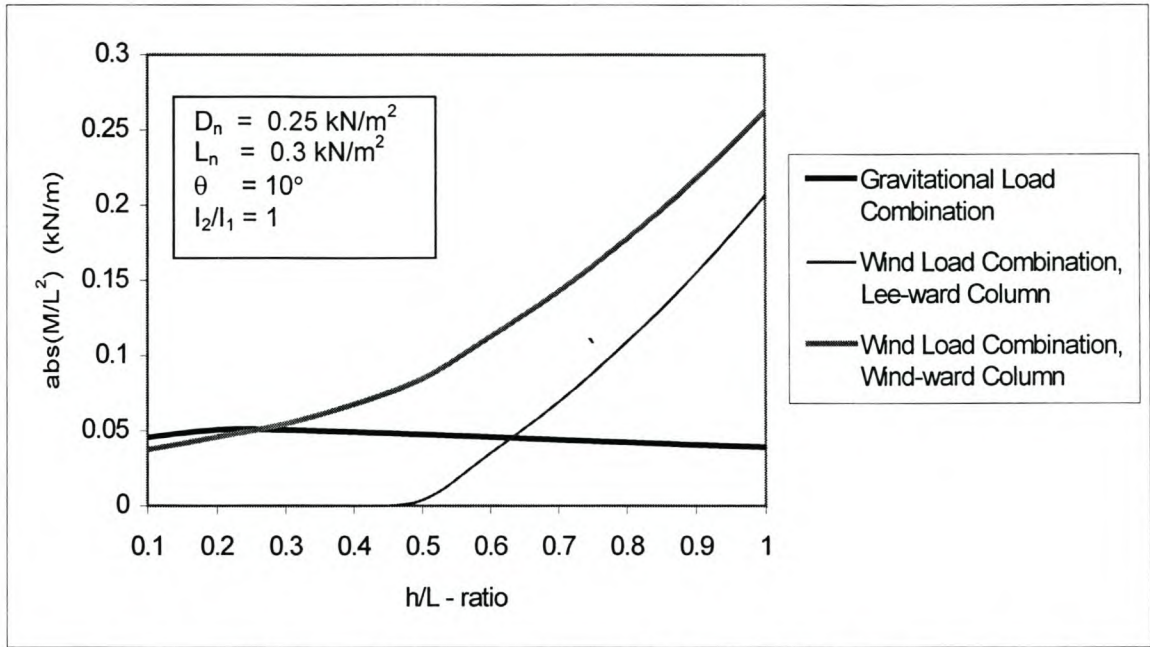
**Figure 3. Comparison of the Moment at Column Eaves induced by the Gravitational and Wind Load Combination for a Building in Terrain Category 2**
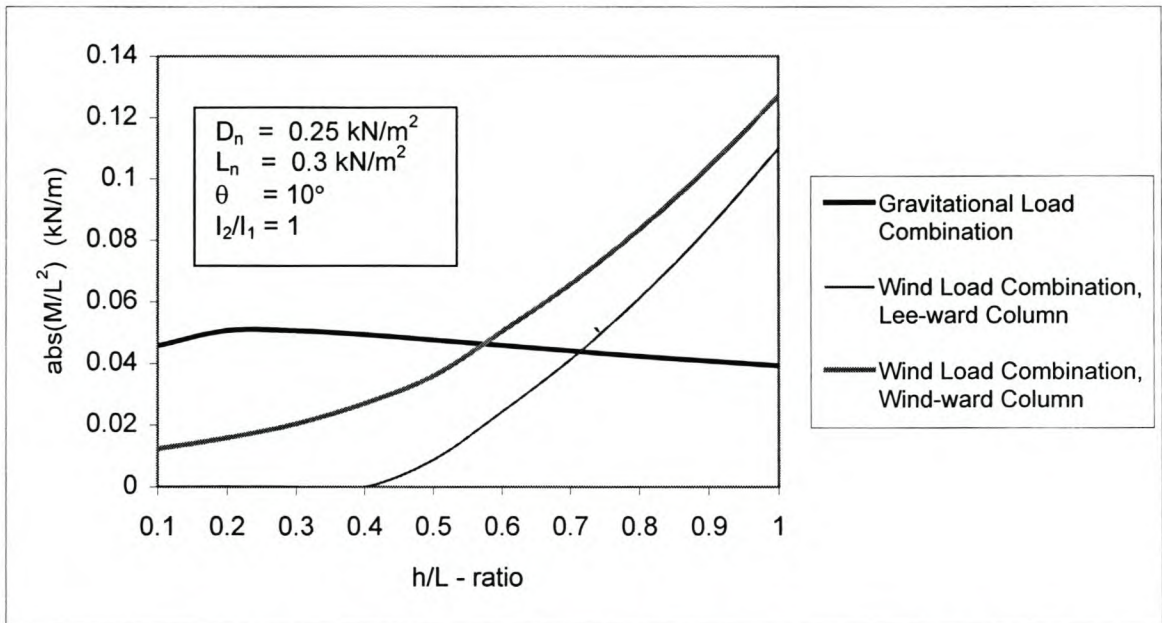


**Figure 4. Comparison of the Moment at Column Eaves induced by the Gravitational and Wind Load Combination for a Building in Terrain Category 3**

Transition ratios may be calculated for different values of the circumstance parameters. Those parameters that do not influence the value of the transition ratio significantly have been identified as non-influential. Table 7 shows the transition ratios obtained for different combinations of circumstance parameters.

**Table 7. Height to Span ratios h/L below which the Gravitational Load Combination dominates in determining the Moment at Column Eaves**

| $D_n$ (kN/m$^2$) | θ (degrees) | $l_2/l_1$ | Terrain Category | h/L-ratio | |
|---|---|---|---|---|---|
| | | | | Leeward Column | Windward Column |
| 0.15 | 3 | 1 | 2 | 0.58 | 0.05 |
| 0.35 | 3 | 1 | 2 | 0.60 | 0.35 |
| 0.35 | 15 | 1 | 2 | 0.67 | 0.39 |
| 0.35 | 15 | 20 | 2 | 0.41 | 0.33 |
| 0.35 | 15 | 20 | 3 | 0.45 | 0.46 |

The $l_2/l_1$-ratio of 1 constitutes cases where a roof beam is used, while that of 20 represents roof trusses. The imposed roof load for Table 7 is taken as 0.3 kN/m$^2$ as prescribed by SABS 0160-1989.

It is evident from Table 7 that both the transition ratios for the leeward column as well as the windward column are sensitive to all the circumstance parameters, except for the roof angle θ.

It is of interest to note the sensitivity towards the Terrain Category of the building. As is evident from Figures 3 & 4, changing the Terrain Category from 2 to 3 and keeping all other parameters constant, results in the transition ratio for the largest moment at column eaves shifting from h/L = 0.25 to h/L = 0.55. The disconcerting aspect of this is that the onus rests on the designer to classify the building as either situated in Terrain Category 2 or 3. The description provided in the SABS 0160-1989: Clause 5.5.2.4 for Terrain Categories 2 & 3, lends itself to subjective interpretation in that different designers may classify the same building into different Terrain Categories which will have a significant effect on the sizes of the main members.

**2.2 Characterising the Type of Building for which Imposed Roof Loads Govern**

One now wants to determine for what type of industrial steel building the imposed roof load is involved in the design of the main structural members. The type of building is defined through the circumstance parameters (Section 2.1). It is important to recognise that the combination of circumstance parameters defining a particular building is not totally ad hoc and that there certainly exists a degree of inter-dependency between the circumstance parameters. For example, buildings with small h/L-ratios would normally have longer spans and therefore roof trusses instead of roof beams would pertain. Also, larger dead loads would apply to buildings with

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longer spans due to the self-weight of the roof members increasing to accommodate the larger Z-modulus necessary to withstand the increased moment. Of course, the one circumstance parameter that is totally independent of the others is the Terrain Category of the building.

By far the majority of industrial buildings are classified as either being in Terrain Categories 2 or 3 (refer to the definitions given in SABS 0160-1989 Clause 5.5.2.4); therefore only considering these two categories is regarded as representative

The inter-dependency of the circumstance parameters is subsequently evaluated in terms of long-span and short-span buildings.

## 2.2.1 The Merging of Circumstance Parameters into Long-Span Buildings

Long-span buildings are classified as light industrial steel buildings with spans in excess of 20m. The type of roof element relevant to this range of buildings is the roof truss and therefore the higher stiffness ratios would apply. Each of the influential circumstance parameters is now bounded in terms of the range of values that they may assume for long-span buildings.

### The Stiffness Ratio $I_2/I_1$

Consider a typical parallel chord lattice girder structure with a small truss depth of say 1m, spanning 20m and connected to a relatively stiff column, say a *356x171x45 I section*. The top and bottom chord for this truss comprises of *90x90x8 angle sections* with a sectional area of 1389mm$^2$ each, which equates to a stiffness for the roof truss of $I_2$ = 1.4E09mm$^4$. The stiffness of the column section is $I_1$ = 121E06mm$^4$, and therefore $I_2/I_1$ = 11.5. Thus, an $I_2/I_1$-ratio of 11.5 can be considered to be low for roof truss type buildings since the truss depth would more often than not be larger than 1m, which would result in higher $I_2/I_1$-ratios. The upper bound is determined in a similar way by considering a relative slender column connected to a roof truss with a depth of 2m. The resulting $I_2/I_1$-ratio for this case is 25.

Therefore, the range of stiffness ratios applicable buildings with spans in excess of 20m would be $11.5 \leq I_2/I_1 \leq 25$. Therefore, $I_2/I_1$ = 18 (midway between 11.5 and 25) is considered to be a representative stiffness ratio for long-span buildings.

*The Dead Load D*

It is assumed that the dead load on the roof including the weight of the main structural members, the roof cladding and insulation, and services suspended from the roof would be in excess of 0.35 kN/m$^2$ (that is 35 kg per m$^2$ floor area) for 20m plus spanning buildings. Thus $\underline{D \geq 0.35 \text{ kN/m}^2}$ for long-span buildings.

*The height to span ratio h/L*

Since the gravitational load combination dominates for smaller h/L-ratios, as is evident from Figures 3 & 4, only the lower bound for the range of h/L-ratios is of interest. If the eaves height is constricted to a minimum of 4m for long-span buildings then it is found that for a 20m-spanning the h/L-ratio = 0.2. A minimum eaves height of 4m is selected for long span buildings for the reason that heights smaller than this may render long-span buildings to being disproportionate. As it is expected that 4m eaves heights would also apply to buildings with spans in excess of 20m, it is safe to assume a lower bound for the h/L-ratio of 0.2. Thus, $\underline{h/L \geq 0.2}$ for long-span buildings.

### 2.2.2 The Merging of Circumstance Parameters into Short-Span Buildings

Short-span buildings are classified as light industrial steel buildings with spans less than 15m. The type of roof element relevant to this range of buildings is the roof beam and therefore the lower stiffness ratios would apply. Each of the influential circumstance parameters is now bounded in terms of the range of values that they may assume for long-span buildings.

*The Stiffness Ratio $I_2/I_1$*

For short-span buildings where the roof element would predominantly be a roof beam the representative stiffness ratio $I_2/I_1$ is bounded between 0.5 and 1.5. $I_2/I_1 = 1$ is considered to be most common since the columns and the roof beam would more often than not be the same size. Therefore, $\underline{0.5 \leq I_2/I_1 \leq 1.5}$ for short-span buildings.

*The Dead Load D*

It is assumed that the dead load on the roof including the weight of the main structural members, the roof cladding and insulation, and services suspended from the roof would be in the order of 0.25 kN/m² (that is 25 kg per m² floor area) for buildings with spans shorter than 15m. Thus $D \approx 0.25$ kN/m² for short-span buildings.

*The Height to Span ratio h/L*

The lower bound for the h/L-ratio is obtained by considering a minimum eaves height of 3m with a span of 15m, which results in h/L = 0.2. Note that this is the same as for long-span buildings. Thus $h/L \geq 0.2$ for short span buildings.

## 2.2.3   Buildings for which the Columns are determined by the Gravitational Load Combination

The limits placed on the circumstance parameters in the previous sections for long-span and short-span buildings are implemented in establishing the extent to which the gravitational load combination determines the sizes of the columns.

For a *long-span building*, with $l_2/l_1$ = 18, D = 0.35 kN/m² (see section 2.2.1) and $\theta$ = 10°, the h/L-ratios below which the gravitational load combination governs in terms of the largest moment at column eaves have been determined as h/L = 0.27 and 0.42 for Terrain Categories 2 & 3 respectively. In other words, for buildings with h/L-ratios of less than these values the gravitational load combination determines the size of the columns.   From Section 2.2.1 it is seen that this range of h/L-ratios are considered to be commonly found in practice.

Considering Table 7, it is observed that for *short-span buildings* with roof beams where $l_2/l_1$ = 1, a higher transition ratio applies than for roof trusses, meaning that the range of buildings where the gravitational load determines the column sizes is larger than for roof trusses. It is observed from Figure 4 that for Terrain Category 3 and h/L-ratios of up to 0.55 the gravitational load governs. This constitutes a wide range of short-span buildings commonly found in practice.

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In conclusion, for buildings with longer spans and relatively low eaves heights of 4-6m, as well as shorter span buildings situated in Terrain Category 3 it can be assumed that the size of the columns will generally be determined by the gravitational load combination.

It must be stressed that the largest moment induced by the wind load combination is the positive moment at the windward side of the building. Thus, in comparing this to the negative moment resulting from the gravitational load combination, it has to be assumed that the unbraced lengths of the compression flanges (inner and outer) are the same for both, i.e. knee-braces are provided for the inner flanges of the columns.

## 2.2.4 Buildings for which the Roof Elements are determined by the Gravitational Load Combination

For *long-span buildings* with roof trusses, the size of the top chord will invariably be determined by the gravitational load combination since this load condition puts the top chord under compression and compressive buckling is the governing failure mechanism. The wind load combination induces compression of the bottom chord and therefore determines its size.

For *short-span buildings* with roof beams it is assumed that the size of the beam is determined by the maximum moment near the ridge of the roof. The reason is that the roof beam is haunched at the eaves-connection, which provides adequate resistance for the moment at that position. This approach is economical, as the eaves moment reduces to zero over a relatively short distance. In contrast, the internal span moment occurs over the major portion of the roof beam, which negates the possibility of using a haunch to withstand such moment. This implies that the beam section has to withstand the internal span moment on its own. The most economical approach is to design the beam section for the larger of the negative or positive moments near the roof-ridge. If the positive moment is larger in magnitude than the negative moment, then knee-braces to the bottom flange are to be provided at such intervals as required for the chosen beam section for lateral support (the top flange being laterally supported by the purlins). By applying this approach, the size of the roof beam is determined by the larger of the positive or negative moments (larger in magnitude) near the ridge of the roof (the positive moment results from the

gravitational load combination and the negative moment from the wind load combination).

For *short-span buildings* with roof beams situated in Terrain Category 2 it is found that a transition ratio of h/L = 0.7 applies for a dead load of 0.25 kN/m$^2$ (see Appendix A). This means that for h/L-ratios less than 0.7 the gravitational load combination produces the larger roof ridge moment. By far the majority of short-span buildings have h/L-ratios of less than 0.7. This range of h/L-ratios increases even more for buildings situated in Terrain Category 3.

Therefore, it is concluded that the imposed roof load plays an even more significant role in the design of the roof element than for the design of the columns. Again, the lower the h/L-ratio of the building, the more pronounced is this dominance.

### 2.2.5 Extent to which the Purlins are determined by the Gravitational Load Combination

The purlins constitute a significant portion of the weight of the building and therefore also a significant portion of the cost. A double span purlin with 5m spans is considered. The critical moments are those that place the bottom flange, which is not laterally supported by the roof cladding, under compression. For the gravitational load this is the negative moment at the support and for the wind load it is the negative moment at 1.875m from the outer support. If one assumes that lateral support to the bottom flange is provided at midspan by sag bars/angles, then the unbraced lengths for the compression flanges for the support moment and the midspan moment can be considered to be more or less the same. As a simplification one can subsequently assume that the purlin size will be determined by the larger of the two moments.

Substituting $W_n$ in Equation (1c) with 0.848 kN/m$^2$ and 0.428 kN/m$^2$ for Terrain Categories 2 & 3 respectively, yields

$$Q_{n,cat\ 2} = -0.9D_n + 1.1 \quad (kN/m^2) \qquad (4a)$$
$$Q_{n,cat\ 3} = -0.9D_n + 0.56 \quad (kN/m^2) \qquad (4b)$$

Equations (4a) and (4b) represent the uplift forces due to the wind for Terrain Categrories 2 & 3 respectively. The negative moments induced by the gravitational and wind load combinations are given as follows:

$$M_{grav} = -0.625w_L \tag{5a}$$

$$M_{wind} = -0.35w_w \tag{5b}$$

where  $M_{grav}$ = gravitational moment at the internal support

$M_{wind}$ = the wind moment at 1.875m from the outer support

Equations (5a) and (5b) are now equated, with $w_w$ in Equation (5b) being replaced with Equations (4a) and (4b); and $w_L$ in Equation (5a) being replaced with $1.2D_n + 0.3$ (kN/m$^2$).  Both cases are solved for $D_n$ and it is found that for $D_n \geq 0.19$ kN/m$^2$ and $D_n \geq 0.01$ kN/m$^2$ the gravitational load combination produces the larger negative moment for Terrain Categories 2 & 3 respectively.

The dead load for the purlins (including self-weight) will very seldom be larger than 0.19 kN/m$^2$, and will always be greater than 0.01 kN/m$^2$.  So, through this rather crude simplification, it may be concluded that the purlin size will mostly be determined by the gravitational load combination for buildings situated in Terrain Category 3, which constitutes a wide range of light industrial steel buildings.  Again, note the momentous effect of the Terrain Category of the building.

## 2.3 Conclusion from the Sensitivity Study

It is concluded that the imposed roof load plays a *significant role* in the design of light industrial steel industrial buildings.  Thus, it is worth investigating the mechanisms of the imposed roof load and modelling its occurrence as it is of vital importance in reliability assessment of the structure.

## CHAPTER 3: EXPERT SURVEY ON IMPOSED LOADS FOR INACCESSIBLE ROOFS

The main purpose of this investigation is to determine a probabilistic model that will realistically simulate the likelihood of occurrence over the range of possible imposed roof load values. Through establishing a probabilistic model for the imposed roof load, a *design value* can be determined which will ensure that prescribed levels of reliability are satisfied, taking into account the uncertainty in the imposed roof load. To that end it is necessary to obtain information on the imposed roof load. The methodology to be implemented in gathering the information is dictated by the nature of the mechanisms that translate into the imposed roof load. Therefore, the question that needs to be answered is that of what is to be measured in order to provide the necessary quantitative data. This can only be answered by identifying and evaluating the load mechanisms.

### 3.1 Identifying the Load Mechanisms translating into the Imposed Roof Load

The SABS 0160-1989 does not provide a clear indication of the load mechanisms for which provision is made through its prescribed imposed roof load value. Some guidance is given in a comment to Clause 5.4.4.1, where the following is stated of nominal imposed roof loads: "These are primarily maintenance or construction loads intended to represent the effects of workmen or stacked materials, etc. Alternatively, the distributed load will cater for limited accumulations of snow, hail or rainwater on roofs." This investigation will therefore further the aim by establishing the load mechanisms that translate into the imposed roof load, and by stating a case for which load mechanisms codified provision is justified.

The identification of load mechanisms is done on the basis of consultation with individuals with substantial experience in fields applicable to the imposed load for inaccessible roofs. Such individuals are subsequently referred to as *experts*. The identification is done in two phases:

1.  Preliminary consultation with a small number of experts so as to establish an initial framework of load mechanisms.
2.  Verification of the load mechanisms identified in phase 1 and incorporation of any additional mechanisms through consultation with a wider field of experts.

The experts who were initially consulted (phase 1) are Professor JV Retief and Professor PE Dunaiski of the Department of Civil Engineering, University of Stellenbosch. The experts consulted in phase 2 of the identification are structural engineers, all of whom have five years or more experience (see Section 3.6.2).

Through these two phases, the possible mechanisms involved in generating the imposed roof load have been identified as follows:

- Workers on the roof during construction.
- Workers on the roof during repair, cleaning and maintenance.
- The stacking of roof cladding during construction.
- Machinery and equipment used during construction, repair and maintenance.
- Machinery supported by the roof during the installation of services or for any other purpose during the lifetime of the building.
- Rainwater, hail and snow accumulating on the roof.

For all these mechanisms there exists high spatial and quantitative variability and high uncertainty due to lack of data on them.

## 3.2 Methodology implemented in obtaining Data on the Imposed Roof Load

The load mechanisms as identified are now evaluated in terms of how they can be quantitatively measured. Since it has already been established that no information is available on them in the literature, one would alternatively consider obtaining information through observations, i.e. conducting a survey of some nature for the said load mechanisms. Table 8 shows the reasoning applicable to each load mechanism in terms of its measurability.

Following from the observations made in Table 8, the flow chart in Figure 5 is presented to illustrate the evolution of theory in deciding on the method of data-gathering to be used for this experiment.

**Table 8. Quantitative Measurability of the Load Mechanisms.**

| Load Mechanism | Measurability |
|---|---|
| Construction Workers | Measurable in terms of arbitrary-point-in-time values. This is achievable through recorded observations at construction sites. Such values do not constitute maximum (design) values. |
| Maintenance, Cleaning and Repair Workers | To a certain extent measurable through arbitrary-point-in-time values, although impractical to achieve sufficient number of observations. Maximum (design) values are certainly not readily observable through surveys. |
| Stacking of Roof Cladding | Same as for Construction and Maintenance Workers. |
| Machinery and Equipment during Construction, Repair and Maintenance | Same as for Construction and Maintenance Workers. |
| Machinery for Functional Purposes during Lifetime | Impractical to be measured in any form of load survey. |
| Rainwater, Hail and Snow | Information to be gathered from documented records of weather bureau or alternate source. |
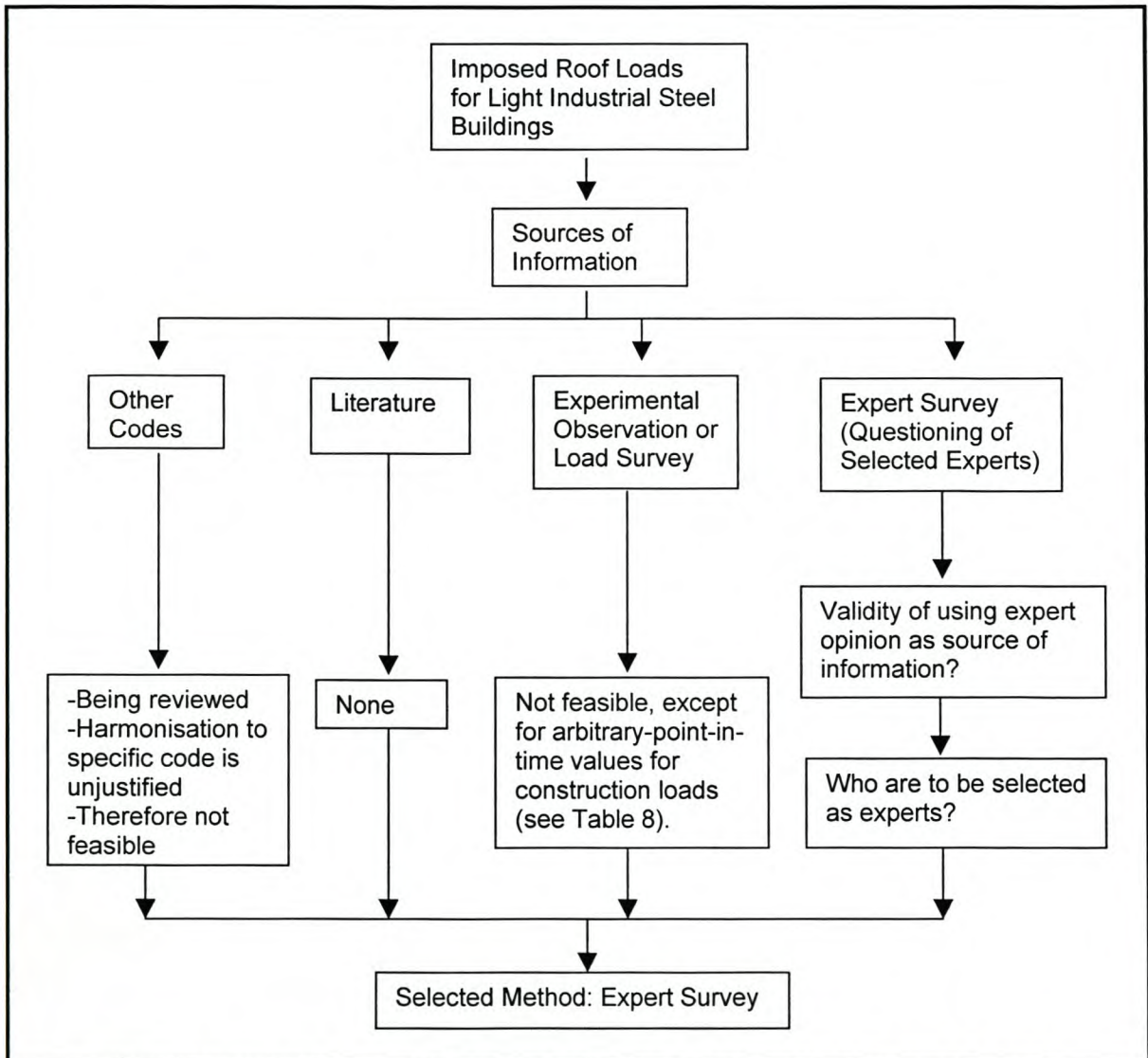


**Figure 5. Selection of the Method of Data-Gathering for the Experiment.**

### 3.2.1 Availability of Information from an Expert Survey

As is evident from Figure 5, the expert survey prevails as the *only* viable option in obtaining quantitative information on the imposed roof load. The main reason for the other sources of information not being appropriate is that the *maximum value* of the imposed roof load (which is of importance to this investigation) is not measurable through any kind of physical load survey, and is only attainable through the knowledge and experience of experts in the appropriate fields. In principle, extreme values can be derived from point-in-time observations, but a *large* database is required, which renders it impractical from a data-gathering standpoint. Experimental observation is to be used to a lesser extent in order to obtain quantitative data on the average values of loads. These observed average values form part of the calibration process as is discussed in Section 3.4.

Since it has been established that none of the other alternatives are viable as sources of information, the question now arises of whether expert opinion can be utilised as a source of information. In principle, do experts have knowledge on the mechanisms of imposed roof loads, and if so, who are they? These issues are addressed in the following section.

### 3.2.2 Selection of Experts

The source of the information lies in the experience of the experts taking part in the survey and will be expressed through their expert opinion. The question is: are there individuals with such experience, and exactly what type of experience is necessary? Three "types" of experience are identified:

1. *Practical experience* – that is experience obtained from observing first hand the load mechanisms on the roof over a substantial period of time.
2. *Reflective experience* – that is experience, or rather expertise, obtained from consideration of the load mechanisms and contemplating about their magnitudes and application.
3. *Philosophy-of-design experience* – that is expertise on a higher level where the load mechanisms are evaluated in terms of whether codified provision for them is justified.

Table 9 introduces the types of experts (defined through their professions) who have been selected, as well as the motivation for their selection in terms of the load mechanisms for which they can provide information and in terms of the three "types" of experience identified in the above.

**Table 9. Selection of the Type of Experts to take part in the Survey**

| Type of Experience | Expert who has such Experience | Reason for Expert having such Experience | Information available through this Experience |
|---|---|---|---|
| Practical | Structural Engineers, Steel- and Roofing Contractors | -Structural engineers with a sufficient number of years of experience would have observed the load mechanisms on the roof during site supervision.<br><br>-Steel and roofing contractors – due to the nature of their profession are directly involved with the construction, maintenance and repair of light industrial steel buildings. | Quantitative information on construction and maintenance and repair loads |
| Reflective | Structural Engineers | Structural engineers are involved in the design of light industrial steel buildings, which prompts them to consider and ponder about the imposed roof loads thereof. Also, some engineers may have been involved with, or know of, failures of structures due to imposed loads on the roof and such information would be invaluable to the study. | Quantitative information on construction and maintenance and repair loads, as well as machinery suspended from the roof |
| Philosophy-of-design | Structural Engineers | Structural engineers experience first hand the implications of what the loading code provides for, and of how well the codes "perform" in practice. Therefore, they would certainly have an opinion (and a valuable one) on the codified provision of imposed loads for inaccessible roofs. | Identifying the types of load mechanisms for which codified provision is warranted |

So, the experts to take part in the survey are steel- and roofing contractors and structural engineers, for the reasons stated in Table 9. To ensure that the structural engineers have sufficient practical, reflective and philosophy-of-design experience, a minimum of five years experience is imposed on them, i.e. only engineers with five or more years of experience will take part in the survey.

## 3.2.3 Managing Expert Opinion Measurement so as to yield Rationally Defendable Results

The expert opinion is to be elicited through a questionnaire put forward to them. As stressed in Section 1.5, it is imperative that experts are selected scrupulously, with carefully consideration of their fields of expertise, and that their opinions are obtained and managed in a scientific manner that yields rationally defendable results. Also, the questions put forward to them must be such that they ensure that all necessary information is obtained from the experts in order to successfully complete the experiment.

Towards establishing a design value for the imposed roof load, the uncertainty in the load needs to be modelled, and therefore it is necessary to obtain this uncertainty from the experts. People are normally not familiar with expressing and quantifying uncertainty. COOKE (1991) developed this process of utilising expert opinions in uncertainty in a scientific manner. It is with reference to this publication that the basic principals for utilising expert opinion are formulated:

"Science aims at rational consensus, and the methodology of science must serve to further this aim. Were science to abandon its commitment to rational consensus, then its potential contribution to rational decision making would be compromised. Expert opinion may, in certain circumstances, be a useful source of data, but it is not a source of rational consensus. Given the extreme differences of opinion encountered in virtually every aspect of engineering science, it is clear that an unmethodological use of expert opinion will not contribute to rational consensus building." - *Cooke.* Therefore, certain principles have been formulated as guidelines for using expert opinion in science, and they are implemented in the study:

1. Reproducibility

    It must be possible for scientific peers to review and if necessary reproduce all calculations. This entails that the calculation models must be fully specified and the ingredient data must be made available. This emphasises the fact that calibrating the experts should not be dependent on the analyst's personal assessment of the reliability of the expert, since this is subjective and differs from analyst to analyst.

2. Accountability

    The source of expert subjective probabilities must be identified. The experts should be identified by name and their individual assessments should be given.

3. Empirical Control

    Expert probability assessments must in principle be susceptible to empirical control. In other words, it must be possible in principle to evaluate expert probabilistic opinion on the basis of possible observations.

4. Neutrality

> The method for combining/evaluating expert opinion should encourage experts to state their true opinions. A poorly chosen method of combining/evaluating expert opinion will encourage experts to state an opinion at variance with their true opinion.

5. Fairness

> All experts are treated equally, prior to processing the results of observations. Since empirical control is acknowledged as the means for evaluating expert opinions, in the absence of any empirical data there is no reason for preferring one expert to another.

The most important of the aforementioned principles to be satisfied in the process of obtaining and managing expert opinion are:

- *Reproducibility.* The outcome of the experiment must not depend on the surveyor, and should be the same if a different surveyor were to undertake the experiment.
- *Empirical Control.* This entails the measurement of the quality of the expert opinion and the consequent empirical combination thereof.

Furthermore, it is the opinion of the author that the experiment must be transparent. The manner in which the survey is conducted, the questions asked, the expert's response to the questions, and all the assumptions and methods used in processing the data obtained from the experts, should be transparent for all to see.

Thus, when constructing the process of obtaining the expert opinions and combining them, these principles are used as the basis to adhere to in every step.

The exercise is to be conducted the same way as any scientific experiment in a laboratory. The following comparison is drawn in Table 10:

**Table 10. Comparison of Laboratory Experiment with Expert Survey**

| Step | Laboratory Experiment | Expert Survey |
|---|---|---|
| 1. Design | Assessment and assembly of all equipment necessary to perform the experiment and to obtain meaningful results. | Assessment of information needed from the experts and constructing the questionnaire accordingly. |
| 2. Set-up | Set-up of the equipment and performing test runs to ensure that the experiment is working properly. | Selection of experts and performing the preliminary consultation and test run of the questionnaire with a selected number of experts. |
| 3. Calibration | Calibration of the experiment by comparison of experimental results with known values for the effects being tested. The experiment equipment is adjusted until sufficient correlation is obtained. | Calibration of the expert survey by adjusting the different weights attributed to their opinions until sufficient correlation is obtained between the combined expert opinions and known values. |
| 4. Conducting | Physically conducting the experiment. | Conducting the survey through interviewing selected experts. |
| 5. Data analysis | Utilising the calibrated experimental results to obtain data for effects of which the values are unknown. | Utilising the calibrated expert opinions with their appropriate weights to obtain data for which the values are unknown. |

The application of the five steps in the expert survey is not in the same order as is shown in Table 10. Rather, the order for the expert survey is as follows:

1. Design
2. Set-up
3. Conducting
4. Calibration
5. Data-analysis

The reason for steps 3 and 4 being switched for the expert survey will become clear subsequently.

The following steps are to be implemented in conducting the expert survey experiment:

1. Design of the experiment.

    A preliminary study is to be performed in order to identify exactly what data will be required from the experts for the successful completion of the experiment. This involves identification of all the possible mechanisms involved in producing the imposed roof load and the information required for probabilistic modelling of the load mechanisms. An initial questionnaire is developed as a means to obtain this data. It is also necessary to decide on the methodology to be implemented in calibrating the experiment.

2. Set-up of the experiment.

   The experts taking part in the survey are to be selected. A consultation session is to be held with a representative number of experts. The contents and formulation of the final questionnaire largely depends on this preliminary consultation session. A test-run of the final questionnaire is performed with a selected number of experts. This allows for minor adjustments to be made to the questionnaire before the full survey is conducted.

3. Conducting of the experiment.

   The final questionnaire is put forward to all the experts who are selected to take part in the survey. Data is obtained from the experts for variables of which the true values are known, and variables of which the values are unknown.

4. Calibration.

   Experts have been asked questions to which the answers are known through observations made by the surveyor (see Section 3.4.1). These are called *seed questions* and the variables to which they relate are subsequently called *seed variables*. Relative weights are then attributed to the experts according to the amount of correlation existing between the particular expert's opinion and the observations.

5. Data analysis.

   The relative weights calculated for each expert are utilised to combine the expert opinions for those variables of which the values are unknown (or not observable in the near future). The variables of which the values are unknown are called the *maximum variables* (see Section 3.4.1).

For practical reasons the seed and maximum questions are put forward to the experts during the same interview and therefore the calibration only takes place after the survey has been conducted.

## 3.3 Probabilistic Modelling of Imposed Roof Loads

This section is concerned with obtaining the necessary information from the expert survey in order to successfully model the imposed roof load probabilistically. To simplify and make the models quantifiable, some assumptions would have to be made. It is important that such assumptions are conservative, i.e. that they would lead to models that produce higher rather than lower load values.

In order to construct the questionnaire it is first necessary to ascertain exactly what information is needed from the experts for the successful modelling of the load mechanisms as identified. This process will only be completed after the preliminary consultation sessions.

The uncertainty associated with imposed roof loads needs to be quantified. First it is necessary to define this uncertainty. Uncertainty in imposed loads generally results from two sources (refer also to ISO 2394-1998 for the general principles to apply when modelling uncertainty):

1. The variability of the *magnitude* of the imposed load.

    For a specific building, the intensity of the imposed load on a roof varies with time as shown in Figure 6.
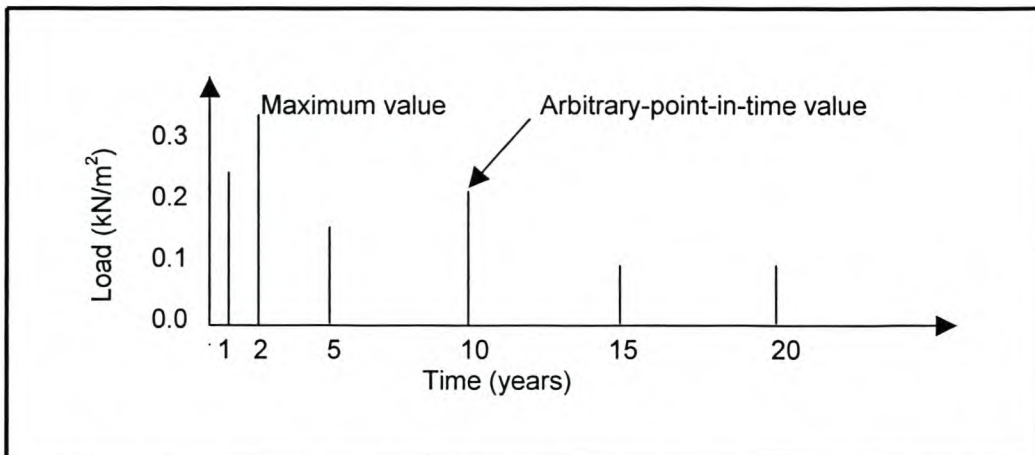


**Figure 6. Variance of the Imposed Roof Load over the Lifetime of a Building**

This is a typical example of the variance of the imposed load for an inaccessible roof over its design life, where the load is applied for very "short" time intervals. For the purpose of this study, only the maximum value over the lifetime of the building is of importance. This value is independent of time since it is the only value that emerges over the lifetime for a specific building.

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The uncertainty in the maximum value results from the fact that for different buildings, different maximum values over the lifetimes will emerge. The variability in the maximum value is therefore a function of the *number* of buildings and not of time. Attention is drawn to the fact that the variability obtained applies only to the variance in *magnitude* of the load.

2. The *spatial variability* of the imposed roof load.

   Modelling of imposed loads as uniformly distributed loads (UDL's) is purely a design simplification. In reality, imposed loads would be better approximated by a number of concentrated loads in arbitrary positions on the tributary area of a specific member. The positioning of the imposed load certainly has an effect on the load effect under consideration. Take for instance the midspan moment for a beam. A concentrated load that occurs near midspan would result in a larger moment than one positioned at an offset from midspan. Obtaining an equivalent uniformly distributed load (EUDL) for the midspan moment would therefore depend on the position of the concentrated load. This variability in the EUDL according to the positioning of the load is referred to as the *spatial variability* of the imposed roof load and it is very relevant to imposed roof loads for inaccessible roofs.

The question now arises of how one is to obtain these two uncertainties from an expert survey, i.e. what questions need to be asked in order to quantify these two uncertainties. This is addressed in Sections 3.3.1 & 3.3.2.

## 3.3.1 Quantifying the Uncertainty in the Magnitude of the Imposed Roof Load

As stated earlier, the maximum lifetime load is of interest for this study, as this is the value for which the building should be designed for, with incorporation of the required factor of safety. For 20 buildings the maximum lifetime loads are illustrated in Figure 7.

The 1-in-20-building maximum value as shown in Figure 7 is to be obtained from the experts. This value is also know as the 95% maximum value which implies that no more than 1 in every 20 buildings would have a larger imposed roof load than this over its lifetime. The uncertainty in this value results from the fact that another set of 20 buildings would yield a different 95% maximum value.
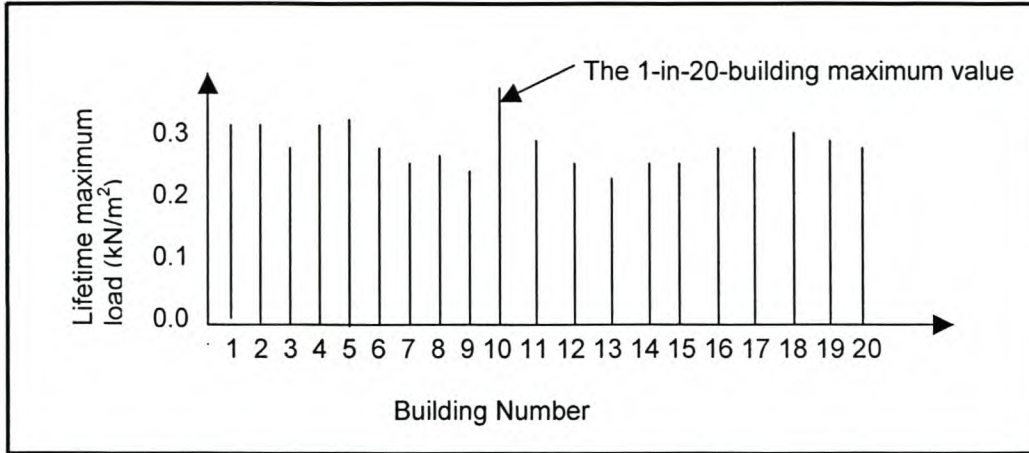
**Figure 7. The Maximum Imposed Roof Loads for 20 Buildings**

The basis for deriving probabilistic models for the maximum values emerging from sets of samples is given in ANG & TANG (1984). The 95% maximum value is a maximum or "extreme" value in two senses: Firstly, the 95% maximum value is the maximum lifetime load, and secondly it is the 1-in-20 building maximum load. Therefore, it is appropriately modelled as an *extreme type 1 basic random variable* for which the probability density function resembles an upper-tail distribution as shown in Figure 8.
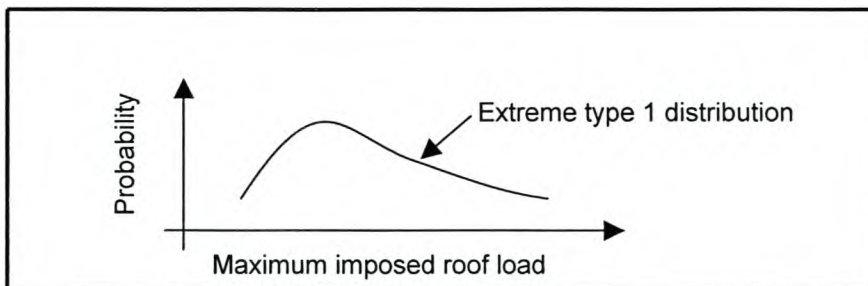


**Figure 8. Probability Density Function of the Maximum Imposed Roof Load**

To obtain the uncertainty in the magnitude of the 95% maximum imposed roof load, experts will be asked to provide their 90%-confidence interval for the magnitude of the imposed roof load being considered. For example, if one were to obtain the variability for the 95% maximum load, the following question would pertain: "What are the two values for the 95% maximum imposed roof load for which you are 90% certain that the true value of the 95% maximum imposed roof load would fall between?" These two values now constitute the 5% and 95% cumulative probability values, and together with the type of distribution (extreme type 1), the variance for the load variable is obtainable. Note that any two values together with their

cumulative probabilities may be used to obtain the variance. Another option may be to compute the variance from the distance between the average and the 95% - values. The expert's "best estimate" constitutes the average value of the basic variable. Notice that this average value is the average value of the 1-in-20 building maximum random variable and therefore actually the 95% maximum value.

### 3.3.2 Uncertainty due to the Spatial Variability of the Imposed Roof Load

This uncertainty is neither readily obtainable nor quantifiable from an expert survey. The approach followed is to position the imposed load, depending on the nature of the load mechanism as well as the member and load effect under consideration, so that it produces the highest value for the relevant load effect. This method would neutralise the effect of spatial variability as it conservatively allows for the loads to be arranged in the worst positions on the tributary area of the member.

As an alternative, the spatial variability could be modelled probabilistically. This would result in a more probabilistic derivation of the loads and a "less conservative" model would be obtained. To perform a full probabilistic derivation would necessitate that all parameters (such as the weight of a person on the roof, the building geometry, etc.) be treated as basic random variables, and this would result in rather "tedious" reliability analyses. Rather, the approach is to treat those parameters for which the contribution in terms of uncertainty is small, as conservative deterministic values.

Thus, for the expert survey there would not be questions pertaining to the spatial variability of the loads.

### 3.4 Calibration of the Experiment

This section is concerned with designing the expert survey so that it allows for measurement of the quality of the expert opinion, i.e. to attribute weights to the different expert opinions. It has already been established that it is imperative that expert opinion be managed effectively and combined scientifically in order to obtain the most representative results. The importance of selecting the correct type of experts, whom in principle would have knowledge of the load mechanisms, has also

been stressed. The reasoning adopted in the selection of experts is set forth in Section 3.3.2. To recapitulate, the two types of experts whom have been selected to take part in the survey are:

1.  Structural engineers with in excess of five years experience

2.  Steel- and roofing contractors

These experts' opinions are to be elicited through a questionnaire put forward to them. The combining of expert opinion may be done in three ways:

1.  Average combination.

    The expert opinions are combined by taking the average of all the opinions. Therefore all experts have equal weights. The selection of experts (in terms of the number and "type") by the surveyor is critical for this way of combining expert opinion.

2.  Weighted combination.

    The expert opinions are combined according to the relative weights attributed to them. Evidently, the experts would have to be ranked (seeded) in terms of the value of their opinions. This way of combination de-sensitises the outcome of the experiment from the selection of experts by the surveyor since those experts whose opinions perform poorly in the measuring process receive small weights.

3.  Best expert.

    Only the expert's opinion that received the highest weight (or highest ranked) is used. This is an extreme case of expert combination.

The weighted combination of expert opinion (number 2 in the above) is implemented in this study. The other two combination methods are used to evaluate the "performance" of the weighted combination through comparison of the results. The way in which the "performance" of a combination method is measured is discussed in Section 4. The methodology adopted in ascribing weights to the individual experts' opinions is set forth in the following section.

### 3.4.1 The Seed Variables

It has been stated in Section 3.3.1 that the 95% maximum values of the loads are of importance to this investigation. As argued in Table 8, the maximum values are not readily measurable through any kind of physical load survey or experimental observation, but are only obtainable through the knowledge and experience of experts. Therefore, quantifying the 95% maximum values for the load mechanisms is defined as the *purpose* of the expert survey. The 95% maximum values translate into the *maximum random variables* through the relation discussed in Section 3.3.1.

In attributing weights to the individual experts' opinions, the fundamental question that needs to be answered is how can one say that expert X's opinion on the maximum variables is closer to the truth than expert Y's. The answer is through the *seed variables*. In principle, seed variables are variables that are observable or measurable by the surveyor so that their values may become known. The experts taking part in the survey are asked *seed questions* to which the answers reflect their best estimates of the seed variables. One can now go forth from the principle that if expert X's assessment of the seed variables correlates better with the observed (true) values of the seed variables than expert Y's, then expert X's assessment of the maximum variables (unknown) would also be closer to the truth. So, the approach is to use what is measurable (seed variables) to find better results for what is not measurable (maximum variables). In essence, the principle of extrapolation of the results is utilised.

In order to contribute to rational consensus, the seed variables have to satisfy the following three requirements:
1. Realisations of seed variables have to be observable and measurable.
2. Seed variables have to be related to the maximum variables in terms of content and nature. In other words, one should be able to confidently say that if a certain expert performs better than another one for the seed variables, then his/her opinion for the maximum variables would also be closer to the truth.
3. All the experts taking part in the survey have to be subjected to the same set of seed questions so that all experts are treated equally.

Subject to the above restrictions, it is a difficult feat to establish viable seed variables, especially in satisfying requirement 2 in the above. Requirement 2 is the most important of the three, as it can be expected that seed variables not related to the

maximum variables would in no way reflect truthfully on an expert's best estimate of the maximum variables. This matter is further complicated by the fact that two types of experts take part in the survey, namely engineers and contractors. Engineers and contractors have different backgrounds in education and professional experience. This may lead to, for instance, the engineers performing better for certain seed questions than the contractors; but if the seed variables are not related to the maximum variables this would have no bearing on the relevant knowledge of the experts. The solution to this problem is to só choose the seed variables that they are directly linked to the maximum variables. The way in which this is done is explained in the following:

Recall that in Section 3.3 Figure 7 the 1-in-20-building maximum value over the lifetime is defined. Similarly, the *average value* of the maximum lifetime values of the structure is also obtainable. This is illustrated in Figure 9 for 20 buildings.
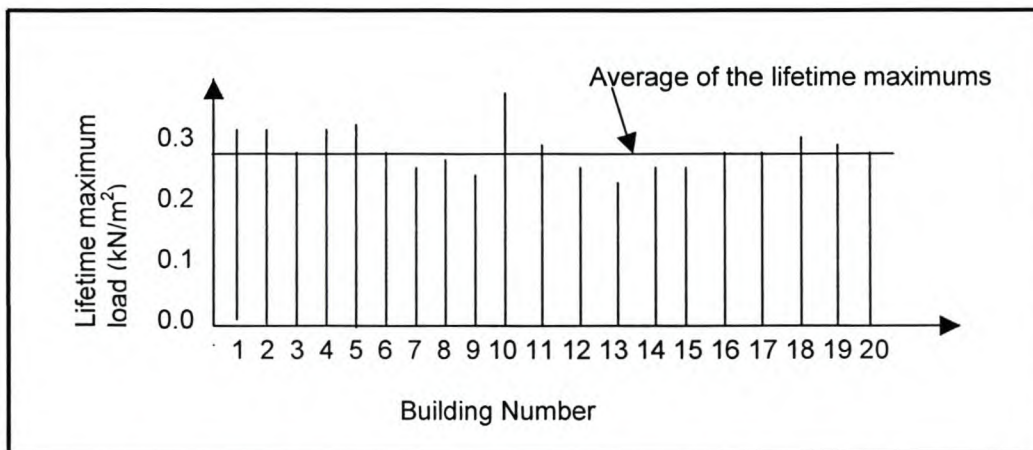


**Figure 9. The Average Imposed Roof Loads for 20 Buildings**

The uncertainty in the average load results from the fact that the average value obtained for a different number of buildings (say 40 buildings) would be different than the average for 20 buildings. This uncertainty can be quantified from the experts through eliciting their best estimates and 90%-confidence intervals as is done for the maximum variables in Section 3.3.1. The 5%, 50% and 95% so obtained from the experts now represent their "estimate" of the average value of the load.

The average values for the construction loads are measurable by conducting a construction site load survey for a number of buildings. Since the construction loads occur only once, the observations made during the site survey can be regarded as

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"maximum" lifetime values. By taking the average of the observed construction loads over a number of buildings, this now constitutes a realisation of the seed variable, *The Average Construction Load,* and the experts' "estimate" on this seed variable can be measured against this realisation. Therefore, the *average construction loads* are to serve as the seed variables. Since there are only three construction load mechanisms identified (see Section 3.1), this restricts the seed variables to three. The implication of having such few seed variables and the way in which this is dealt with is discussed in Section 4.

There is uncertainty in the seed variables since the average value is obtained through a limited number of observations, therefore it is not deterministic and is to be modelled probabilistically. An appropriate distribution function is to be selected for the seed variables. An important property of the seed variables is its close relationship with the maximum construction variables (the one being the average and the other the maximum value of the same variable), thus ensuring that experts are judged on relevant information.

The relationship between the seed variables and the maximum variables is illustrated in Figure 10.
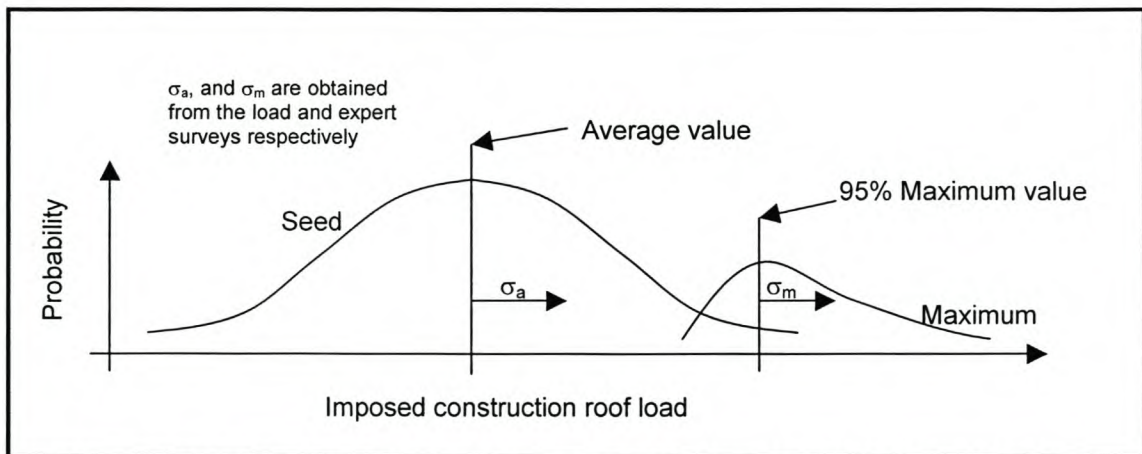


**Figure 10. Probability Density Functions of the Seed and Maximum Variables**

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In summary of the calibration procedure the following flow chart is presented:
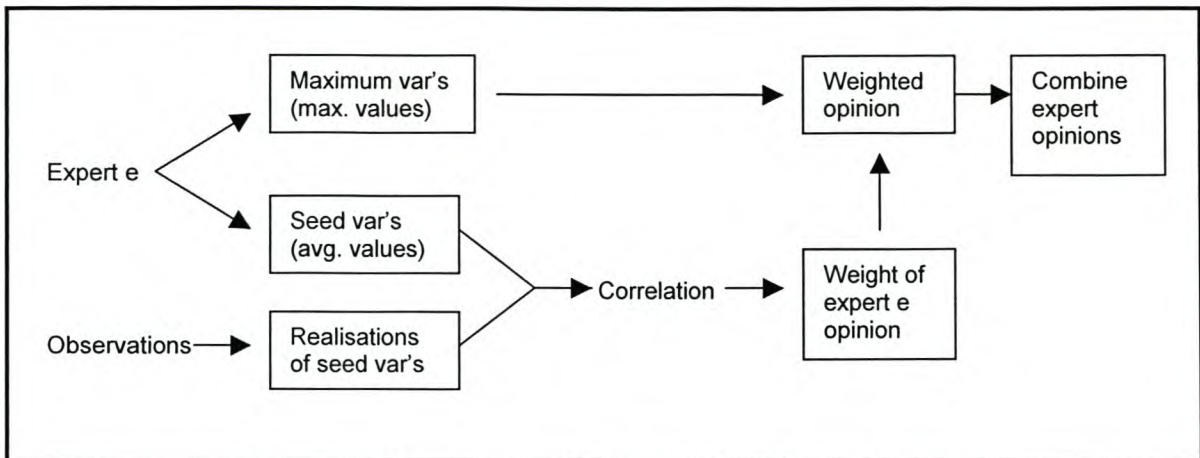


**Figure 11. Summary of the Calibration Procedure**

## 3.5 Information to be obtained on the Load Mechanisms

The purpose of this section is to perform the preliminary planning on the individual load mechanisms in order to establish exactly what information will be required from the survey.

### 3.5.1 Expressing the Load Mechanisms in Expert Terms

The SABS 0160-1989 code provides for imposed roof loads by prescribing uniformly distributed loads (UDL's) dependent on the tributary area of the member under consideration (see Section 1.2). In order to evaluate SABS provisions for imposed roof loads, the load mechanisms would ultimately also have to be expressed in terms of uniformly distributed loads. However, these uniformly distributed loads are not readily or justifiably obtainable from the experts. Attempting to elicit quantitative information on the load mechanisms from the experts in terms of uniformly distributed loads would seriously detriment the validity of the results of the experiment. The reason for this is threefold:

1. Take for instance the load resulting from workers on the roof during construction. An expert will only be able to provide a meaningful estimate of the resulting UDL if he/she takes into consideration the number of workers that are likely to be on the roof at a specific time, what the weight is of a typical worker and how the workers would be distributed on the roof. This, he/she would then have to

convert into an UDL before an answer could be presented. As is evident, this procedure is rather taxing on the expert, and it allows for a large amount of human errors to be made.

2. A situation may occur where two experts predict the same number of workers on the roof, but the corresponding UDL's for the two experts differ if they where to convert the number of workers into UDL's themselves. The reason for this is that their conversion processes differ, since it could be based on different assumptions. It is best to have a uniform conversion process to be used for all experts, and to be conducted by the surveyor to ensure that "apples are compared with apples." Also, steel- and roofing contractors would not have such a good understanding of UDL's as engineers do, which would also lead to incongruities in the results.

3. Engineers would more than likely be susceptible to anchorage – that is where they "anchor" their answers to the prescribed codified values of the imposed roof load. Answers that are based on SABS prescribed loads would obviously be of no use to this investigation since it is exactly the SABS that is to be evaluated. Anchorage will be promoted if the answers are to be given in terms of UDL's since this agrees with the way in which the SABS provides for the loads.

To avoid the problems stated in the above, the experts will be asked to state their answers in terms of the quantities of the load mechanisms that translate into the imposed roof load, and not the load. So, for construction workers on the roof, the experts are to provide their answers in terms of the *number* of workers on the roof. Converting the number of workers on the roof to an UDL could then be done separately by the analyst with known mathematical procedures.

Through this approach the "expertise" of the experts is utilised, and it is recognised that more accurate results will be achieved if the *load mechanisms* are measured through expert opinion, and not the loads.

### 3.5.2 Workers on the Roof during Construction, Repair, Cleaning and Maintenance

The imposed load due to workers on the roof during construction, repair, cleaning and maintenance is to be quantified through eliciting the number of workers on the roof from the experts, as explained in the previous section. It is anticipated that high spatial variability exists in the positioning of workers on the roof. Therefore, the workers are to be positioned on the roof so as to maximise the critical load effects for the structural member under consideration. For these load effects, equivalent uniformly distributed loads (EUDL's) may then be calculated which would result in the same magnitudes for the load effects as for the original configuration of workers on the roof.

The SABS 0160-1989 (as well as other loading codes) expresses the imposed roof load as a function of the tributary area of the member under consideration. The relation is that the prescribed imposed load decreases as the tributary area increases (see Section 1.2). This phenomenon is readily manifested in the load resulting from workers on the roof due to the high spatial variability associated with this load. The maximum values for the number of workers would be different for different areas due to the fact that the ratio of probable-number-of-workers-on-an-area to area-size increases as the size of the area decreases. For example, it is more likely that there be 1 worker on a tributary area of 1 $m^2$ than 2 workers on a tributary area of 2 $m^2$.

Intuitively, to expect from the experts to relate different number of workers to different area sizes would not yield results of any merit. A way in which this problem may be solved is by providing the expert with some physical concept with which he/she is well familiar. The primary load carrying members for light industrial steel buildings are the purlins and frames. Therefore, large tributary areas can be accounted for by the tributary areas for frames and small tributary areas can be accounted for by the tributary areas for purlins. These two areas provide two cases for which the SABS provision for imposed roof loads may be evaluated. A final decision on this can only be made once the preliminary consultation with experts has been completed (see Section 3.6.3).

### 3.5.3   The Stacking of Roof Cladding during Construction

After the cladding is secured into place (when the construction phase is over), the frames are subjected to the gravitational load due to one bay's cladding.  This weight should be (and is) treated as the dead load component of the total load (by the SABS ultimate limit-states design provisions) since there is low variability in this.  However, when over-stacking occurs this load should be treated as an imposed load because of high uncertainty in the amount of over-stacking which occurs.   It would be convenient if this additional load due to the over-stacking of cladding were expressed in terms of the number of bays' cladding stacked on one frame.   The additional imposed load would then be the dead load due to one bay's cladding times the additional number of bay's cladding stacked on the rafter: $L_{over\text{-}stacking} = D_{cladding}$ x n, where n = *additional* number of bays.  The high uncertainty in n warrants $L_{over\text{-}stacking}$ being modelled as an imposed load.

To determine the imposed load for smaller tributary areas (such as for purlins) it is necessary to establish the uncertainty involved in whether or not cladding is stacked on the purlins.  Stacking on the purlins can certainly be regarded as bad practice by the building contractor and common sense says that the purlins should not be designed for such a load.   However, when conducting a reliability analysis and assessing failure probability the uncertainty in this is relevant and should therefore be determined.

### 3.5.4   Machinery and Equipment supported by the Roof

It is to be expected that quantitative values for these mechanisms would not be readily obtainable from the experts.  High uncertainty exists regarding these load mechanisms due to a lack of information on them.  Through the surveying process this uncertainty will be reduced by assessing the degree of agreement which exists for the selected number of experts.

An issue that needs to be resolved is that of whether or not it is warranted to include these load mechanisms (machinery installed in the roof over the lifetime of the structure) in a reliability study.  One can argue that the client should bring any changes made to the building (functional changes), which impose loads on the roof, to the engineer's attention and that the building should be re-evaluated.  If damage to

the building occurs as a result of such actions by the client (without notifying the engineer) the engineer cannot be held responsible.

Such a philosophy would have the following effect on the reliability of the structure: Since the magnitude of the load as well as its position is known to the engineer, the resistance of the structure is adjusted accordingly. The limit-state criterium for the resistance is (refer to SABS 0160-1989) $0.9 \text{Resistance} \geq 1.2 \text{Dead Load} + 1.6(0.3^* + \text{Load}_{equipment})$. It is evident that this would result in a higher resistance and therefore more reliable structure, than when the $\text{Load}_{equipment}$ is not known and accounted for by the engineer.

Since the imposed load due to equipment or machinery suspended from the roof occurs only in isolated instances and has a very localised load effect on the structure, it would not be economical to increase the prescribed imposed load of the SABS in order to make provision for such types of imposed loading. Through utilising the philosophy-of-design experience of the engineers, further clarity on this matter will be obtained (see Section 3.6.2).

### 3.5.5   Rainwater, Hail and Snow accumulating on the roof.

This load mechanism differs from the others in the sense that it is the only imposed load that strictly has all the attributes associated with a stochastic variable. This statement is based on the fact that rain, hail and snow are occurrences of nature that are totally uncontrollable by man and their variability is inherently random. For all other load mechanisms there certainly is a degree of human control that can be implemented.

Between rainwater, hail and snow accumulating on the roof the latter two can certainly be regarded as the largest in terms of the load it produces on the roof, and it is therefore the hail and snow loads that will be considered further.

The experts whom have been selected to take part in the survey, namely engineers and steel- and roofing contractors, would not have knowledge about hail and snow

*The SABS 0160-1989 prescribes a minimum uniformly distributed imposed load of 0.3 kN/m$^2$ for inaccessible roofs

precipitation since this is not included in their fields of experience as indicated in Table 9.

Information on hail and snow precipitation would have to be obtained from records of the weather bureau or an alternative source, and the modeling of this basic variable does not justify the conducting of an expert survey. Therefore, there are no questions in the survey pertaining to the load due to hail and snow accumulating on the roof.

### 3.6 Preliminary Consultation

In developing the questionnaire it is first necessary to do preliminary consultation with a number of experts. Through this process it is possible to observe first hand the experts reactions to the questions, i.e. how well they understand the questions, and to adhere to any further commentary and advice which they have on the subject. The final questionnaire could then be developed and streamlined through elimination of irrelevant questions, adjustment of current questions and addition of necessary questions as identified in the consultation. For this reason the initial consultation session would be an interactive process with the surveyor being present at the time of questioning.

It has been established that two types of experts are to partake in the survey, namely structural engineers and steel- and roofing contractors. Apart from the quantitative questions, i.e. questions pertaining to the seed and maximum variables, there are also the so-called method and philosophy-of-design questions. The method questions are primarily questions concerned with how the imposed loads are applied and how they can subsequently be modelled. These questions will therefore only appear in the preliminary questionnaire. The philosophy questions are questions pertaining to the design philosophy to be adopted for certain types of imposed roof loads. The philosophy questions will primarily be directed to engineers. Obtaining the engineers opinions on these matters will aid in the process of stating a case for which loads codified provision is justified.

### 3.6.1 Experts to take part in Preliminary Consultation

In Section 3.2.2 the type of expert to take part in the survey is selected, whilst in this section the specific experts are personally selected. The selection of experts is based on the expert's experience, professional status, qualifications and availability. PARTNERSHIP DE VILLIERS and PROFFESOR P DUNAISKI were used as the initial contacts from where other potential experts were identified.

The experts who took part in the preliminary consultation are six practising civil engineers, two steel contractors and one roofing contractor. They are presented in Table 11.

**Table 11. Experts taking part in the Preliminary Consultation**

| Name of Expert | Type of Expert | Company | Years of Experience |
|---|---|---|---|
| P J de Villiers | Structural Engineer | Partnership de Villiers | 28 |
| I P de Villiers | Structural Engineer | Partnership de Villiers | 29 |
| W Hugo | Structural Engineer | Partnership de Villiers | 12 |
| F Heyman | Structural Engineer | Partnership de Villiers | 49 |
| F van Zyl | Structural Engineer | Raath and Van Zyl | 29 |
| A Ellmer | Structural Engineer | Ellmer and Partners | 29 |
| D Payne | Roofing Contractor | Scheltema Roof Sheeting | 18 |
| M Papanicolau | Steel Contractor | Union Steel | 16 |
| I Gillmore | Steel Contractor | Target Steel | 25 |

Firstly the philosophy-of-design questions directed to the engineers, as well as their responses are presented, followed by the conclusions made by the surveyor for these questions. Secondly the method and quantitative questions directed to all the experts, together with the surveyor's conclusions from this consultation session are presented.

### 3.6.2 Consultation Session regarding Philosophy-of-Design Questions

Included in this section are the philosophy-of-design questions and the engineers' responses to them. As stated previously, the quantitative questions are omitted in this presentation. The main purpose of the philosophy-of-design questions is to establish what the views of engineers are on codified provision for imposed roof loads.

The following load mechanisms have been identified as sources of the imposed roof load for which it is debatable if codified provision is warranted:

- Stacking of roof cladding on the purlins.
- Services suspended from the roof such as lighting, water mains, air-conditioning, etc.
- Equipment suspended from the roof and / or the loads involved during installation of such equipment.

Table 12 summarises the experts' opinions on the above.

Other issues regarding imposed roof loads, which were addressed (in an informal manner), are:

- What is the engineers responsibility regarding occupancy changes during the lifetime of the structure?
- What are the explanations for the overseas loading codes (ASCE, BS, EURO) having a higher prescribed imposed load for inaccessible roofs?
- What are the experts' opinions on the current prescribed imposed roof load of the SABS 0160-1989?

Table 13 summarises the experts' opinions on the above issues.

**Table 12. Alternative Sources of the Imposed Roof Load and the Appropriate Treatment thereof**

| Expert | Provision for stacking of roof cladding on purlins | | Provision for the weight of services suspended from the roof. | | | Are there any other special cases that produces a large imposed roof load. For example, during the installation of services or equipment in the roof? Or equipment hanging from the roof. How should these cases be provided for? |
| | Does Stacking on Purlins occur? | Does SABS need to provide for overstacking on purlins through it's prescribed live roof load? Provide reason. | Weight of services | | How do you provide for the weight of services suspended from the roof (such as waterpipes, lighting and air-conditioning)? | |
| | | | Average (kN/m$^2$) | Maximum (kN/m$^2$) | | |
| I.P. de Villiers PhD structures Pr. Ing | Never occurs | No, building contractors' responsibility to ensure no overstacking on purlins, therefore weight of sheets is dead load. | 0.1 | 0.15 | For normal cases, I use 0.1 kN/m$^2$ dead load. For special cases, such as large water pipes in the roof (of which I should be made aware of by the client) I determine the weight for that specific case and treat it as a dead load. | No, workmen involved in installation of services are easily covered by the 0.3 kN/m$^2$ of the code. Equipment suspended from the roof should be brought under the engineers attention by the client and should be provided for in addition to the 0.3 kN/m$^2$ of the code |
| P.J de Villiers Pr. Ing | Never occurs | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. | 0.05 - 0.1 | 0.15 | For normal cases, I use 0.1 kN/m$^2$ dead load. For special cases, such as large water pipes in the roof (of which I should be made aware of by the client) I determine the weight for that specific case and treat it as a dead load. | No, workmen involved in installation of services are easily covered by the 0.3 kn/m$^2$ of the code. Equipment suspended from the roof should be brought under the engineers attention by the client and should be provided for in addition to the 0.3 kN/m$^2$ of the code. |
| F. Heyman Pr. Ing | Occurs, as a result of negligence from building contractors | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. If structural damage occurs during construction as a result of overstacking or weight of workmen, the building contractor should correct it on site at his own expense. | 0.1 | 0.2 | For normal cases, I use 0.1 kN/m$^2$ dead load. For special cases, such as large water pipes in the roof (of which I should be made aware of by the client) I determine the weight for that specific case and treat it as a dead load. I know of engineers who accept that the 0.3 kN/m$^2$ live roof load value of the SABS makes provision for services. | Yes, during occupancy changes of the building. An extreme situation can also occur where the building is situated next to a sports ground and spectators would climb onto the roof. However, this cannot be provided for through the SABS. |

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**Table 12. Continued**

| | | | | | | |
|---|---|---|---|---|---|---|
| W. Hugo<br>Pr. Ing | Should not but does occur | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. | 0.1 | 0.15 | When there is small uncertainty in the geometry of the services, I use 0.1 kN/m$^2$ dead load, otherwise where larger uncertainty exists, such as for large shopping centres I use 0.1 kN/m$^2$, live load. | No, I have not encountered any such cases in my experience. Few people are involved in installation of services and equipment involved is supported by the ground. The code cannot provide for equipment hanging from roof as this should be determined for each specific case. |
| F. van Zyl<br>PhD structures<br>Pr. Ing | Never occurs | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. | 0.1 | 0.15 | For normal cases, I use 0.1 kN/m$^2$ dead load. For special cases, such as large water pipes in the roof (of which I should be made aware of by the client) I determine the weight for that specific case and treat it as a dead load. | No, workmen involved in installation of services are easily covered by the 0.3 kn/m$^2$ of the code. Equipment suspended from the roof should be brought to the engineers attention by the client and should be provided for in addition to the 0.3 kN/m$^2$ of the code. |
| A. Ellmer<br>Pr. Ing | Never occurs | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. | 0.2 - 0.3 | 0.3 | In preliminary design I use 0.2 kN/m$^2$ dead load. This is for typical services such as electric cables and water sprinklers. This is on the safe side. | No, the load due to workmen on the roof during installation is nominal. Equipment hanging from the roof I treat as dead load. |

## Table 13. General Aspects regarding Imposed Roof Loads

| Expert | What is the engineers' responsibility in terms of occupancy changes during the lifetime of the structure? | What is the reason for the overseas loading codes (ASCE, BS, Euro) having a higher imposed roof load for inaccessible roofs than the SABS has? | Any comments on the current prescribed load intensities of the SABS for the imposed roof load? |
|---|---|---|---|
| I.P. de Villiers PhD structures Pr. Eng | It is not possible to provide for occupancy changes in the original design. The client and contractors are responsible for notifying the engineer if any changes are to take place. | Snow, as well as more severe weather conditions in general in these countries. | The current values of SABS are a bit too conservative. |
| P.J de Villiers Pr. Eng | The engineer has to re-design the structure for its new purpose / loading. Uneconomical to increase the prescribed values to compensate for such changes. | Snow, as well as more severe weather conditions in general in these countries. | The current values of SABS are a bit too conservative. |
| F. Heyman Pr. Eng | The engineer has to design for the clients' needs at that stage. In case of changes the engineer should be consulted with. | Snow, as well as more severe weather conditions in general in these countries. | The current values of the SABS are acceptable. |
| W. Hugo Pr. Eng | It is not possible to provide for occupancy changes in the original design. The client and contractors are responsible for notifying the engineer if any changes are to take place. | Snow, as well as more severe weather conditions in general in these countries. | The current values of SABS are a bit too conservative. |
| F. van Zyl PhD structures Pr. Eng | The engineer has to design for the clients' needs at that stage. In case of changes the engineer should be consulted with. | Snow, as well as more severe weather conditions in general in these countries. | The current values of the SABS are acceptable. |
| A. Ellmer Pr. Eng | The engineer has to design for the clients' needs at that stage, and he should inform the client for what purpose the building is designed for, and that any changes are to be reported to the engineer beforehand. | Snow, as well as more severe weather conditions in general in these countries. Specifically Germany is more conservative. This is also due to the fact that more money is available than in developing countries. | Sufficient reliability is not obtained with the current SABS roof live loads. |

From observing the experts' opinions on the basis-of-design questions, it is evident that they are certainly consistent in their opinions. A summary of the results and conclusions made by the surveyor are presented in Table 14.

**Table 14. Results and Conclusions from the Preliminary Consultation regarding Philosophy-of-Design Questions**

| Topic | Experts' opinions and percentage of experts having the given opinions | Conclusions made by the surveyor |
|---|---|---|
| Should the SABS loading code provide for the stacking of roof cladding on the purlins? | Yes – 0%<br>No – 100% | The consensus amongst the experts suggests that the stacking of roof cladding on the purlins is not to be regarded as a load mechanism in this study. |
| Do you make extra provision for the roof load due to services suspended from the roof over and above the SABS's prescribed load? | Yes – 100%<br>No – 0% | The consensus amongst the experts suggest that engineers in general are aware of the roof load due to services and provide separately for it. |
| Do you treat the weight resulting from services as dead or imposed load? | Dead Load – 83%<br>Imposed Load – 0%<br>Depends on Uncertainty – 17% | It is concluded that the weight of services is generally treated as dead load, suggesting that the experts perceive there to be low uncertainty in the weight of services. The correct way would be to treat each case on merit, i.e on how certain you are about the layout and weight of services. Only 17% of experts provided this answer. |
| Are there any special cases such as equipment suspended from the roof or workmen involved in installing such equipment, which should be provided for by SABS? | Yes – 0%<br>No, the structure should be redesigned for each case and such loads are to be treated as dead loads – 100% | This issue is important since it confirms that there are no extra load mechanisms which have been omitted from the initial selection (see Section 3.1). |
| What is the reason for the overseas' loading codes having higher prescribed imposed roof loads than the SABS? | Snow and generally more severe weather conditions – 100% | The experts' answers reflect the belief that snow load provisions result in the imposed load combination being less critical (as explained in Section 1.2). In the comparative study (Section 1) it was established that the other codes provide separately for snow-loads. |
| Do you think the current SABS imposed roof load is too conservative? | Too conservative – 50%<br>Acceptable – 33%<br>Non-conservative – 17% | It would be interesting to compare this with the outcome of the experiment. |
| What is the engineers responsibility in terms of occupancy/functional changes during the lifetime of the structure? Does the engineer need to design the building for possible future changes? | Yes – 0%<br>No – 100% | It was anticipated that the engineers would not design the structure for possible future changes, since this would be uneconomical. This issue is relevant since it affirms the perception that codified provision should not be too general, and that the designer is still to implement his/her own discretion. |

In summary, the two most important conclusions made from the philosophy-of-design matters are:

1. No extra load mechanisms, which have not been included in the initial selection, have been identified.

2. It has been confirmed that the load mechanisms which have been identified as debatable in terms of their appropriateness for codified provision, are not to be provided for in the SABS loading code. Particularly, the issue regarding the stacking of cladding on the purlins is not to be included in the final questionnaire.

The same philosophy-of-design questions will be put forward to all the engineers during the final survey.

### 3.6.3 Consultation Session regarding the Method- and Quantitative Questions

As stated previously, the method- and quantitative questions are put forward to all the experts (and not only to engineers as for the philosophy-of-design questions). Table 15 is a summary of the questions put forward to the experts, as well as the commentary and conclusions from the surveyor on the experts' responses. To be concise, the individual experts' responses to the questions are not presented here, but they are incorporated in the commentary and conclusions by the surveyor.

Since the experts were encouraged to provide their opinion on the questions as well as any extra commentary they have on the subject, this consultation session took on the format of an open ended conversation and was not limited to questions stated in Table 15. The information obtained is used in developing and optimising the final questionnaire.

**Table 15. Consultation Session regarding the Method- and Quantitative Questions**

| Question | Commentary and Conclusions |
|---|---|
| 1. How many workers would there normally be on an area for one portal frame, say 20m span x 5m spacing, during installation of the roof sheets? Provide your best estimate as well as your 90% - confidence interval. | The experts were initially unfamiliar with questions expecting from them to express their own uncertainty. Therefore, it was necessary to provide further explanation on certain concepts such as best estimates and 90% - confidence intervals. It is to be expected that such explanation will also be necessary when interviewing other experts, thus it is imperative that there be verbal contact between the surveyor and the expert during the interview. Also, it would be of value if these explanations where documented and presented in the final questionnaire as a means of training the expert. |
| 2. How many workers, on a 20 x 5m area, is the maximum number used during installations of the roof sheets? That is the number of workers that is large enough that there is only a 5% probability that this number of workers will be exceeded, i.e. no more than 1 in every 20 buildings would have such a large amount of workers. Provide your best estimate as well as your 90% - confidence interval. | It is also necessary that the formulation of these questions be adapted so that the experts understand exactly that their answers relate to the average over a number of buildings and the maximum per 20 buildings. Particularly they have to understand that the average for a number of buildings does not necessarily have to be an integer value. Also, although the average value is actually the average of the maximum lifetime values for a number of buildings, this will not be brought to the expert's attention. The reason being that this will only confuse the expert. The construction load will in any event be regarded by the expert as a once-off occurrence that is independent of time. |
| 3. Would there be workers on the adjacent frames at the same time or would there be one group that moves from frame to frame? | The purpose of Question 3 is to determine whether the information obtained through Questions 1 and 2 (and with the aid of Question 4) is valid. If it is the case that teams of workers operate simultaneously on adjacent rafters it can certainly happen that workers of the two teams are distributed in such a manner that they superimpose loads on one particular rafter that is larger than that which would be imposed by one group working in isolation. The experts where of the opinion that there is only one group of workers moving from frame to frame. |
| 4. Is the number of workers per frame dependent on the span of the frame and/or the spacing of the frames? If so discuss what criteria is used to determine the number of workers to be used. | The methodology adopted in converting the number of workers on larger areas ($>15m^2$) to an equivalent uniformly distributed load ($kN/m^2$) requires that the number of workers per spanning meter of the frame be known (see Section 5.1). This is in accordance with the criterion used by roofing contractors in which the number of workers required on a frame increases linearly with the span length of the frame. Thus, it will be most efficient if the questions pertaining to the number of workers on larger areas (i.e. tributary areas for frames) were structured in such a manner so that the answers obtained are in terms of number of workers per spanning meter of the frame. |

## Table 15. Continued

| | |
|---|---|
| 5. How many workers would there normally be on an area for one purlin, say 5m span × 1.5m spacing? Provide your best estimate as well as your 90% confidence interval. | The number of workers on a frame is only dependent on the span of the frame. Therefore, in obtaining the EUDL for smaller areas (such as tributary areas for purlins) it is necessary to establish how these workers are distributed on the frame, in other words the likelihood of workers congregating together on smaller areas should be determined. Upon discussion with the experts, it was their opinion that their response to questions relating different area sizes to possible number of workers on them would not be of significant value. Rather, the question should be so formulated that it provides the expert with some physical concept with which he/she is well familiar. Therefore it is decided that it would be most practical and efficient if questions pertaining to smaller areas were accounted for by the tributary area of a *purlin*. Since the primary load carrying members for light industrial steel buildings are the purlins and frames, there would be no loss in thoroughness through this approximation. The number of workers on the area for a purlin can then be arranged so as to produce the most adverse effects for the load effect under consideration. |
| 6. How many workers, in your opinion, on a 5 x 1.5m area is the maximum number? That is the number of workers that is so large that there is only a 5% probability that this amount of workers will be exceeded, i.e. no more than one in every 20 buildings would have such a large number of workers. Provide your best estimate as well as your 90% confidence interval. | |
| 7. How are the workers distributed over the purlins when the sheeting is put in place? Do you foresee any specific reasons for workers to congregate? If so, discuss. | When modelling the number of workers on the tributary area for a purlin, it is treated as a basic stochastic variable. Therefore, if there is no information about the distribution of workers on the frames, i.e. congregational habits of workers, the uncertainty for this basic variable is very large. Question 7 is concerned with limiting this uncertainty through the knowledge of experts. The experts have knowledge on this due to the fact that they have on-site experience, and their recollection of instances where such congregation has taken place is valuable. The experts were of the opinion that during the procedure of spreading the cladding over the purlins, the workers are distributed evenly over the full length of the frame, but that they may congregate on a small area in case of an emergency or for whatever other reason they have. Question 7 is to be included in the final questionnaire. |
| 8. Is additional support provided for the stacking of roof cladding? | The stacking of roof cladding is an important mechanism in producing the imposed roof load. Questions 8 to 10 are concerned with establishing the extent to which stacking of cladding imposes loads on the structure. The experts were all of the opinion that no additional support is provided for the stacking of cladding, which renders it being modelled as an imposed roof load. Question 8 is to be included in the final questionnaire. During the discussions with steel- and roofing contractors that followed from Question 9, it was obvious that these experts' answers were initially not entirely honest in the sense that they provided the "politically correct" answer which is that no over-stacking occurs, as this constitutes good building practice. It had to be stressed that the purpose of the survey is not to scrutinise the expert, but to obtain his expert opinion. This again emphasises the fact that there must be verbal contact between the expert and the surveyor during the interview. Through question 10, it was determined that the height of the sheets is governed by the amount of bays that the stack of sheets is to cover. This is in accordance with the assumption made in Section 3.5.3. |
| 9. Are there any predetermined positions where sheeting is stacked? For example directly over the frame or on the purlins? | |
| 10. How high are the sheets stacked? Is there any predetermined height or does it vary for different types of cladding? What roof area do you normally cover per stack of sheeting? | |
| 11. How many bays' cladding would normally be stacked on one frame? Provide your best estimate and 90% - confidence interval. | The same commentary and conclusions as for Questions 1 and 2 apply to Questions 11 and 12. |
| 12. How many bays' cladding is the maximum amount that is stacked on one frame? Provide your best estimate and 90% - confidence interval. | |
| 13. Do you use any equipment in excess of 20 kg that is to be supported by the frame or the purlins during construction? | Question 13 is primarily directed at steel- and roofing contractors and is concerned with limiting the uncertainty that exists for these load mechanisms. The steel- and roofing contractors were of the opinion that no equipment in excess of 20kg is used. After this initial consultation it may be anticipated that the answers to question 13 would be the same for all the building and roofing contractor experts. Nevertheless these questions are important since it would establish that there is low uncertainty regarding these load mechanisms, and should be included in the final questionnaire. |
| 14. When doing roof maintenance, cleaning and repair do the same number of workers and weight of equipment apply as during construction? How is the number of workers on the roof determined? | The experts were of the opinion that the number of workers is substantially less and that no heavy equipment is used. Question 14 needs to be re-structured so that the information needed to probabilistically model the imposed roof load due to maintenance is obtained in the same fashion as it is done for the imposed roof load due to construction activities on the roof. |

A general observation that was made during the preliminary consultation regarding the quantitative questions is that some of the experts could not provide 90%-confidence intervals or best estimates for some of the questions, as they maintained that they did not have sufficient knowledge. They were then urged to provide an upper and lower limit for the given variable which would represent the range of values in their opinion for which there is zero probability that the true value of the variable would fall outside. The wider the range the more uncertain the expert is about the variable. The reason for obtaining the expert opinion in these cases in such a format will become clear in Section 4.

Through the preliminary consultation it also became evident that some issues regarding the limitation of uncertainty are only applicable to steel- and roofing contractors. Questions relating to these issues are only put forward to steel- and roofing contractors in the final questionnaire and not to engineers.

The information obtained and the conclusions drawn from the preliminary consultation sessions are implemented in formulating the final questionnaire to be put forward to the experts. This process also involves elimination and addition of certain questions, as well as fine-tuning of others.

## 3.7   The Final Questionnaire

Subsequent to the preliminary consultation sessions, the final questionnaire is developed.  The questions are structured in such a manner that the data obtained from the experts is relevant to the design of the experiment.

The questionnaire consists of two parts:

1.   An introductory part.

   The expert is given background information into the scope and purpose of the survey as well as a brief lecture in calibrating his/her uncertainty and on how the questions should be answered.  This part is necessary as a means of training the expert in order to obtain better results.

2.   A questioning part.

   This is the section where the actual survey takes place, therefore the expert's opinion is obtained.  Answering each question would be an interactive process since there would be verbal contact between the expert and the surveyor.  A certain amount of training is also done when each question is presented to the expert.

Certain sections of the questionnaire only apply to steel- and roofing contractors, whilst others only apply to structural engineers.  The font of the sections that only apply to engineers are denoted as follows: `section applying to engineers.` The font of the sections that only apply to steel- and roofing contractors are denoted as follows:  section applying to steel- and roofing contractors.  Commentary on questions by the surveyor, which does not form part of the survey, is denoted as follows: *<commentary by the surveyor>*.

The final questionnaire is presented in Table 16.

## Table 16. The Final Questionnaire

*Survey on Imposed Roof Loads of Light Industrial Steel Buildings*

Firstly I would like to thank you for participating in this survey on imposed roof loads of light industrial steel buildings. Your opinion and expertise will make an important contribution to the success of the survey.

The purpose of this survey is to evaluate the magnitude of imposed roof loads, as well as the possible mechanisms involved with producing the imposed roof load. This includes loads that occur on the roof as a result of construction and maintenance activities such as installation of roof cladding and cleaning and repair operations, which is why you have been selected to take part in this survey. The scope of this survey in terms of the type of industrial building to be considered is limited to single storey steel frame or truss structures.

I would like to emphasise the fact that you should attempt to keep you answers as general as possible. This means that you are not to provide answers that are only applicable to you and your company policy, but rather let your answers be governed by what you have encountered in your experience in the field and the knowledge you have of certain relevant instances.

It is also important that you understand that the intention of this survey is not to examine you and your methods, but simply to utilise your expertise in the relevant fields. In other words, if you or your company or others you know of apply methods which are considered as deviations you should not omit these instances since you will not be judged in any way. Your answers should reflect what is currently happening in reality and not what should be happening.

During the questioning, it will be necessary for you to express your opinion in terms of the uncertainty you associate with both the average values and maximum values of the various loads on the roof. This uncertainty you have to express in terms of your 90% - confidence bounds. The 90% - confidence bounds is best explained through an example: Say, for instance you were to estimate the age of a stranger. Judging by his appearance you would have a best estimate of his age – say 40 years, but you would not be certain of this. However there would be a minimum and maximum value for your estimate of his age for which you are 90% certain his true age would fall between – for instance you would be 90% certain that he is between 33 and 48 years old. It is these upper and lower limits for your uncertainty that will be required from you.

### Question 1: Number of Workers on the Frames during Installation of the Roof Sheeting

It is required that you provide your answers in terms of the number of workers per spanning meter of the frame. If you are uncomfortable with providing your answers in this manner, provide the number of workers for the following three frame spans: 15m, 30m and 60m.

(a) What is the average number of workers that would occur on one frame during the installation of the roof sheeting? This number constitutes the average value over a number of buildings, and therefore does not have to be an integer. Provide your best estimate as well as the minimum and maximum values of the interval for which you are 90% confident that it contains the true average value.

(b) What value for the number of workers on a frame is large enough that there is only a 5% chance that this value will be exceeded. This means that in no more than 1 in 20 occasions would there be more workers on a frame than this value. Simply put, what is the maximum number of workers that can occur on one frame. Provide your best estimate as well as the minimum and maximum values of the interval for which you are 90% confident that it contains the true maximum value.

### Question 2: Number of Workers on the Purlins during the Installation of Roof Sheeting

(a) What is the average number of workers that would occur on one purlin between the frames during the installation of the roof sheeting? This number constitutes the average value over a number of buildings, and therefore does not have to be an integer. Provide your best estimate as well as the minimum and maximum values of the interval for which you are 90% confident that it contains the true average value.

(b) What value for the number of workers on a purlin is large enough that there is only a 5% chance that this value will be exceeded. This means that no more than 1 in every 20 buildings would have more workers on a purlin than this value. Simply put, what is the maximum number of workers that can occur on one purlin. This number does not have to be an integer since load-sharing will occur for workers between two purlins. Provide your best estimate as well as the minimum and maximum values of the interval for which you are 90% confident that it contains the true maximum value.

(c) Does it happen that workers congregate on a small area, and if so what are the reasons for this?

## Table 16. Continued

**Question 3: Stacking of Roof Cladding**

It is required that you provide your answer in terms of the number of bay's cladding stacked on one frame prior to installation of the cladding.

(a) What is the average number of bay's cladding that is stacked on one frame. This number constitutes the average value over a number of buildings, and therefore does not have to be an integer. Provide your best estimate as well as the minimum and maximum values of the interval for which you are 90% confident that it contains the true average value.

(b) What value for the number of bay's cladding stacked on one frame is large enough that there is only a 5% chance that this value will be exceeded. This means that no more than 1 in every 20 buildings would have more bay's cladding stacked on one frame than this value. Simply put, what is the maximum number of bay's cladding that could be stacked on one frame? Provide your best estimate as well as the minimum and maximum values of the interval for which you are 90% confident that it contains the true maximum value.

**Question 4: Number of Workers on the Frames during Maintenance, Cleaning and Repair Operations**

It is required that you provide your answers in terms of the number of workers per spanning metre of the frame. If you are uncomfortable with providing your answers in this manner, provide the number of workers for the following frame spans: 15m, 30m and 60m.

(a) What value for the number of workers on a frame is large enough that there is only a 5% chance that this value will be exceeded. This means that no more than 1 in every 20 buildings would have more workers on a frame than this value. Simply put, what is the maximum number of workers that can occur on one frame during maintenance and cleaning and repair operations. Provide your best estimate as well as the minimum and maximum values of the interval for which you are 90% confident that it contains the true maximum value.

**Question 5: Number of Workers on the Purlins during Maintenance and Cleaning and Repair Operations.**

(a) What value for the number of workers on a purlin is large enough that there is only a 5% chance that this value will be exceeded. This means that no more than 1 in every 20 buildings would have more workers on a purlin than this value. Simply put, what is the maximum number of workers that can occur on one purlin during maintenance and cleaning and repair operations. This number does not have to be an integer since load-sharing will occur for workers between two purlins. Provide your best estimate as well as the minimum and maximum values of the interval for which you are 90% confident that it contains the true maximum value.

(b) Does it happen that workers congregate on a small area during maintenance and cleaning and repair operations, and, if so, what are the reasons for this?

**Question 6: General Aspects regarding Loads due to Construction and Maintenance**

(a) Do you provide any additional support for the weight of stacked materials and workmen?

(b) Is there any heavy equipment involved in erecting the roof sheeting or during maintenance and cleaning and repair operations. Heavy equipment being too heavy for a single workman to handle, in excess of 10 kg.

(c) Are you aware of any roof sheeting companies that use methods that can be regarded as highly non-conservative.

(d) Are there any other operations on the roof causing significant roof loads?

**Table 16. Continued**

| |
|---|
| **Question 6: General Aspects regarding Loads due to Construction and Maintenance** |
| (a) Do you make extra provision for the load due to services suspended from the roof over and above the SABS's prescribed load? |
| (b) Do you treat the weight resulting from services as dead or imposed load? |
| (c) Are there any special cases such as equipment suspended from the roof or workmen involved in installing such equipment, which produce significant imposed roof loads and should be provided for by the SABS prescribed imposed roof load? |
| (d) What is the engineers' responsibility in terms of occupancy / functional changes during the lifetime of the structure? Does the engineer need to design the structure for possible future changes? |
| (e) What is the reason for the overseas' loading codes having a higher prescribed imposed roof load than the SABS? |
| (f) Do you think the current SABS imposed roof load is too conservative? |

| |
|---|
| *<Depending on the response from the experts an alternative question (question 7) to questions 1-5 may be asked. See Section 4.3.1 for clarification>* |
| **Question 7: Alternative Question to Questions 1 – 5.** |
| (a) What is the range of values for which you can confidently say that the true value would fall within, i.e. the range of values for which there is zero probability that the true value would fall outside? |

## 3.7.1 Evaluation of the Questionnaire

It is necessary to evaluate the questionnaire in terms of whether it satisfies all the prescriptions set forth by *Cooke*, for being a scientific study aimed at reaching rational consensus:

- The first principle to be adhered to is that of *reproducibility,* i.e. would it be possible for scientific peers to review and reproduce the experiment as well as all calculations? This certainly holds true for questions 1 to 5.

- The second principle to be adhered to is that of *accountability,* i.e. the source of the expert subjective estimates must be identified. This is accomplished through obtaining the expert's name, professional status and qualifications prior to the interview.

- The third principle to be adhered to is that of *empirical control,* i.e the experts' probability assessments must in principle be susceptible to empirical control. In other words, it must be possible in principle to evaluate expert probabilistic opinion on the basis of possible observations. This is done through field investigations where the outcomes or realisations of the seed variables are

observed and documented. This will then be measured against the expert's opinions on them so as to attribute weights to their opinions, as explained earlier.

- The fourth principle to be adhered to is that of *Neutrality,* i.e. the method for combining/evaluating expert opinion should encourage experts to state their true opinions. An example of disobeying this principle is where the experts rate themselves on how good an expert they think they are by assigning self-weights to their opinions. COOKE (1991) states that the use of self-weights makes it very difficult to satisfy the principles of reproducibility and accountability. With reference to the final questionnaire, it is evident that no self-weighting system is applied.

- The fifth and final principle to be adhered to is that of *Fairness*, i.e. all experts should be treated equally, prior to processing the results of observations. Since empirical control is acknowledged as the means for evaluating expert opinions, in the absence of any empirical data there is no reason for preferring one expert to another. This again underlines the fact that an objective empirical method must be used in order to assign relative weights to the expert's opinions. Assessment of the reliability of a given expert by the analyst is certainly not a scientific method of combining expert opinions as it relies heavily on the subjective opinion of the analyst which is not the same for all analysts and therefore the experiment is not reproducible. Of course, the analyst must "prefer" one expert to another when he decides which experts to consult. That is why a minimum of 5 years experience is imposed on the engineers to take part in the survey. This provides an unambiguous rationale for how suitable experts are elicited.

Another matter to take note of is that of heuristics and biases. When called upon to estimate probabilities or determine degrees of belief, people do not usually perform mental calculations, but rely instead on various rules of thumb. Such rules are called heuristics, which may lead to predictable errors or biases. *Cooke* describes four heuristics that should be accounted for in the survey and the questionnaire is subsequently evaluated in terms of these heuristics:

1. Availability

   This is where experts provide their estimates of a certain variable based on the ease with which observations (realisations) of the variable can be retrieved from memory. An example of an unwanted bias resulting from this heuristic would be where people are asked to estimate the probabilities of death from various

causes where they would typically overestimate the risks of "glamorous" and "well-publicised" causes (shark attack, tornadoes) and underestimate "unglamorous" causes (stomach cancer, heart disease). This type of heuristic is not relevant to the study and the resulting bias is hence ignored.

2. Anchoring

When asked to provide an estimate for a certain variable, subjects sometimes will fix on an initial value and then "adjust" or "correct" this value. Thus it is important that during the interview the expert is not given any "examples" of imposed roof load quantities since this would encourage him/her to anchor his/her estimates to these values. As can be seen there are no such "examples" given in the questionnaire. The fact that quantitative values are elicited from the experts in terms of the load mechanisms and not UDL's also contributes to the experts (especially engineers) not fixing their opinions to existing codified UDL's.

3. Representativeness

When asked to judge the conditional probability p(A|B) that event A occurs given that B has occurred, subjects seem to rely on an assessment of the degree of similarity between events A and B. This type of heuristic is not relevant to the study and the resulting bias is hence ignored.

4. Control

People tend to act as if they can influence situations over which they have no control whatsoever. This heuristic may become relevant where roofing contractors are asked how many bays' cladding are stacked on one frame. Since the contractor would feel that he can control this and that he would not stack more than one bay's cladding on a frame, this would be his answer. However, he does not always have control, nor does he speak for all roofing contractors. This is brought to his attention in the introductory part of the questionnaire where it is stated that his answers should not be dictated by his or his company's policy, but rather by what he has encountered during his experience in the field.

In summary, the scientific use of expert opinion is ensured through implementing the following steps:

- Selection of experts who's expertise is relevant to this experiment.

- Conducting a preliminary survey in order to adjust and optimise the questionnaire for the full survey.
- Allowing the experts to express their confidence in their own answers through carefully constructed questions.
- Combining the various experts' opinions in a manner that allows for a degree of empirical control.

## 3.8 Conducting the Survey

The expert survey is conducted to obtain the experts' opinions on the seed and maximum questions, and the philosophy-of-design questions. Furthermore, a construction site survey is conducted to obtain the data necessary to model the seed questions.

## 3.8.1 Expert Survey

The survey was conducted amongst 31 experts. The experts' names, their professional status and the number of years experience they have are presented in Table 17. The minimum number of years of experience the experts had to have in the relevant fields is 5 years, as decided earlier. The quantitative expert opinions obtained through the survey, i.e. the seed and maximum variables, are set forth in Appendix C, whilst the philosophy-of-design opinions are presented in Appendix D.

At this stage it is stated that the expert opinions on the philosphy-of-design questions were generally the same as for the preliminary consultation and therefore the assumptions made on the design of the experiment are still valid and are to be implemented in the subsequent sections.

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**Table 17. Experts who took part in the Expert Survey**

| Expert | Professional Status | Years Experience |
|---|---|---|
| H Loubscher | Pr Civil Eng | 27 |
| IP de Villiers | Pr Civil Eng | 29 |
| PJ de Villiers | Pr Civil Eng | 28 |
| F Heyman | Pr Civil Eng | 49 |
| W Hugo | Pr Civil Eng | 12 |
| G Bastiaanse | Pr Civil Eng | 7 |
| W Jordaan | Pr Civil Eng | 24 |
| G Adema | Pr Civil Eng | 20 |
| A Davis | Pr Civil Eng | 25 |
| W Kleinhans | Pr Civil Eng | 23 |
| A Eckermans | Pr Civil Eng | 21 |
| F van Zyl | Pr Civil Eng | 29 |
| A Ellmer | Pr Civil Eng | 29 |
| E Houting | Pr Civil Eng | 22 |
| P Storey | Pr Civil Eng | 20 |
| D Payne | Roofing contractor | 18 |
| A Loynes | Roofing contractor | 20 |
| J Jacobs | Roofing contractor | 15 |
| G McNeil | Roofing contractor | 20 |
| J van Breda | Roofing contractor | 12 |
| C Eksteen | Roofing contractor | 17 |
| I Gillmore | Steel contractor | 25 |
| D Scott | Steel contractor | 27 |
| C Lutzeller | Steel contractor | 17 |
| A Kilpin | Steel contractor | 16 |
| G Lackey | Steel contractor | 13 |
| M Papanicolau | Steel contractor | 16 |
| W du Plessis | Steel contractor | 22 |
| Foreman #1 | Site Foreman | 10 |
| Foreman #2 | Site Foreman | 15 |
| Foreman #3 | Site Foreman | 12 |

Note that, in addition to engineers, steel- and roofing contractors, three site foremen who were encountered during the site survey (and who were available to be questioned) were also surveyed. Due to the weighted combination of expert opinion being used, the fact that an additional "type" of expert is being surveyed does not influence the outcome of the experiment.

### 3.8.2 Construction Site Survey

The realisations of the seed variables were documented through conducting a construction site survey. The three seed variables are:

1. The average number of workers on a rafter during the installation of the sheeting.
2. The average number of workers on a purlin during the installation of the sheeting.
3. The average number of bays' cladding stacked on 1 rafter during the installation of the sheeting.

For all three the seed variables a sample distribution is obtained by observing the quantity of each variable at all the sites. The sites were identified by the experts partaking in the survey. The surveyor visited the sites during the period of erection of the roof sheeting and spent an hour on site documenting the observations for the three seed variables. Although the surveyor was not present on site for the full duration of the construction phase, it is assumed that the time spent is sufficient to identify the physical procedure and routine of installing the cladding. Table 18 summarises the various characteristics of the construction sites involved in the survey. The values for the seed variables as observed by the surveyor are set forth in Section 4.2.2.

**Table 18. Characteristics of the Construction Sites used in the Survey**

| Site number | Site Location | Plan Dimensions | Height of Eaves (m) | Type of Roof | Spacing of Frames (m) | Spacing of Purlins (m) | Roof Angle (°) |
|---|---|---|---|---|---|---|---|
| 1 | Table View, Cape Town | 20 x 40 m | 7 | Beam | 4.5 | 1.8 | 10 |
| 2 | Table View, Cape Town | 30 x 50 m | 7 | Truss | 5.5 | 1.7 | 8 |
| 3 | Paarden Island, Cape Town | 15 x 30 m | 5 | Beam | 4 | 1.7 | 10 |
| 4 | Centurion, Pretoria | 180 x 180 m | 17 | Truss | 5.4 | 1.6 | 3 |
| 5 | Centurion, Pretoria | 50 x 50 m | 12 | Truss | 5 | 1.6 | 9 |
| 6 | Stikland, Cape Town | 20 x 40 m | 7 | Truss | 5 | 1.7 | 12 |
| 7 | Somerset West, Cape Town | 30 x 42 m | 5 | Truss | 6 | 1.6 | 11 |
| 8 | Somerset West, Cape Town | 35 x 110 m | 5.5 | Truss | 5.5 | 1.6 | 10 |
| 9 | Somerset West, Cape Town | 15 x 32 m | 4 | Beam | 4 | 1.8 | 9 |
| 10 | Stikland, Cape Town | 17 x 21 m | 4 | Beam | 5 | 1.7 | 12 |
| 11 | Stikland, Cape Town | 3 x 12 x 30 m | 5 | Beam | 5 | 1.6 | 10 |
| 12 | Stikland, Cape Town | 25 x 44 m | 6 | Truss | 5.5 | 1.7 | 8 |
| 13 | Stikland, Cape Town | 13 x 28 | 3.5 | Beam | 4 | 1.8 | 8 |
| 14 | Stikland, Cape Town | 17 x 30m | 5 | Beam | 4.5 | 1.8 | 8 |
| | Average = | | 5.8 | | 4.9 | 1.7 | 9.6 |
| | Maximum = | | 12 (17) | | 5.5 | 1.8 | 12 |
| | Minimum = | | 3.6 | | 4 | 1.6 | 8 (3) |

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### 3.9 Information obtained through the Expert- and Construction Site Surveys

Subsequent to the surveys conducted, the following information on the load mechanisms of the imposed roof load has been obtained (as shown in Figure 12).
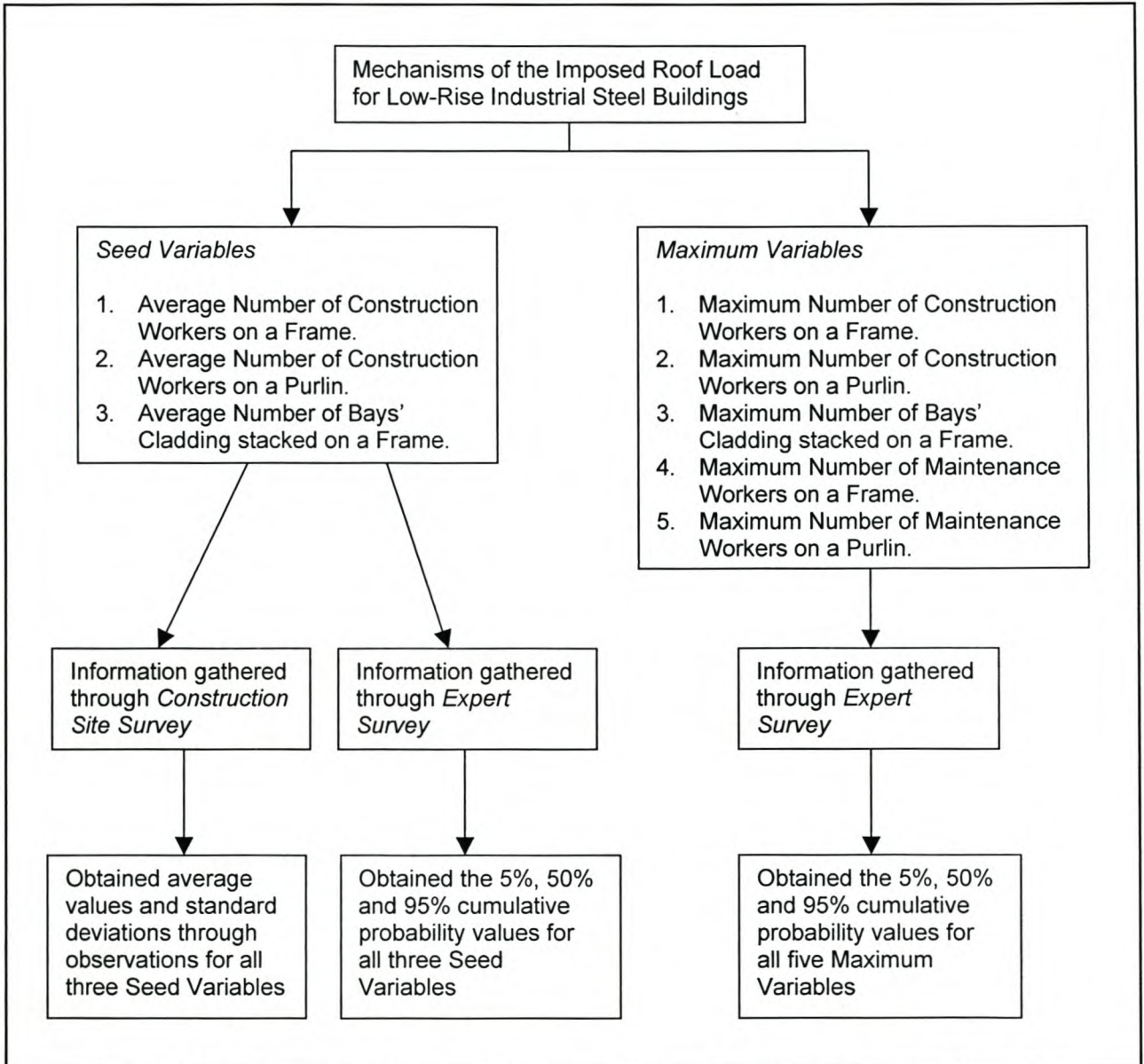


**Figure 12. Information Obtained through the Expert- and Construction Site Surveys**

## CHAPTER 4: COMBINING EXPERT OPINION THAT ALLOWS FOR EMPIRICAL CONTROL

Calibration of the experts represents a form of empirical control on subjective probability assessments. The expert opinion was elicited in a suitable format for the Classical Method as proposed by *Cooke* to be used in the calibrating procedure. Certain modifications brought to the Classical Method by TER HAAR and RETIEF (1997) are also incorporated. The objective of this section is to set out the principles of the Classical Method which were implemented, as well as further enhancements to the model brought about by the analyst. A spreadsheet programme, EXCAL, has been developed for calibration of the experts and for obtaining the combined opinion (see Appendix E).

As explained earlier, the quality of a certain experts' quantitative opinion can be measured by comparing it to the values of realisations of the seed variables. According to the amount of correlation that exists between the experts' opinion and the observed three seed variables, a weight is calculated for each expert. By normalising these weights the quality of a certain experts' opinion is put into perspective with those of the other experts. The normalised weight, $w_N(e)$, for expert e is:

$$w_N(e) = \frac{w_U(e)}{\sum_E w_U(e)} \qquad (6)$$

where $w_u(e)$ = unnormalised weight for expert e.

E = total number of experts.

The set of normalised weights can be used to find a combined opinion for a certain survey variable, using the opinions of all the experts. The weighed opinion of all the experts is called the opinion of the Decision-Maker (DM). For a variable X, the opinion of DM is:

$$\text{Opinion of } DM_X = \sum_E w_N(e) \times (\text{opinion of expert } e) \qquad (7)$$

Obtaining the opinion of the DM for the seed variables, provides the opportunity to measure the DM opinion against the realisations of the seed variables, i.e. the DM can be treated as any other expert. If the DM opinion performs better (i.e. obtains a

higher weight) than the best expert as well as the average of the expert opinions, it is argued that the weighted combination of expert opinions for the maximum or unknown variables will also perform better. For this to hold true it is necessary that the seed and maximum variables share a common field of expertise, and as can be seen from Section 3.4.1 this is satisfied by the seed variables so chosen for this survey.

## 4.1 Expert Opinion Measurement Methodology

Recall that through the expert survey, the 5%, 50% and 95% cumulative probability values for the seed and maximum variables have been elicited from the experts. These values are subsequently referred to as the $x_1$, $x_2$ and $x_3$ - values respectively. The purpose of this section is to set forth the methodology implemented in determining the combined $x_1$, $x_2$ and $x_3$ for the maximum variables, thereby defining a probability distribution for each of the maximum variables. As explained earlier, this involves attributing weights to the experts whereby their combined opinion may be found. The logic of this process, as well as a description of the various concepts and parameters involved, is set forth in Figure 13. Figure 13 is to serve as a reference map for the reader to interpret the various sub-components of the calibration process in terms of their roles in the global context of the experiment.
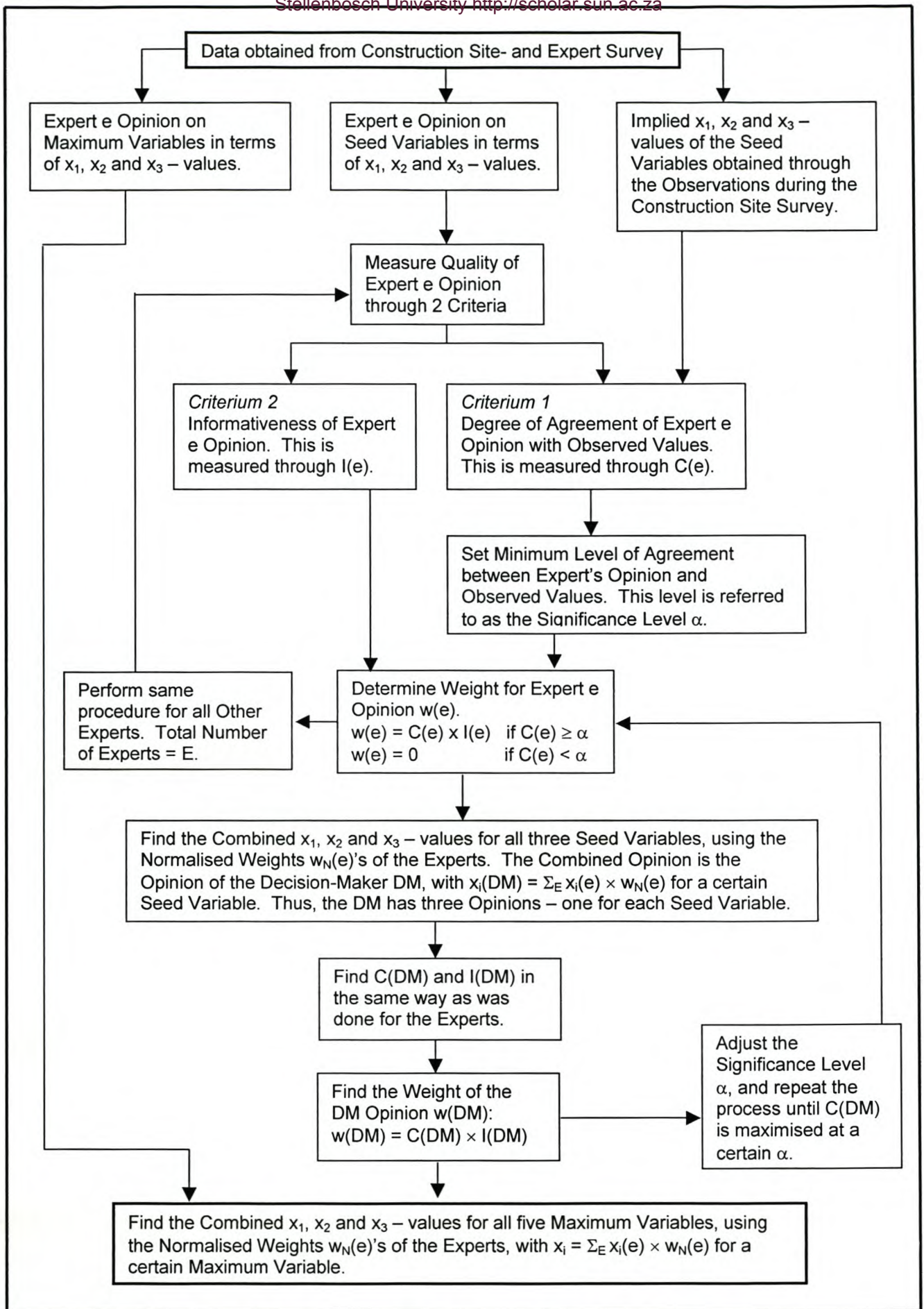
**Figure 13. Summary of the Expert Opinion Combination Methodology.**

Figure 13 shows that the first step towards calibration of the experiment is to measure the quality of the expert opinion for the seed variables. The $x_1$, $x_2$ and $x_3$ - values obtained from expert e implies a probability distribution for a certain seed variable X. To make this probability distribution finite, an upper and lower limit for the range of values of the seed variable have to be established – they are the $x_0$ and $x_4$ - values respectively. The range of values between $x_0$ and $x_4$ represents all the possible values that seed variable X may assume and is referred to as the *intrinsic range* of the variable. The $x_0$ and $x_4$ - values are determined as follows:

$$x_0 \quad = \min(x_1) - 0.1 \times [\max(x_3) - \min(x_1)] \qquad (8a)$$

$$x_4 \quad = \max(x_3) + 0.1 \times [\max(x_3) - \min(x_1)] \qquad (8b)$$

where $\min(x_1)$ = minimum of the $x_1$ – values elicited from the E experts

$\max(x_3)$ = the maximum of the $x_3$ – values elicited from the E experts

As is evident, $x_0$ and $x_4$ are dependent on the percentage increase and decrease of $x_3$ and $x_1$ respectively. For Equations (8a) and (8b) this percentage = 10%. It will subsequently be shown that the calibration process is insensitive to the magnitude of this percentage (see Section 4.3.4).

The probability distribution of expert e for seed variable X is defined through four probability bins as shown in Figure 14:



P1 = P4 = probability that a realisation occurs within Bins 1 or 4 = 5%
P2 = P3 = probability that a realisation occurs within Bins 2 or 3 = 45%
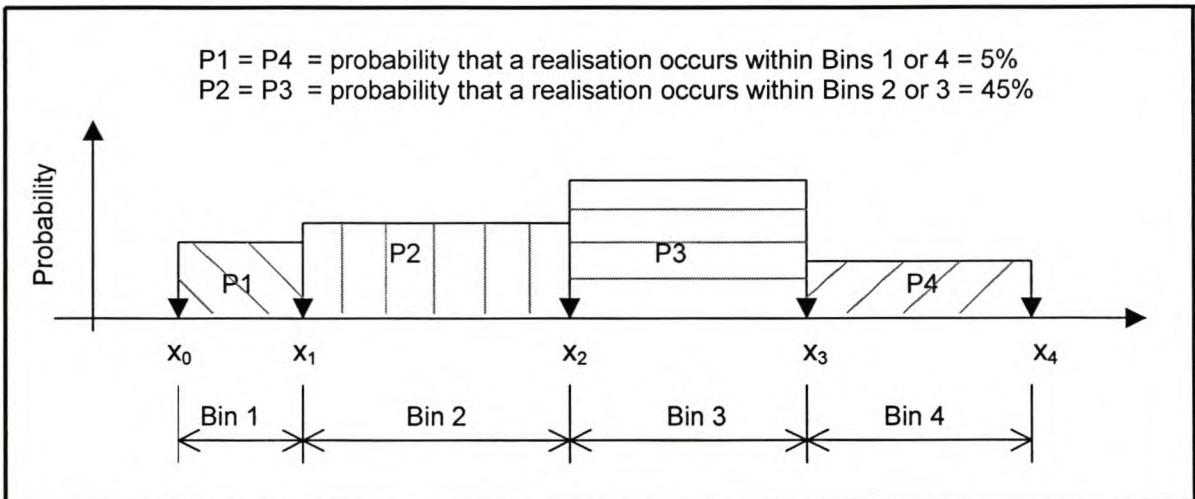
**Figure 14. Expert's Probability Distribution for a certain Seed Variable X.**

Note that the probability density function has a constant value for a certain bin.

The quality of the opinion of expert e can now be measured using the implied distribution shown in Figure 14. This measurement is done according to two criteria as shown in Figure 13:

1. How well do the experts' distributions of the seed variables correlate with the true, or observed, distributions of the seed variables?

2. How informative is the experts' opinion? The least informative would be a uniform distribution over the intrinsic range of the variable, representing high uncertainty. The experts' implied distribution can be measured against this uniform distribution.

### 4.1.1 Criterium 1: Agreement of Expert Opinion with Observed Values

The four probability bins defined in Figure 14 provide the basis for comparing expert opinion to the known seed variables. Let the experts' assessment of the seed variable be P, and S a sample distribution generated by N samples (the realisations for the N seed variables) from the experts' distribution, then the *relative information* of S with respect to P is given by:

$$I(S,P) = \sum_{i=1}^{r} S_i \ln\left(\frac{S_i}{P_i}\right)$$ 

(9)

where $I(S,P)$ = relative information

$S_i$ = number of hits in bin i / number of seed variables

$P_i$ = theoretical probability for bin i

r = number of bins

This part of the Classical Method as proposed by COOKE (1991) is modified for application to this study. First the model as proposed by COOKE is presented in its purest form and then the relevant changes are discussed.

### 4.1.1.1 Classical Method as proposed by COOKE

As proposed by COOKE, Bin i is awarded exactly one hit when the realisation of a seed variable falls within that bin. The realisation of a seed variable is treated as a deterministic value with no uncertainty involved. $S_i$ in Equation (9) represents the "known" or "true" probability of Bin i over the number of seed variables as observed

through the realisations of the seed variables. Put in other words, $S_i$ represents the true probability that a seed variable has an outcome or realisation falling within Bin i. With $P_i$ representing the experts' probability of an outcome in Bin i, $S_i$ and $P_i$ are compared through $\ln(S_i/P_i)$. Multiplying this with $S_i$ (thereby using $S_i$ as an importance or weight factor for Bin i) and adding it for all four bins yields $I(S,P)$, which is referred to as the relative information of S with respect to P. $I(S,P) = 0$ only if $S_i = P_i$ for all i, that is only if the probability distributions S and P are identical. Thus, the lower the value of $I(S,P)$ the better the experts' distribution correlates with the observed distribution of the seed variables.

Note the influence of the number of seed variables on this exercise. For three seed variables, the possible values of $S_i$ are 0, 0.33, 0.67 and 1. Knowing that the values of the probability bins ($P_i$'s) are either 0.05 or 0.45, such few seed variables does not allow for accurate comparison of $S_i$ to $P_i$. An increased number of seed variables would solve this problem, however, the number of seed variables are restricted by the fact that they have to share a common field of expertise with the maximum variables.

Another area of concern is this: If a realisation or hit were to occur at the boundary of two bins, to which bin would the hit be allocated? Furthermore, the closer a hit falls to the boundary of two bins, the more "unfair" it's considered to award the full hit to the bin in which it occurs and not allocate a certain portion of the hit to the adjacent bin.

The manner in which the above two problems are dealt with is explained in the subsequent section.

### 4.1.1.2 Alterations to the Classical Method for Application to this Experiment

The statistics obtained on the seed variables through the site surveys allow for the observed seed variables to be modelled probabilistically. The lognormal-distribution is used to represent the seed variables and the effect of this choice of distribution on the experiment is measured in Section 4.3.3. Obtaining the first two moments now allows for the observed seed variable to be modelled as in Figure 15.
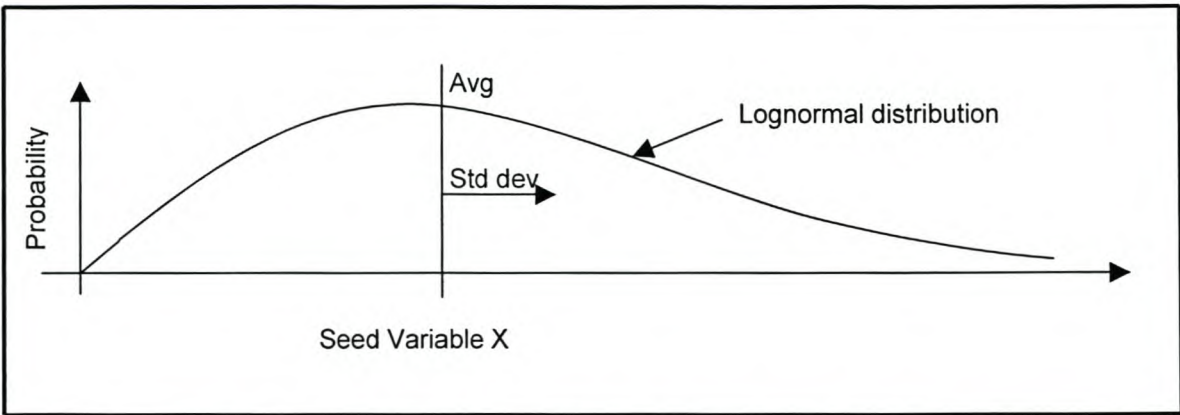
**Figure 15. Probabilistic Model for the Observed Seed Variables**

In this manner uncertainty is introduced into the observed values or realisations of the seed variables. The distribution function shown in Figure 15 represents one hit from a realisation of a seed variable. The allocation of the hit to the four probability bins is illustrated through superimposing Figure 14 on Figure 15.



$F_X(x)$ = the cumulative probability of X at x
$P1 = F_X(x_1)$
$P2 = F_X(x_2) - F_X(x_1)$
$P3 = F_X(x_3) - F_X(x_2)$
$P4 = 1 - F_X(x_3)$
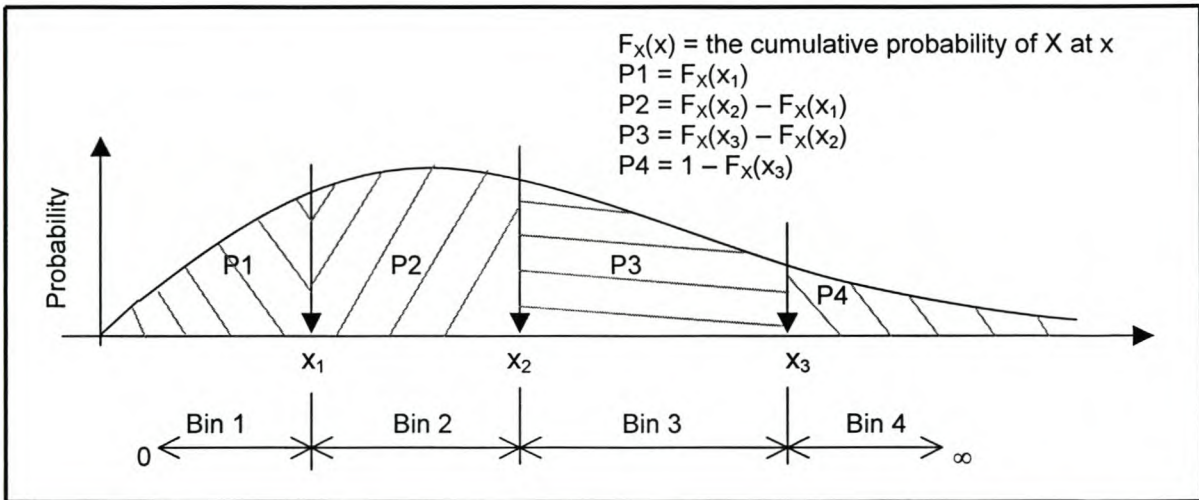
**Figure 16. Proportioning of Observed Seed Variable into four Probability Bins**

The proportion of the hit allocated to Bins 1,2,3 and 4 are the probability areas P1, P2, P3 and P4 respectively, with P1+P2+P3+P4 = 1. If for expert e, the $x_1$, $x_2$ and $x_3$ - values for each seed variable are such that P1, P2, P3, and P4 have the values 0.05, 0.45, 0.45 and 0.05 respectively, then the relative information I(S,P) for expert e would be zero. If the realisations were to be deterministic as proposed by COOKE with only one hit per bin it would not be possible for I(S,P) to be zero. Therefore, this alternative method effectively measures the expert's distribution against the observed distribution of the seed variables. The sensitivity of the calibration of experts with

respect to the allocation of hits that occur on the boundary of two bins is therefore eliminated through the probabilistic modelling of the seed variables.

In summary, the probabilistic modelling of the realisations reduces the discretisation effect due to the limited number of probability bins and seed variables.

From this point forward the Classical Method as proposed by COOKE in its unaltered form is again applied to the calibration procedure.

The next step is to decide whether the opinion of expert e agrees sufficiently with the observed values for the seed variables to be included in establishing the opinion of the Decision-Maker DM.

The hypothesis $H_0$ is now tested against the alternative hypothesis $H_1$ with

$H_0$: The sample distribution S (observed values) belongs to the expert's distribution P

$H_1$: The sample distribution S (observed values) does not belong to the expert's distribution P.

The above hypothesis is evaluated at a certain significance level $\alpha$. The significance level represents the minimum allowable probability that $H_0$ is true for $H_0$ to be accepted. The probability that $H_0$ is true is given by the test statistic:

$$C(e) = 1 - \chi_R^2[2NI(S,P)] \tag{10}$$

where  $C(e)$   = calibration score for expert e

$\chi_R^2$   = chi-square distribution with R degrees of freedom

R    = number of quantiles elicited (three in this case)

N    = number of seed variables (three in this case).

$C(e)$ is used subsequently as the measure of how well the experts' distribution compares with that of the observed values for the seed variables, where $C(e) = 1$ if the experts' distribution and the sample distribution (observed values) are identical.

If $C(e)$ is smaller than the chosen significance level, then it is rejected that S belongs to P. In this case an expert receives a zero weight. By increasing $\alpha$ step by step, more and more experts will receive zero weights. This will result in different DM's for each significance level and at one of these significance levels the optimum DM will be

reached. How the optimum DM is reached and at what significance level this takes place, is discussed in Section 4.2.2.

### 4.1.2 Criterium 2: Information Value of the Expert Opinion

The probability bins can also be used to measure the confidence of an expert in his/her quantitative opinion. This is done by obtaining the relative information of the expert's opinion with respect to that of a uniform distribution over the intrinsic range of the variable. The uniform distribution represents the least informative distribution. This is illustrated in Figure 17.
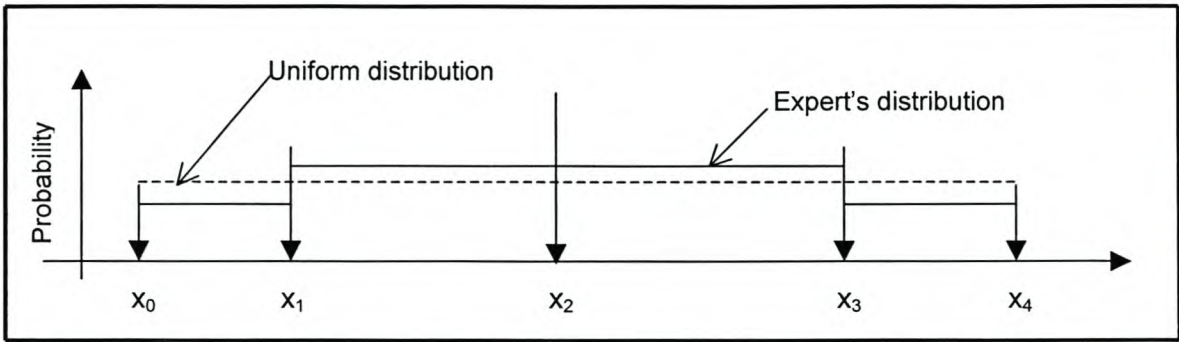


**Figure 17. Comparison of the Experts' Distribution with the Uniform Distribution**

The average relative information over the N seed variables of expert e's opinion with respect to the uniform distribution is:

$$I(e) = \frac{1}{N} \times \sum_{i=1}^{N} [\ln(x_{i4} - x_{i0}) + \sum_{r=1}^{4} P_r \ln \frac{P_r}{x_{ir} - x_{ir-1}}] \qquad (11)$$

where $x_{ir}$ = The $x_r$ – value of expert e for seed variable i

$I(e)$ is called the information score for expert e and is equal to zero if expert e's distributions are equal to the uniform distributions for all the seed variables. The more confident an expert is in his/her opinion, the more his/her distribution would tend towards a deterministic value (with zero uncertainty). An expert who has zero uncertainty in his/her opinions for all seed variables would receive an information score of $+\infty$. This is illustrated in Figure 18.
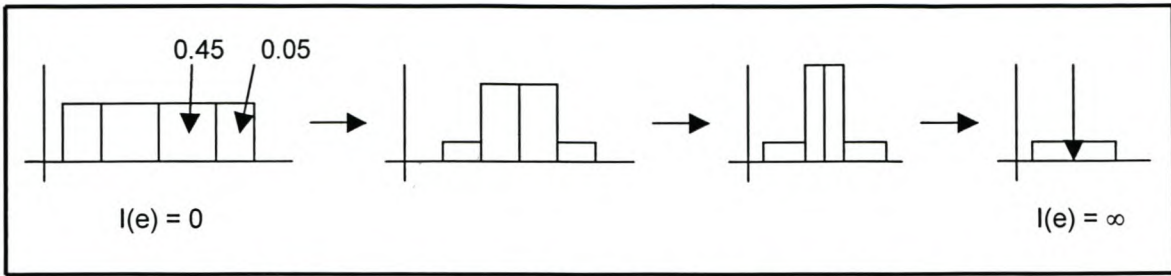
**Figure 18. Increasing Information Score as Uncertainty decreases**

### 4.1.3 The Opinion of the Decision-Maker

With C(e) and I(e) known for expert e, the non-normalised weights are calculated as follows:

$$w_U = C(e) \times I(e), \qquad \text{if } C(e) \geq \alpha \qquad (12a)$$

$$w_U = 0, \qquad \text{if } C(e) < \alpha \qquad (12b)$$

Using the normalised weights $w_N$ obtained through Equation (6), the combined opinion of the DM can now be calculated from Equation (7). The combined $x_1$, $x_2$ and $x_3$ - values represent the opinion of the DM with:

$$x_1(DM) = \sum_E w_N(e)x_1(e) \qquad (13a)$$

$$x_2(DM) = \sum_E w_N(e)x_2(e) \qquad (13b)$$

$$x_3(DM) = \sum_E w_N(e)x_3(e) \qquad (13c)$$

### 4.1.3.1 Commentary on Combination of Expert Opinion

Say we have two independent random variables X and Y, and we want to combine them in order to obtain variable Z. If we assign equal weights to X and Y, then

$$Z = 0.5X + 0.5Y \qquad (14)$$

The probability density function for Z is obtained through the following:

$$\text{For all } z: \ p(Z = z) = \int_{-\infty}^{\infty} p(X = x) \times p\left(Y = \frac{z - 0.5x}{0.5}\right) dx \qquad (15)$$

Equation (15) can be extended to combine any number of independent variables.

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This is in contrast to what is done in combining the expert opinions to find the distribution of the DM (Equations (13a-c)). However, Equations (13a-c)) are valid since the $x_1$, $x_2$ and $x_3$ - values of the experts are perfectly correlated, i.e. $x_1$ for expert A is perfectly correlated with $x_1$ for expert B and so forth. This correlation is explained by recognising that, for example, the $x_3$ - values (and $x_1$ and $x_2$ - values) obtained from the experts represent the 95% cumulative probability values and therefore are not merely random realisations of the variables. Therefore, it can be stated that for two experts with equal weights the following applies:

$$p(Z \leq 0.5\ x_3(\text{expert A}) + 0.5\ x_3(\text{expert B})) = 0.95 \qquad (16)$$

where Z = combined distribution

Equation (16) is valid due to the fact that $x_3$ of expert A is perfectly correlated with $x_3$ of expert B.

### 4.1.3.2 Optimising the Opinion of the Decision-Maker

The opinion of the DM can now be measured against the realisations of the seed variables as shown in Figure 13. By altering the significance level $\alpha$, different DM opinions are obtained. The optimum significance level can now be selected so as to produce the optimum DM. Only the opinions of experts with calibration scores greater than or equal to the optimum $\alpha$ are used in obtaining the optimum DM opinion as implied through Equations (12a) & (12b). These experts with their respective weights can now be used in obtaining the combined opinions for the maximum variables.

It is necessary to establish a criterium that can be used to select the optimum DM opinion. Intuitively the DM with the maximum weight $W(DM)$ as determined through Equation (12a) is to be used. However, TER HAAR and RETIEF (1997) showed it to be more correct to select the optimum DM at the significance level that maximises the calibration score $C(DM)$. The calibration score measures the agreement between the DM's distributions of the seed variables and the observed distributions of the seed variables. Selecting the maximum $W(DM)$ does not guarantee that the maximum $C(DM)$ has been selected because $W(DM\ A)$ may be larger than $W(DM\ B)$ due to the relative information $I(DM\ A)$ being larger than $I(DM\ B)$. In such a scenario, selecting

DM A over DM B would mean that the larger I-value, and therefore the most confident DM, has been rewarded. This is non-conservative since it rewards over-confidence. The following criterium is used in selecting the optimum DM:

1. The optimum DM will be at the maximum C(DM).

2. If the maximum C(DM) occurs at more than one $\alpha$, select the DM at the $\alpha$ which yields the maximum I(DM).

The expert opinions on the maximum variables can now be combined using the weights that yield the optimum DM.

## 4.2 Application of the Classical Method and Alterations to the Survey on Imposed Loads for Inaccessible Roofs

In this section the assumptions and procedures which were implemented in applying the Classical Method for expert measurement to the experiment are set forth, and the results from the calibration process are presented. The theory presented in Section 4.1 is applied to the survey on imposed loads for inaccessible roofs. Therefore this section is predominantly results-orientated. Refer to Figure 13 for guidance through the subsequent sections.

### 4.2.1 Modelling of the Seed Variables

The observed realisations of the seed variables, against which the expert opinions are measured, are presented in Table 19. The following standard equations apply to calculating probability the moments of the three variables (formulae obtained from GUTTMAN and WILKS (1965)):

$$\mu = \frac{1}{N} \times \sum_{i=1}^{N} \text{observation } i \qquad (17a)$$

$$\sigma_{inherent} = \sqrt{\frac{1}{N-1} \sum_{i=1}^{N} (\text{observation } i - \mu)^2} \qquad (17b)$$

$$\sigma_{sample} = \frac{\sigma_{inherent}}{\sqrt{N}} \qquad (17c)$$

where   N   = number of observations = 14

The sample uncertainty $\sigma_{sample}$ represents the uncertainty due to the fact that the sample size is limited to N observations and therefore the mean value $\mu$ for the sample will differ from that of the entire population. In other words, the uncertainty lies in the fact that if a different sample size were used, say 20 observations (or very large), then the average value would differ from what it is now. The sample uncertainty therefore represents the uncertainty in the *average value* of the seed variable. This is exactly what is measured through the questionnaire put forward to the experts (refer to Sections 3.7 & 3.4.1) and the sample uncertainty is therefore the relevant uncertainty to be used to model the seed variables as far as the calibration of the experts is concerned.

**Table 19. Realisations of the Seed Variables**

| Site number* | Seed 1: Number of workers on a frame in terms of the equivalent number of workers for a 20m-spanning frame | Seed 2: Number of workers on a purlin in terms of the number of workers | Seed 3: Number of bays' cladding stacked on a frame |
|---|---|---|---|
| 1 | 5.00 | 1 | 1 |
| 2 | 4.67 | 2 | 1 |
| 3 | 6.67 | 1 | 1 |
| 4 | 2.22 | 3 | 3 |
| 5 | 2.00 | 2 | 2 |
| 6 | 5.00 | 2 | 1 |
| 7 | 3.33 | 1 | 1 |
| 8 | 4.57 | 2 | 1 |
| 9 | 4.00 | 1 | 1.5 |
| 10 | 3.53 | 1 | 1 |
| 11 | 3.33 | 1 | 1 |
| 12 | 4.80 | 2 | 2 |
| 13 | 4.62 | 2 | 1.5 |
| 14 | 4.67 | 2 | 1 |
| $\mu$ | 4.17 | 1.64 | 1.36 |
| $\sigma_{inherent}$ | 1.22 | 0.63 | 0.60 |
| $\sigma_{sample}$ | 0.32 | 0.17 | 0.16 |

*Refer to Table 18 for site description

With $\mu$ and $\sigma_{sample}$ known for each seed variable, together with it's selected distribution type, the realisations of the seed variables can now be modelled probabilistically. This is summarised in Table 20.

**Table 20. Probabilistic Models for the Seed Variables**

| Seed Variable | Units | Distribution | 1st moment | 2nd moment |
|---|---|---|---|---|
| 1. Construction workers on a frame* | Number of workers | Lognormal | 4.17 | 0.32 |
| 2. Construction workers on a purlin | Number of workers | Lognormal | 1.64 | 0.17 |
| 3. Cladding stacked on a frame | Number of bays | Lognormal | 1.36 | 0.16 |

*Seed variable 1 is in terms of the number of workers on a 20m-spanning frame.

The lognormal distribution function is chosen to model the seed variables. The reason is that the lognormal distribution function can only assume values larger of equal to zero. Physically, the seed variables cannot assume values smaller than zero and therefore the lognormal distribution function is appropriate. The sensitivity of the results of the experiment towards the choice of the distribution for the seed variables is examined in Section 4.3.3.

## 4.2.2 Selecting the Optimum Decision-Maker

The experts are now calibrated according to Criteria 1 & 2 in Sections 4.1.1 & 4.1.2 respectively. The individual experts' opinions on the seed variables are set forth in Table 21. The experts' opinions are combined using their respective normalised weights to find the opinion of the DM as explained in Section 4.1.3. The DM Opinion obtained is subsequently evaluated in terms of:

- The Calibration Score C(DM)
- The Relative Information I(DM)
- The Weight W(DM)
- The Proportion of the total number of experts for which $C(e) \geq \alpha$.

The DM opinion is evaluated in terms of the above properties at different significance levels ($\alpha$'s). This is shown in Figure 19. Note that in Figure 19, the DM properties are only shown at those $\alpha$'s where a change in the properties has taken place, i.e. one or more experts have "dropped out" since the previous $\alpha$. Thus, the DM properties remain the same in-between the data-points shown in Figure 19.
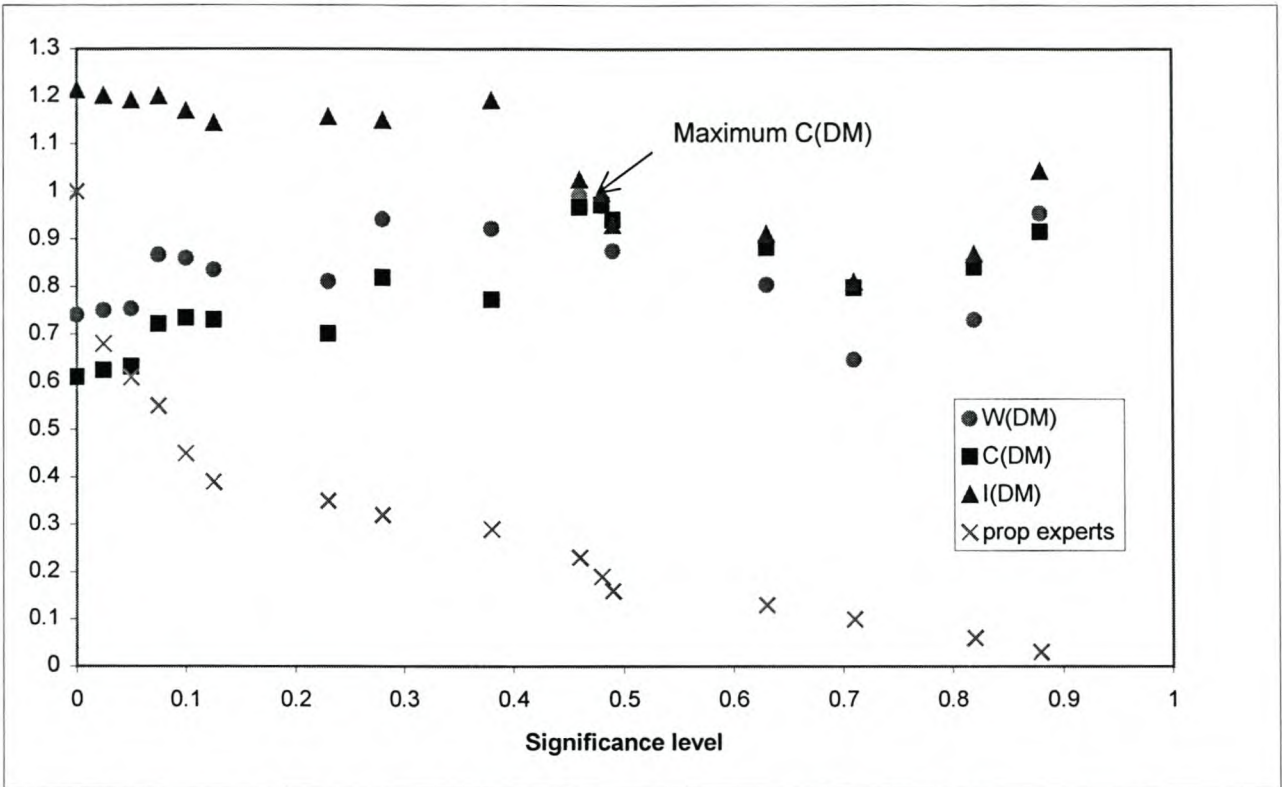
**Figure 19. Properties of the Decision-Maker at various Significance Levels**

Applying the Criterium in Section 4.1.3.2 in selecting the optimum DM, the DM at a significance level of 0.48 is selected as it is at this significance level where C(DM) is maximised. The number of experts who have non-zero weights at $\alpha = 0.48$ is 6. A significance level of 0.48 can generally be regarded as high in probabilistic terms. This means that the experts' distributions generally correlate well with the observed distributions of the seed variables.

From Figure 19 it is observed that I(DM) increases sharply for significance levels of 0.7 and upward, where the DM opinion is comprised of three or less experts' opinions. The reason for this is that the same intrinsic range, as determined for the original number of experts (31), still applies in determining the I(DM) when only three or less experts' opinions comprise the DM opinion. Recall that I(e) for a certain expert e measures the amount of "disagreement" between the expert's distribution and that of a uniform distribution over the intrinsic range, where I(e) increases as the "disagreement" increases. The amount of "disagreement" can be defined as the distance the $x_1$ and $x_3$ - values of the expert are away from the $x_0$ and $x_4$ - values respectively (see also Figure 18). When the $x_0$ and $x_4$ - values are determined by taking into account *all* the experts' opinions, a wider intrinsic range results than would be when it is determined only for those experts whose opinions comprise that

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of the DM. So, it is to be expected that the combined expert opinion of only a few experts would "disagree" more with the wider intrinsic range as determined for the original (larger) number of experts, and hence, I(DM) for this small number of experts will increase (see also Section 4.3.4).

**Table 21. Experts' Opinions on the Seed Variable**

| Type of Expert | Expert | Seed 1: Average number of construction workers on a frame | | | Seed 2: Average number of construction workers on a purlin | | | Seed 3: Average number of bays' cladding stacked on a frame | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | $x_1$ | $x_2$ | $x_3$ | $x_1$ | $x_2$ | $x_3$ | $x_1$ | $x_2$ | $x_3$ |
| Civil Engineers | H Loubscher | 3.67 | 6.67 | 9.67 | 0.99 | 1 | 1.01 | 1.05 | 1.5 | 1.95 |
| | IP de Villiers | 2.10 | 3.00 | 3.90 | 1.05 | 1.5 | 1.95 | 1.1 | 2 | 2.9 |
| | PJ de Villiers | 2.00 | 2.86 | 5.00 | 0.99 | 1 | 1.01 | 1.1 | 2 | 2.9 |
| | F Heyman | 2.40 | 6.00 | 9.60 | 1.05 | 1.5 | 1.95 | 1.05 | 1.5 | 1.95 |
| | W Hugo | 4.80 | 12.00 | 19.20 | 0.99 | 1 | 1.01 | 1.15 | 2.5 | 3.85 |
| | G Bastiaanse | 2.50 | 3.33 | 4.00 | 1 | 1.5 | 2 | 1.05 | 1.5 | 1.95 |
| | W Jordaan | 2.50 | 2.86 | 3.33 | 0.99 | 1 | 1.01 | 1.05 | 1.5 | 1.95 |
| | G Adema | 2.22 | 2.86 | 3.33 | 0.99 | 1 | 1.01 | 0.99 | 1 | 1.01 |
| | A Davis | 3.33 | 4.00 | 5.00 | 0.99 | 1 | 1.01 | 1 | 1.5 | 2 |
| | W Kleinhans | 2.86 | 3.33 | 4.00 | 0.99 | 1 | 1.01 | 1.1 | 2 | 2.9 |
| | A Eckermans | 2.50 | 3.33 | 4.00 | 0.99 | 1 | 1.01 | 1.05 | 1.5 | 1.95 |
| | F van Zyl | 1.45 | 5.50 | 9.55 | 1.15 | 2.5 | 3.85 | 1.15 | 2.5 | 3.85 |
| | A Ellmer | 2.50 | 2.86 | 3.33 | 0.99 | 1 | 1.01 | 0.99 | 1 | 1.01 |
| | E Houting | 2.08 | 5.83 | 9.58 | 1.05 | 1.5 | 1.95 | 1.1 | 2 | 2.9 |
| | P Storey | 2.90 | 11.00 | 19.10 | 1.1 | 2 | 2.9 | 1.05 | 1.5 | 1.95 |
| Roofing Contractors | D Payne | 4.95 | 5.00 | 5.05 | 0.99 | 1 | 1.01 | 0.99 | 1 | 1.01 |
| | A Loynes | 3.33 | 4.00 | 5.00 | 0.99 | 1 | 1.01 | 1 | 1.5 | 2 |
| | J Jacobs | 3.96 | 4.00 | 4.04 | 1 | 1.5 | 2 | 0.99 | 1 | 1.01 |
| | G McNeil | 8.00 | 9.00 | 10.00 | 0.99 | 1 | 1.01 | 1 | 2 | 3 |
| | J van Breda | 2.86 | 3.33 | 5.00 | 1 | 1.5 | 2 | 1 | 2 | 3 |
| | C Eksteen | 2.50 | 2.86 | 4.00 | 0.99 | 1 | 1.01 | 0.75 | 1 | 1.25 |
| Steel Contractors and Site Foremans | I Gillmore | 2.50 | 2.86 | 3.33 | 2 | 2.5 | 3 | 1 | 1.5 | 2 |
| | D Scott | 3.96 | 4.00 | 4.04 | 0.99 | 1 | 1.01 | 0.99 | 1 | 1.01 |
| | C Lutzeller | 2.22 | 2.67 | 3.33 | 0.99 | 1 | 1.01 | 1 | 1.5 | 2 |
| | A Kilpin | 2.50 | 2.67 | 2.86 | 0.99 | 1 | 1.01 | 0.99 | 1 | 1.01 |
| | G Lackey | 1.82 | 2.00 | 2.50 | 0.99 | 1 | 1.01 | 0.99 | 1 | 1.01 |
| | M Papanicolau | 2.67 | 3.33 | 3.64 | 1 | 1.5 | 2 | 1 | 1.5 | 2 |
| | W du Plessis | 3.64 | 4.00 | 4.44 | 0.99 | 1 | 1.01 | 0.99 | 1 | 1.01 |
| | Foreman #1 | 4.95 | 5.00 | 5.05 | 0.99 | 1 | 1.01 | 1 | 1.5 | 2 |
| | Foreman #2 | 4.95 | 5.00 | 5.05 | 0.99 | 1 | 1.01 | 0.99 | 1 | 1.01 |
| | Foreman #3 | 5.66 | 5.71 | 5.77 | 0.99 | 1 | 1.01 | 0.99 | 1 | 1.01 |

* Seed variable 1 is in terms of the number of workers on a 20m-spanning frame.

Note that in Table 21 there is very low uncertainty in certain seed variables as estimated by some of the experts. This is revealed for the cases where the values of $x_1$ and $x_3$ are very close to each other. The reason for this is that certain experts

provided only deterministic values for these seed variables and uncertainty had to be introduced by the analyst for the Classical Method to apply. The effect of the amount of uncertainty introduced is measured in Section 4.3.2.

The opinions of the six experts who received non-zero weights at $\alpha = 48\%$ and whose opinions therefore comprise the DM opinion are shown in Table 22. Their opinions are ranked in terms of increasing C(e) - value, in other words, in terms of how well their estimates of the seed variables compare with the observed values of the seed variables. The two rows at the bottom of Table 22 comprise of the opinion of the DM and the implied $x_1$ to $x_3$ - values from the observations.

### Table 22. Experts' Opinions comprising the Decision-Maker Opinion

| | Seed 1: Average number of construction workers on a frame | | | Seed 2: Average number of construction workers on a purlin | | | Seed 3: Average number of bays' cladding stacked on a frame | | | C(e) | I(e) | W(e) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | $x_1$ | $x_2$ | $x_3$ | $x_1$ | $x_2$ | $x_3$ | $x_1$ | $x_2$ | $x_3$ | | | |
| Experts' Opinions | 2.00 | 2.86 | 5.00 | 0.99 | 1 | 1.01 | 1.1 | 2 | 2.9 | 0.48 | 2.12 | 1.02 |
| | 2.10 | 3.00 | 3.90 | 1.05 | 1.5 | 1.95 | 1.1 | 2 | 2.9 | 0.62 | 1.10 | 0.68 |
| | 2.50 | 3.33 | 4.00 | 1 | 1.5 | 2 | 1.05 | 1.5 | 1.95 | 0.70 | 1.32 | 0.93 |
| | 2.08 | 5.83 | 9.58 | 1.05 | 1.5 | 1.95 | 1.1 | 2 | 2.9 | 0.82 | 0.68 | 0.55 |
| | 2.40 | 6.00 | 9.60 | 1.05 | 1.5 | 1.95 | 1.05 | 1.5 | 1.95 | 0.87 | 0.89 | 0.77 |
| | 2.86 | 3.33 | 5.00 | 1 | 1.5 | 2 | 1 | 2 | 3 | 0.92 | 1.04 | 0.95 |
| DM | 2.35 | 3.89 | 5.90 | 1.02 | 1.40 | 1.77 | 1.06 | 1.83 | 2.59 | 0.97 | 1 | 0.97 |
| Obser-vations | 3.68 | 4.2 | 4.72 | 1.38 | 1.63 | 1.94 | 1.11 | 1.35 | 1.64 | 1 | - | - |

* Seed variable 1 is in terms of the number of workers on a 20m-spanning frame.

Note that the DM opinion compares more favourably with the observed $x_1$ to $x_3$ - values than any of the experts' opinions do. This is confirmed by the fact that the DM has the highest C(e) - value. From inspection it is concluded that the experts' opinions compare the least favourable with Seed Variable 3 and the most favourable with Seed Variable 1. The performance of the DM opinion is evaluated in the following section.

### 4.2.3 Performance of the Decision-Maker

Recall that in Section 3.4, three ways were identified in which expert opinions may be combined. To recapitulate, they are:

1. Average combination. The experts' opinions are combined by taking the average of all the opinions. Therefore all experts receive equal weights.

2. Best expert. Only the expert's opinion that received the highest calibration score (or highest rank) is used.

3. Weighted combination. The experts' opinions are combined according to the relative weights attributed to them. This combination applies to the experiment.

The performance of the DM for the weighted combination as implemented in this investigation can now be measured against that of the DM for the average combination and the best expert. This is done in terms of C(DM), I(DM) and W(DM) for the different DM's and is shown in Table 23.

**Table 23. Comparison of different combinations to find DM**

| Combination | C(DM) | I(DM) | W(DM) |
|---|---|---|---|
| 1. Average | 0.36 | 1.4 | 0.5 |
| 2. Best Expert | 0.92 | 1.04 | 0.95 |
| 3. Weighted | 0.97 | 1 | 0.97 |

W(DM) for the weighted combination is 2% higher than W(DM) for the best expert and 94% higher than W(DM) for the average combination. Most importantly though, C(DM) for the weighted combination is 5% higher than C(DM) for the best expert and 270% higher than C(DM) for the average combination. As is stressed in Section 4.1.3.2, the calibration score C(DM) is the most important measure of how well the combined experts' opinion performs.

It is also noted that only using the best expert's opinion (Combination 2) is a rather extreme type of combination. The more experts who contribute to the DM opinion, the more one can say that the DM opinion is a result of consensus building which, intuitively, is perceived to be more "reliable" and correct. Only using one expert would not contribute to such consensus building.

The fact that I(DM) is the smallest for the weighted combination means that the other DM's are more confident in their opinions. All things considered, the results from Table 23 show that the weighted combination of expert opinion, as is implemented in this experiment, outperforms that of the average and best expert combinations.

## 4.3 Sensitivity of the Experiment to Assumptions made by the Analyst

The calibration of the experiment is dependent on certain assumptions made by the analyst. This is in disagreement with *Principle 1* for using expert opinion in science (see Section 3.2.3), i.e. the requirement that the experiment be reproducible, since all analysts may not make the same assumptions. It is therefore necessary to prove that the outcome of the experiment will not be significantly effected by these assumptions, in other words the experiment should not be sensitive to these assumptions.

### 4.3.1 Experts with High Uncertainty

As stated in Section 3.6.3, certain experts could not provide 90%-confidence intervals or best estimates for some of the questions as they maintained that they did not have sufficient knowledge. For the experts to be properly calibrated it is necessary that all experts provide opinions on all the seed variables. In order to obtain an opinion from these experts they were urged to provide the range of values for which they are 100% certain that the true value of the seed variable would fall within. These two values now constitute the $x_0$ and $x_4$ - values for the experts' distributions. This is shown in Figure 20.
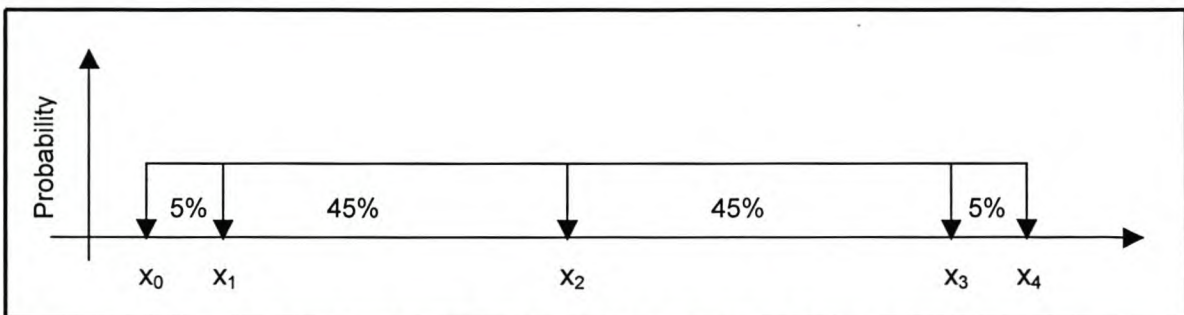


**Figure 20. Expert's Distribution for High Uncertainty**

The experts' distributions for such cases are represented by the uniform or minimal information distributions. If a particular expert's $x_0$ and $x_4$ - values correspond with the intrinsic ranges for all seed variables then I(e) for that expert would be zero and

the weight of the expert opinion would also be zero. The expert is therefore penalised in this way for his/her high uncertainty.

The $x_1$, $x_2$ and $x_3$ - values for the expert are calculated according to the proportion of the total area that falls between any of these values, as shown in Figure 20. With $x_1$, $x_2$ and $x_3$ known the expert can now be calibrated as any other expert and the experiment therefore remains consistent.

### 4.3.2 Experts with Low Uncertainty

Some of the experts did not provide 90%-confidence intervals for certain seed variables. Instead they committed themselves to one value only (the best estimate) which therefore results in no uncertainty and a deterministic opinion for that particular seed variable. For Equation (8) to be applied it is necessary that the experts opinions be modelled probabilistically. Thus, uncertainty needs to be introduced into these experts' opinions.

Having the best estimate or $x_2$ - value for the variable, the $x_1$ and $x_3$ - values can now be found by applying a percentage of over- and undershoot to the $x_2$ - value. This percentage needs to be relatively low so as to resemble the low uncertainty in the expert's opinion. The sensitivity of the experiment to the magnitude of this percentage is measured in terms of the effect it has on the weight of the DM opinion W(DM) as well as the effect on the combined $x_3$ - value for seed variable 2 of the DM. These are shown in Figures 21 & 22 for the effect on W(DM) and $x_3$ respectively.
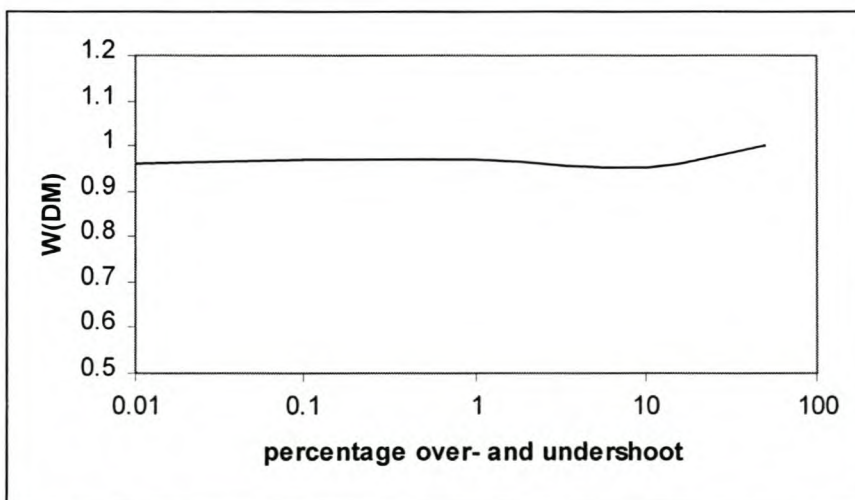


**Figure 21. Effect of percentage over- and undershoot on W(DM)**
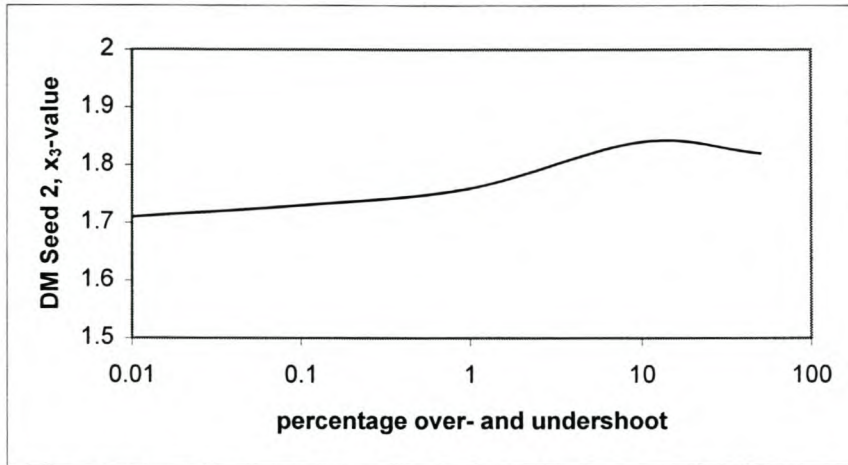
**Figure 22. Effect of percentage over- and undershoot on the $x_3$ - value of DM Seed Variable 2**

As is evident from Figures 21 & 22, the effect of the percentage over- and undershoot is less pronounced and indeed rather insignificant for the lower percentages. As the magnitude of this percentage increases, the effect on the two variables under consideration increases. This is in accordance with the fact that the experts' original opinion was deterministic, corresponding with no uncertainty. The percentage over- and undershoot to be used in the experiment is chosen as 1% although any values smaller than 1% could also have been used with no significant change in the result.

**4.3.3 The Distribution Function of the Seed Variables**

As stated previously, the lognormal-distribution function is selected to model the realisations of the seed variables. The sensitivity of the experiment to the choice of distribution function is again measured in terms of the effect on W(DM) and the $x_3$ - value for Seed Variable 2 of the DM. The alternative distribution function against which this is measured is the *normal distribution*. The results are shown in Table 24.

**Table 24. Effect of the Distribution Function of the Seed Variables**

|  | Lognormal Distribution | Normal Distribution |
|---|---|---|
| W(DM) | 0.97 | 1.00 |
| Seed 2, $x_3$ | 1.77 | 1.8 |

For both effects under consideration it can be seen that the distribution type, be it normal or lognormal, has no significant effect. It is therefore concluded that the

experiment is not sensitive to the choice between Normal and Lognormal distribution functions, which are the two main contenders for the distribution of the seed variables.

### 4.3.4 The Intrinsic Range

An assumption on the intrinsic range was required to calculate the experts' distributions for the seed variables. The number of experts whose opinions were used in finding the DM opinion (i.e. those who received non-zero weights) is six which is much less than the 31 experts who were surveyed. Therefore it is to be expected that if the intrinsic range were taken only for those experts who received non-zero weights, this would result in a smaller intrinsic range than for the total number of experts. However, according to COOKE, as long as the *same intrinsic range* is used throughout, it does not significantly affect the weights that are awarded to each expert.

A larger intrinsic range also results in experts who are less confident (or alternatively more uncertain) receiving higher calibration scores than would be the case for a smaller intrinsic range. This means that over-confidence is not rewarded and a more conservative combined expert opinion will be obtained. It is therefore decided that the intrinsic range where all the expert opinions are incorporated is valid for this experiment.

An assumption was also necessary on the percentage of over- and undershoot to be applied to the $x_3$ and $x_1$ - values to find the $x_4$ and $x_0$ - values respectively. From Equations (8a) & (8b) it is observed that the choice for this experiment was 10%. The sensitivity of the experiment to the percentage of over- and overshoot is again measured in terms of the effect on W(DM) and the $x_3$ - value for Seed Variable 2 of the DM. This is shown in Table 25.

**Table 25. Effect of the percentage over- and undershoot of the Intrinsic Range.**

| %over- and undershoot | W(DM) | Seed 2, $x_3$ |
|---|---|---|
| 1 | 0.88 | 1.76 |
| 5 | 0.92 | 1.77 |
| 10 | 0.97 | 1.77 |
| 20 | 1.08 | 1.78 |

Only for higher percentages of over- and undershoot does W(DM) become significantly influenced, while the $x_3$ - value for Seed Variable 2 remains more or less constant. It can be noted that the reason for the variation in W(DM) is due to I(DM) increasing as the intrinsic range becomes larger. Intuitively one would want to keep the percentage of under- and overshoot smaller rather than larger and it is therefore concluded that the experiment is not sensitive to this.

The sensitivity study shows that the calibration process is not sensitive to any of the assumptions made by the analyst.

## 4.4 Combining Expert Opinion for the Maximum Variables

The normalised weights calculated for the experts are now used to obtain their combined opinion on the maximum variables. This is the final step in conducting the experiment as indicated in Figure 13. The calibration of the experiment is completed and the calibrated equipment (the weighted expert opinions) is subsequently used to obtain information on the yet unknown variables (the maximum variables).

The maximum variables are (see Table 26 for the probabilistic characteristics):
- Maximum Variable 1
  The maximum number of construction workers on a frame.

- Maximum Variable 2
  The maximum number of construction workers on a purlin.

- Maximum Variable 3
  The maximum number of bays' cladding stacked on a frame.

- Maximum Variable 4
  The maximum number of maintenance workers on a frame.

- Maximum Variable 5
  The maximum number of maintenance workers on a purlin.

Although Maximum Variables 4 & 5 are not directly linked to the seed variables, as are Maximum Variables 1 to 3, they still share a common field of expertise with the seed variables and therefore the calibrated experts' opinions also apply to them.

Equations (13a-c) are applied to the experts' opinions on the maximum variables to calculate their combined opinion. The opinions of the experts who received non-zero weights as well as their combined opinion for the maximum variables are summarised in Table 26.

**Table 26. Experts' Opinions on the Maximum Variables modelling the Imposed Roof Load Mechanisms**

| $W_N(e)$: Normalised Weight of Expert e | Maximum Variable 1 Maximum number of construction workers on a frame* | | | Maximum Variable 2 Maximum number of construction workers on a purlin | | | Maximum Variable 3 Maximum number of bays' cladding stacked on a frame | | | Maximum Variable 4 Maximum number of maintenance workers on a frame* | | | Maximum Variable 5 Maximum number of maintenance workers on a purlin | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 5% | 50% | 95% | 5% | 50% | 95% | 5% | 50% | 95% | 5% | 50% | 95% | 5% | 50% | 95% |
| 0.139 | 2.10 | 3.00 | 3.90 | 1.05 | 1.50 | 1.95 | 1.10 | 2.00 | 2.90 | 2.10 | 3.00 | 3.90 | 1.05 | 1.50 | 1.95 |
| 0.207 | 4.00 | 5.00 | 8.89 | 3.96 | 4.00 | 4.04 | 1.10 | 2.00 | 2.90 | 1.15 | 2.50 | 3.85 | 1.98 | 2.00 | 2.02 |
| 0.157 | 2.40 | 6.00 | 9.60 | 1.05 | 1.50 | 1.95 | 1.05 | 1.50 | 1.95 | 1.05 | 1.50 | 1.95 | 0.99 | 1.00 | 1.01 |
| 0.190 | 4.00 | 5.00 | 5.71 | 1.00 | 2.00 | 4.00 | 1.05 | 1.50 | 1.95 | 2.00 | 2.50 | 2.86 | 1.05 | 1.50 | 1.95 |
| 0.113 | 2.08 | 5.83 | 9.58 | 1.05 | 1.50 | 1.95 | 1.10 | 2.00 | 2.90 | 1.47 | 2.67 | 3.87 | 0.99 | 1.00 | 1.01 |
| 0.194 | 5.00 | 6.67 | 10.00 | 1.00 | 2.00 | 4.00 | 2.00 | 4.00 | 7.00 | 2.00 | 2.86 | 3.33 | 1.05 | 1.50 | 1.95 |
| **Combined Opinion** | **3.46** | **5.30** | **8.00** | **1.63** | **2.21** | **3.17** | **1.26** | **2.21** | **3.37** | **1.63** | **2.50** | **3.27** | **1.23** | **1.47** | **1.71** |

*Maximum Variables 1 & 4 are in terms of the number of workers on a 20m-spanning frame
The three values per maximum variable are the 5%, 50% and 95% cumulative probability values

## CHAPTER 5: THE MODELLING OF EQUIVALENT UNIFORMLY DISTRIBUTED LOADS IN PROBABILISTIC TERMS

The purpose of this section is to establish a probabilistic model for each of the load mechanisms of the maximum variables. It is important to note here what is meant by a load mechanism. A load mechanism in this context refers to the *physical process* taking place on the roof, for instance the workers on the roof during construction, resulting in certain load effects.

Basically the modelling of the load mechanisms entails modelling of the uncertainties associated with each variable. To that end it is necessary that the values obtained from the experts be converted into equivalent uniformly distributed loads (EUDL's). Ideally one would want to have a function $f(x)$ which transforms the three quantiles of the maximum variables into three EUDL's from where the uncertainty can be determined in terms of $kN/m^2$ - values.

The first step is to find $f()$ so that:

$$Y = f(X)$$

where  X = unrefined value for Maximum Variable
Y = EUDL value for Maximum Variable

It is anticipated that Y will also be a function of the member geometry and stiffness and the positioning of the loads on the tributary area of the member. It would be advantageous if Y could only be expressed in terms of X so as to make it as generally applicable as possible. The reason for this is that ultimately the values obtained are to be compared with the prescribed loads of the SABS loading code which applies to all cases of building geometry and load positioning. To facilitate this, Y is to be proven insensitive to certain building geometries, and certain assumptions would have to be made on the positioning of the loads. With the aim on attaining the *minimum* level of reliability which is provided for by the current SABS prescribed loads and load factors, it is important that the assumptions made are conservative.

The rationale applied in relating the different load mechanisms to EUDL's is as follows: The load effects which are most critical in the design of the building are considered. A load effect can be considered critical if it is the determining factor for

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the sizes of the main members of the structure; in other words the cost of the structure is determined by the magnitude of these load effects. For these load effects, the load mechanisms are then applied to the structure in such a manner so as to produce the most adverse effects, i.e. to maximise the said load effects. An EUDL can now be calculated that produces the same load effect as for the aforementioned case.

The generic example of a typical light industrial steel building shown in Figure 2 is again used in the modelling process.

## 5.1 Maximum Variable 1: Maximum Number of Construction Workers on a Frame

This section is concerned with determining a probabilistic model for Maximum Variable 1. The first step towards this is to convert the load mechanism to an EUDL.

The two critical load effects are:
1. The maximum moment at column eaves
2. The maximum moment on the roof element

The number of workers on the frame obtained from the expert survey are so positioned on the tributary area as to maximise these load effects.

## 5.1.1 An Equivalent Uniformly Distributed Load for the Maximum Moment at Column Eaves

The maximum number of workers on the frame has been established through the expert survey. In this section the moment at column eaves $M_{eaves}$ is *maximised* for the geometric effect when the workers are congregated at the ridge of the roof, i.e. at midspan of the roof element. The EUDL that induces the same (maximised) $M_{eaves}$ as when the workers are congregated at the roof ridge is then calculated. Suppose they are positioned in a straight line on the roof so that their imposed loads may be schematically presented as shown in Figure 23.
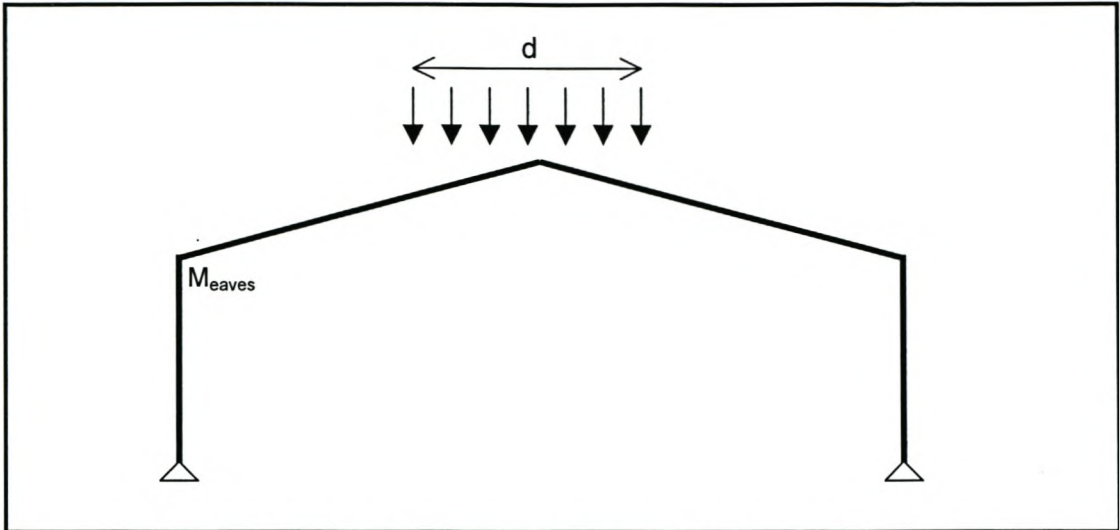
**Figure 23. Workers congregated at Roof Ridge over Distance d**

The configuration of workers as shown in Figure 23 is an unlikely occurrence, and although this may seem as an overly conservative assumption it must be stressed that ultimately *minimum* levels of reliability are to be assessed.

The moment at the column eaves $M_{eaves}$ can now be calculated using the following methodology (refer to Figure 2 for the definition of variables): For a uniformly distributed load w (kN/m) over the full length of the span L, $M_{eaves,w}$ is given by Equation (2a) in Section 2.1. Equation (2a) is repeated here as follows:

$$M_{eaves,w} = \frac{-wL^2(3+5m)}{16N} \qquad (18)$$

where  $M_{eaves,w}$  = the moment at column eaves due to a uniformly distributed load w

$w, L, h, \theta, l_2, l_1$  = as defined in Figure 2

$N$  = 2(k + 1) + m + m(1 + 2m)

$k$  = $\dfrac{2l_2 h \cos\theta}{l_1 L}$

$m$  = 1 + $\phi$

$\phi$  = $\dfrac{L \tan\theta}{2h}$

Applying this load w (kN/m) as a concentrated load P at the ridge of the roof yields the following for the moment at column eaves (formula from the STEEL DESIGNERS MANUAL):

$$M_{eaves,p} = \frac{PL(1+2m)}{4N} \qquad (19)$$

where  $M_{eaves,p}$ = the moment at column eaves due to a concentrated load P

P     = wL   (kN)

N & m = as for Equation (18)

The moment at the column eaves produced by the workers distributed as in Figure 23 $M_{eaves}$ can now be determined by linear interpolation between $M_{eaves,p}$ and $M_{eaves,w}$ as follows:

$$M_{eaves} = M_{eaves,w} + \left(1 - \frac{d}{L}\right)\left(M_{eaves,p} - M_{eaves,w}\right) \qquad (20)$$

where  $M_{eaves}$  = the moment at column eaves due to workers congregated at the roof ridge as shown in Figure 23

d     = as shown in Figure 23

From Equation (20) it is evident that $M_{eaves} = M_{eaves,w}$ when d = L, and $M_{eaves} = M_{eaves,p}$ when  d = 0.   Attention is drawn to the fact that linear interpolation is an approximation.  The accuracy of this approximation is evaluated by performing a computer analysis for specific cases (see Section 5.1.1).

Dividing Equation (20) by $M_{eaves,w}$ yields

$$\frac{M_{eaves}}{M_{eaves,w}} = \frac{d}{L} + \frac{M_{eaves,p}}{M_{eaves,w}}\left(1 - \frac{d}{L}\right) \qquad (21)$$

$\dfrac{M_{eaves,p}}{M_{eaves,w}}$ can be simplified to the following:

$$\frac{M_{eaves,p}}{M_{eaves,w}} = \frac{12 + \dfrac{8f}{h}}{8 + \dfrac{5f}{h}} \tag{22}$$

where f, h = as defined in Figure 2

It is impossible for f/h to be less than or equal to zero as this constitutes a zero or inverted roof pitch. In turn it is also highly unlikely that f/h be more than 1 as a building with such dimensions is impractical from a space utilisation perspective. Thus, f/h can be confidently bounded as follows:

$$0 \le \frac{f}{h} \le 1$$

The above bounds on f/h impose the following bounds on $M_{eaves,p}$ / $M_{eaves,w}$:

$$1.50 \le \frac{M_{eaves,p}}{M_{eaves,w}} \le 1.54$$

For all practical purposes one can now safely assume that $M_{eaves,p}$ / $M_{eaves,w}$ = 1.52. This implies that $M_{eaves,p}$ / $M_{eaves,w}$ is not dependent on the geometric properties of the building. Practically speaking, this means that workers congregating at the roof ridge of a building with, say a span L = 20m, would not result in a significantly different $M_{eaves,p}$ / $M_{eaves,w}$ - ratio than for a building with, say a span L = 10m.

Substituting $\dfrac{M_{eaves,p}}{M_{eaves,w}}$ = 1.52 into Equation (21) yields

$$\frac{M_{eaves}}{M_{eaves,w}} = 1.52 - 0.52\frac{d}{L} \tag{23}$$

Multiplying Equation (23) with $M_{eaves,w}$ and substituting Equation (18) for $M_{eaves,w}$ and $M_{eaves}$ yields

$$\frac{w_{eaves}L^2\left(3+5m\right)}{16N} = \frac{w_{workers}L^2\left(3+5m\right)}{16N}\left(1.52-0.52\frac{d}{L}\right) \qquad (24)$$

where $w_{eaves}$ = the equivalent uniform load distributed over the full span L

that results in $M_{eaves}$

$w_{workers}$ = the weight of the workers distributed over the full span L

Recalling Section 3.7, the number of workers on the frame were obtained from the experts in terms of the number of workers per spanning meter of the frame. In accordance with SABS 0160-1989 Clause 5.4.4.3 where it is assumed that the weight of a person on the roof is 90kg (0.9 kN concentrated load), $w_{workers}$ is defined as follows:

$$w_{workers} = 0.9n \ \ (kN/m) \qquad (25)$$

where n = number of workers per spanning meter obtained from

the expert survey

An assumption now has to be made on the spacing of the workers in order to express d in terms of n. In this case the onus rests on the analyst to make such an assumption. It is assumed that the workers would not be positioned less than 750mm from each other so that,

$$d = 0.75nL - 0.75 \qquad (m) \qquad (26a)$$
$$\approx 0.75nL \qquad (m) \qquad (26b)$$

The assumption d = 750mm is again conservative. Note that Equation (26a) is simplified to Equation (26b). The reason is that the portion omitted from Equation (26a) for this simplification has no significant contribution to the quantity of d.

Equations (25) & (26b) are now substituted into Equation (24) to yield $w_{eaves}$

$$w_{eaves} = 0.9n\left(1.52-0.39n\right) \qquad (kN/m) \qquad (27)$$

### 5.1.1.1 Critical Appraisal of the Conversion Methodology

The validity of Equation (27) is evaluated in terms of the following:

- Validity of the linear interpolation
- Alternative configurations of workers on the frame

The computer programme, PROKON STRUCTURAL ANALYSIS, is again used for the evaluation.

***Validity of the Linear Interpolation***

The linear interpolation applied in Equation (20) to find $M_{eaves}$ is graphically illustrated in Figure 24.
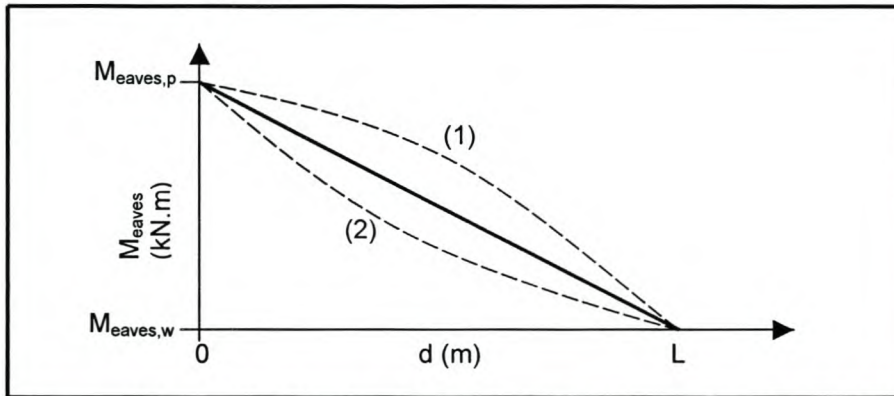


**Figure 24. Linear Interpolation of $M_{eaves}$ in terms of d**

The two dashed lines (1) & (2) in Figure 24 represent the true but unknown possible interpolation functions. If line (1) is the true interpolation function then this suggests that Equation (20) (and therefore also Equation (27)) under-estimates the true value of $M_{eaves}$ and vice versa for line (2).

The amount of over- or under-estimation is now assessed through implementation of PROKON STRUCTURAL ANALYSIS. The building properties have no bearing on this assessment as it is already proven that the interpolation function Equation (20) is insensitive to variations in building geometries and stiffnesses. Figure 25 shows the model used for the PROKON analyses with eight workers on the roof (which is equal to the $x_3$ - value of Maximum Variable 1).
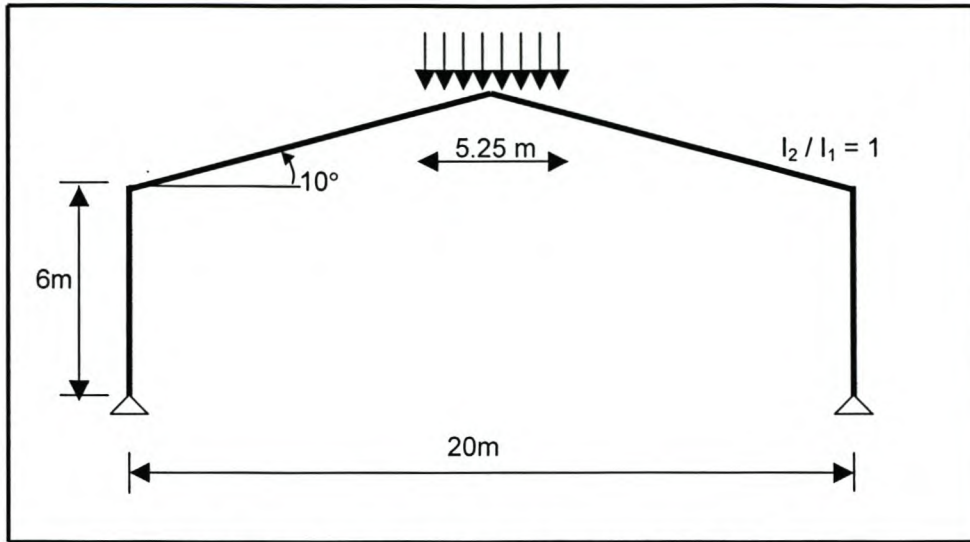
**Figure 25. Model for EUDL for Moment at Column Eaves**

The results of the PROKON analysis are now compared with those of Equations (23) & (27) (see Appendix F for the PROKON analyses output). The comparison is presented in Table 27.

**Table 27. Comparison of Computer Analyses with calculated Values for $M_{eaves}$**

| | Theoretical number of workers $n_t$* | Practical number of workers $n_p$* | Adjusted weight per worker = $0.9 \times n_t/n_p$ (kN) | $M_{eaves}$ through PROKON analyses (kN.m) | $M_{eaves}$ through Equations (27) & (23) (kN.m) | Ratio of $M_{prokon}$ to $M_{equations}$ |
|---|---|---|---|---|---|---|
| $x_1$ | 3.4 | 3 | 1.02 | 6.01 | 5.76 | 1.04 |
| $x_2$ | 5.2 | 5 | 0.94 | 9.13 | 8.62 | 1.06 |
| $x_3$ | 8 | 8 | 0.9 | 13.81 | 12.74 | 1.08 |

*The number of workers is in terms of the number of workers on a 20m spanning frame

The results show that the linear interpolation of Equation (20) under-estimates $M_{eaves}$ and therefore also $w_{eaves}$ by 4 to 8%. The amount of under-estimation decreases from $x_3$ to $x_1$ due to the load imposed by the fewer workers being closer to a concentrated load at the roof ridge and therefore the error made by the linear interpolation is not felt so severely.

*Alternative Configurations of Workers on the Frame*

The worker configuration as shown in Figure 23 may not be the most conservative in the sense of maximising $M_{eaves}$. $M_{eaves}$ needs to be evaluated for alternative

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configurations of workers. Staying with eight workers on the frame ($x_3$ - value), the following alternative has been identified in Figure 26.
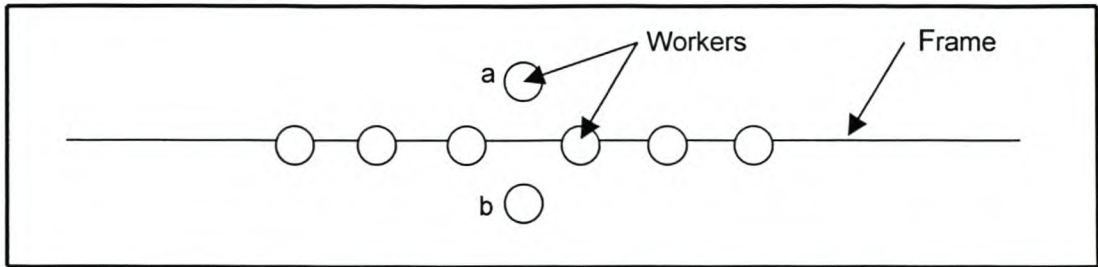


**Figure 26. Plan view of Alternative Worker Configuration**

The two workers who were positioned on the opposite ends of the line in the original configuration are now moved to positions a & b as shown in Figure 26. It is assumed that the portion of the weight of workers a & b supported by the shown frame = (weight of worker)×(1 – (0.75/spacing of frames)). This means that their effect will be felt most severely when the frames are spaced far apart. By assuming that the maximum spacing for the frames is 6m the following comparison is drawn (see Appendix F for the PROKON analyses output):

$M_{eaves}$ for original configuration      = 13.81 kN.m

$M_{eaves}$ for alternative configuration    = 13.58 kN.m

As is evident, $M_{eaves}$ for this alternative configuration is less than for the original. Obviously, taking away more workers from the ends and positioning them near the middle so that their plan projection resembles a squarer layout would further decrease the value of $M_{eaves}$. For fewer workers on the roof, i.e. the $x_2$ and $x_1$ - values, the value of $M_{eaves}$ would also further decrease (see Appendix F). It is subsequently concluded that the original configuration is the most conservative.

### 5.1.1.2 Conclusions from the Critical Appraisal of the Conversion Methodology

The following two conclusions are made:

- The linear interpolation of Equation (27) under-estimates $w_{eaves}$. This is provided for by using computer analyses to obtain $w_{eaves}$ for $x_1$ to $x_3$ (see Appendix F) and then calculating the first two moments. Equation (27) will be adjusted accordingly and then implemented as a direct means of obtaining the first two moments of Maximum Variable 1 without first converting the $x_1$ to $x_3$ - values to EUDL's. The

results from the computer analyses are then used as verification.  This is done in Section 5.1.2.

- The chosen configuration of workers is accepted as the most conservative.

The under-estimation error made due to the linear interpolation implemented in Equation (27) is provided for by increasing Equation (27) with 8.4%, which corresponds to the increase for the $x_3$ - value (see Table 27).  An assumption is also necessary on the spacing of the frames in order to do the final conversion of Equation (27) to kN/m$^2$ - values.  Applying the aforementioned to Equation (27) yields

$$w_{eaves} = 0.9n\left(1.52 - 0.39n\right) \times \frac{1.084}{s} \quad (kN/m) \tag{28}$$

where  s = spacing of the frames   (m)

Note that the smaller the spacing of the frames the larger $w_{eaves}$ is.  This is explained by recognising that the number of workers on a frame is not dependent on the spacing of the frames.  This was concluded from the preliminary consultation session with building and roofing contractors (see Section 3.6.3).  In other words, there would not be more workers on the roof of a building with larger frame spacings than for one with smaller spacings.  Therefore, in order to achieve the same kN/m imposed load on the frame for smaller spacings, a larger kN/m$^2$ - value would have to be applied on the tributary area than would be necessary for larger spacings.

Consistent with the principle of attaining minimum levels of reliability, a conservative assumption on the spacing of the frames is necessary.  It would be of value if an additional spacing of frames is considered which would resemble a common situation in practice.  The two spacings of frames to be considered are s = 4m and s = 5m, where s = 5m represents the average spacing of frames and s = 4m is the conservative value.  Note that from this point forward the exercise takes on two routes in that the probabilistic model is to be determined for both 4m and 5m spacing of frames.

### 5.1.2 Probabilistic Modelling of the EUDL for the Maximum Moment at Column Eaves

For the reasons stated in Section 3.3.1, Maximum Variable 1 is modelled as an *extreme type 1 basic random variable*. A spreadsheet programme PROBMOD has been developed to perform the modelling process. This is presented in Appendix G.

The first two probability moments for Maximum Variable 1 are determined in two ways and the results are subsequently compared:

- The first two moments through Equation (28).
- The first two moments by first converting the $x_1$ to $x_3$ - values to kN/m$^2$ - values.

#### *The First Two Probability Moments through Equation (28)*

The methodology is implemented in two steps as follows:

- *Step 1. Obtain the first two probability moments in terms of n (workers per spanning meter).* $x_1$, $x_2$ and $x_3$ are respectively the 5%, 50% and 95% cumulative probability values of the asymptotic type 1 extreme distribution and therefore the number of standard deviations between these values are known. The standard deviation $\sigma$ can now be determined from the "distance" between these values. If the combined expert opinion were to exactly "fit" the extreme type 1 distribution, $\sigma$ between $x_1$ - $x_2$, $x_1$ - $x_3$ and $x_2$ - $x_3$ must be exactly the same. However, since the combined expert opinion is not expected to exactly "fit" the extreme type 1 distribution, it can be expected that $\sigma_{x1\text{-}x2} \neq \sigma_{x1\text{-}x3} \neq \sigma_{x2\text{-}x3}$. It can be argued that for the purpose of this experiment there is more interest in the $x_2$ to $x_3$ region which constitutes the maximum range of values, and that the experts would also have a better sense of this region and would therefore provide more accurate estimates than for $x_1$ - $x_2$ or $x_1$ - $x_3$. Since $x_1$ constitutes a minimum value of a maximum variable this is not as easily assessable for the experts as, for instance, $x_3$ which constitutes a maximum value of a maximum variable. Thus, $\sigma_{x2\text{-}x3}$ is selected as the best representation of the second moment and $x_2$ as the first moment $E(X)$. $\sigma_{x2\text{-}x3}$ is to be compared with $\sigma_{x1\text{-}x3}$ in order to identify any major discrepancies that may exist.

For an extreme type 1 variable, the cumulative probability distribution is given by (formulae for the extreme type 1 distribution are obtained from VAN DEVENTER (2000)):

$$F_X(x) = \exp(-\exp(-\alpha(x - \mu))) \tag{29}$$

where $\alpha$ & $\mu$ = the two defining parameters of the extreme type 1 distribution

By substituting $x = x_2$ and $F_X(x_2) = 50\%$, and $x = x_3$ and $F_X(x_3) = 95\%$ into Equation (29), one obtains two simultaneous equations with two unknowns namely $\alpha$ and $\mu$. Solving these for $\alpha$ and $\mu$, $\sigma_{x2\text{-}x3}$ can now determined through the following relationship (which applies to the extreme type 1 distribution):

$$\sigma_{x_2 - x_3} = \sqrt{\frac{1.645}{\alpha^2}} \tag{30}$$

$\sigma_{x_1 - x_3}$ can now be obtained in a similar way and the results are as follows:

$$\sigma_{x_2 - x_3} = 0.0724 \text{ workers / spanning meter (or 1.44 workers / 20m)}$$

$$\sigma_{x_1 - x_3} = 0.0715 \text{ workers / spanning meter (or 1.43 workers / 20m)}$$

Evidently there is minimal discrepancy between $\sigma_{x_2 - x_3}$ and $\sigma_{x_1 - x_3}$. $\sigma_{x_2 - x_3}$ is selected as representative for the reasons stated earlier. The first two moments so calculated are:

$$E(n) = 0.27$$

$$\sigma_n = 0.072$$

where n = number of workers per spanning meter.

- *Step 2. Convert the first two probability moments to kN/m² - values.* This is done through implementation of Equation (28) so that,

$$E(w_{eaves}) = 0.9E(n)(1.52 - 0.39E(n)) \times \frac{1.084}{s} \qquad (kN/m^2) \tag{31}$$

$$\sigma_w = \frac{\partial}{\partial n}(\text{Equation (28)}) \times \sigma_n \times \frac{1.084}{s} \qquad (kN/m^2)$$

$$= 0.9(1.52 - 0.78n) \times \sigma_n \times \frac{1.084}{s} \qquad (kN/m^2) \tag{32}$$

Due to Equation (28) being quadratic in n, the first derivative $\partial/\partial n$(Equation (28)) is not independent of n. Therefore the most likely realisation of n namely n* is to be selected in order to obtain $\sigma_w$. Assuming that the most likely realisation n* is the expected value E(n), $\sigma_w$ can now be found. The results of Equations (31) & (32) are summarised in Table 28.

**Table 28. First Two Moments through Equation (28)**

|  | s = 5m | s = 4m |
|---|---|---|
| E(w) (kN/m²) | 0.072 | 0.089 |
| $\sigma_w$ (kN/m²) | 0.020 | 0.025 |

***The First Two Probability Moments by first converting the $x_1$ to $x_3$ - values to kN/m² - values***

The $x_1$, $x_2$ and $x_3$ - values are converted to kN/m - values using the computer analyses as explained in Section 5.1.1.1 (also refer to Appendix F). The results are:

$$x_1 = 0.24 \text{ kN/m}$$
$$x_2 = 0.36 \text{ kN/m}$$
$$x_3 = 0.53 \text{ kN/m}$$

The first two moments are now determined directly from the "distance" between these values as explained earlier, and divided by the frame spacing s to convert them to kN/m² - values. Again $\sigma_{x2\text{-}x3}$, and $\sigma_{x1\text{-}x3}$ are calculated to identify any major discrepancies:

$$\sigma_{x_2-x_3} = 0.019 \text{ kN/m}^2 \quad (5\text{m frame spacing})$$

$$\sigma_{x_1-x_3} = 0.019 \text{ kN/m}^2 \quad (5\text{m frame spacing})$$

As is evident, no discrepancy exists between $\sigma_{x2\text{-}x3}$ and $\sigma_{x1\text{-}x3}$. The calculated values for the first two moments are shown in Table 29 for s = 5m and s = 4m.

**Table 29. First Two Moments through first converting to EUDL for $M_{eaves}$**

|  | s = 5m | s = 4m |
|---|---|---|
| E(w) (kN/m²) | 0.072 | 0.089 |
| $\sigma_w$ (kN/m²) | 0.019 | 0.023 |

The conclusions from the results of the two methods implemented in the aforementioned as shown in Tables 28 & 29 are presented in Section 5.1.5.

### 5.1.3 An EUDL for the Maximum Moment in the Roof Element

As for the maximum moment at column eaves, it is assumed that the maximum moment in the roof element $M_{roof}$ occurs when the workers are all grouped together at the ridge of the roof, i.e. at midspan of the roof element.

Following the same logic as for the derivation of $M_{eaves}$, $M_{roof}$ is also determined by interpolation between $M_{roof,p}$ (moment due to concentrated load at the roof ridge) and $M_{roof,w}$ (moment due to distributed load over full span of frame) according to d. $M_{roof,w}$ and $M_{roof,p}$ are given by (formulae from the STEEL DESIGNERS MANUAL, refer to Figure 2):

$$M_{roof,w} = \frac{wL^2}{8} - \frac{wL^2(3+5m)}{16N} \tag{33}$$

$$M_{roof,p} = \frac{PL(N-m(1+2m))}{4N} \tag{34}$$

where $M_{roof,w}$ = the moment at the roof ridge due to a uniformly distributed
load w

$M_{roof,p}$ = the moment at the roof ridge due to a concentrated load P at
The roof ridge

$m$ $= 1 + \dfrac{f}{h}$

$N$ $= 2\left(\dfrac{l_2 h \sin\theta}{l_1 f} + 1\right) + m + m(1+2m)$

$P$ $= wL$

Note that Equations (33) & (34) assume that the maximum moment occurs at the roof ridge. Although this is not exactly true, it will subsequently be observed that there is no loss in thoroughness through this approximation. Now $M_{roof}$ can be calculated by linear interpolation between $M_{roof,w}$ and $M_{roof,p}$ as follows:

$$M_{roof} = M_{roof,w} + \left(1 - \frac{d}{L}\right)(M_{roof,p} - M_{roof,w}) \tag{35}$$

where $d$ = as shown in Figure 23.

Dividing Equation (35) by $M_{roof,w}$ yields

$$\frac{M_{roof}}{M_{roof,w}} = \frac{d}{L} + \frac{M_{roof,p}}{M_{roof,w}}\left(1-\frac{d}{L}\right) \tag{36}$$

Simplification of $\dfrac{M_{roof}}{M_{roof,w}}$ yields

$$\frac{M_{roof,p}}{M_{roof,w}} = \frac{4B}{2B + 5m + 9m^2} \tag{37}$$

$$\text{where} \quad B = 2\left(\frac{l_2 h \sin\theta}{l_1 f} + 1\right) + m$$

From Equation (37) it is apparent that, contrary to Equation (22), $M_{roof,p} / M_{roof,w}$ is not independent of the building geometry and relative stiffness. This implies that it would be worse for workers to congregate at the ridge of a building with, say, a longer span than it would be for a shorter span as far as the maximum moment on the roof is concerned.

$M_{roof,p} / M_{roof,w}$ is dependent on the following three building parameters:
- The height to span ratio h/L
- The roof angle $\theta$
- The stiffness ratio $l_2/l_1$

The influence of each of these on $M_{roof,p} / M_{roof,w}$ is now determined by performing a parametric study where one parameter is varied and the other two kept constant and observing the effect this has on $M_{roof,p} / M_{roof,w}$. This parametric study was carried out with the aid of the spreadsheet programme PARSTUDY and the results are shown in Appendix H. Attention is drawn to the fact that PARSTUDY calculates the *maximum moment* in the roof element which is at a slight offset from the ridge of the roof.

The influence of the three building parameters on $M_{roof,p} / M_{roof,w}$ is as follows:
- *The height to span ratio h/L.* An increase in h/L results in a decrease in $M_{roof,p} / M_{roof,w}$.
- *The roof angle $\theta$.* An increase in $\theta$ results in an increase in $M_{roof,p} / M_{roof,w}$.
- *The stiffness ratio $l_2/l_1$.* An increase in $l_2/l_1$ results in a decrease in $M_{roof,p} / M_{roof,w}$.

Bounds are now imposed on the range of possible values that each of the building parameters may assume:

- $0.15 \leq h/L \leq 0.5$. A building with $h/L = 0.15$ would only reach a viable height $h = 3m$ at a span $L = 20m$. A building with such dimensions is proportionally distorted and it is highly unlikely that a building would have a $h/L$ - ratio smaller than 0.15. A building with $h/L = 0.5$ would only reach a viable span $L = 10m$ at a height $h = 5m$. A building with such dimensions is also proportionally distorted and it is highly unlikely that a building would have a $h/L$ - ratio larger than 0.5.

- $3° \leq \theta \leq 15°$. It is highly unlikely that a roof angle would be smaller than $3°$ since this would pose a problem with waterproofing of the roof. A building with a roof angle larger than $15°$ takes on the configuration of a building with an attic and attics are certainly not common to low-rise industrial steel buildings. It is also ineffective space utilisation to have a roof angle of larger than $15°$.

- $0.5 \leq l_2/l_1 \leq 1.5$ for roof beams, and $11.5 \leq l_2/l_1 \leq 25$ for roof trusses. Refer to Sections 2.2.1 & 2.2.2 for an explanation on how the bounds for the $l_2/l_1$ - ratio are determined.

The aforementioned bounds can also be confirmed by observing that all geometries documented from the site surveys in Table 18 fall well within these bounds.

The building parameters now need to be chosen so that it maximises $M_{roof,p} / M_{roof,w}$ since this would lead to a larger EUDL and therefore a more conservative model. However, it would be overly conservative to combine the building parameters in such a way that the "worst" extremes occur in the same building, i.e. the lower bound for $h/L$, the upper bound for $\theta$ and the lower bound for $l_2/l_1$ for roof beams. It is highly unlikely for a building to have such a combination of parameters.

A method of combination that would yield conservative but not unrealistic results is that of *Turkstra's Rule*, where the extreme value of one variable is combined with the average of the other two variables. The extreme values are to be the upper or lower bounds of the variables and the average values will be those taken from the site surveys summarised in Table 18 (except for $l_2/l_1$).

Three possibilities arise:

- $h/L = 0.15$, $\theta = 10°$ and $l_2/l_1 = 18$, where $h/L = 0.15$ is the lower bound for $h/L$ and the remaining two parameters take on their average values. The average $l_2/l_1$ - ratio for roof trusses is used in this combination since $h/L = 0.15$ would pertain to buildings with spans L in excess of 20m which would predominately have roof trusses. $M_{roof,p} / M_{roof,w} = 2.3$ (see Appendix H) for this combination of parameters.

- $\theta = 15°$, $h/L = 0.275$ and $l_2/l_1 = 1$, where $\theta = 15°$ is the upper bound for $\theta$ and the remaining two parameters take on their average values. In this case the average $l_2/l_1$ for roof beams is used since this would be the more conservative alternative. $M_{roof,p} / M_{roof,w} = 3.2$ (see Appendix H) for this combination.

- $l_2/l_1 = 0.5$, $h/L = 0.275$ and $\theta = 10°$, where $l_2/l_1 = 0.5$ is the lower bound for $l_2/l_1$ for roof beams and the remaining two parameters take on their average values. $M_{roof,p} / M_{roof,w} = 3.2$ (see Appendix H) for this combination.

Thus, $M_{roof,p} / M_{roof,w} = 3.2$ is selected as the maximum value for $M_{roof,p} / M_{roof,w}$ and is subsequently used.

Substituting $M_{roof,p} / M_{roof,w} = 3.2$ into Equation (36) yields

$$\frac{M_{roof}}{M_{roof,w}} = 3.2 - 2.2\frac{d}{L} \tag{38}$$

We now proceed in the same way as in Section 5.1.1 from Equation (23) to (27) to find

$$w_{roof} = 0.9n(3.2 - 1.65n) \ (kN/m) \tag{39}$$

### 5.1.3.1 Critical Appraisal of the Conversion Methodology

Equation (39) is evaluated in terms of the following:
- Validity of the linear interpolation
- Alternative configurations of workers on the frame

The computer programme, PROKON STRUCTURAL ANALYSES, is again used for the evaluation. The dimensions and relative stiffnesses of the building used in the

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computer analyses are so chosen that they comply with values of the building parameters that resulted in $M_{roof,p} / M_{roof,w} = 3.2$, namely $\theta = 15°$, $h/L = 0.275$ and $l_2/l_1 = 1$. The model used for the PROKON analysis with the chosen properties is shown in Figure 27.
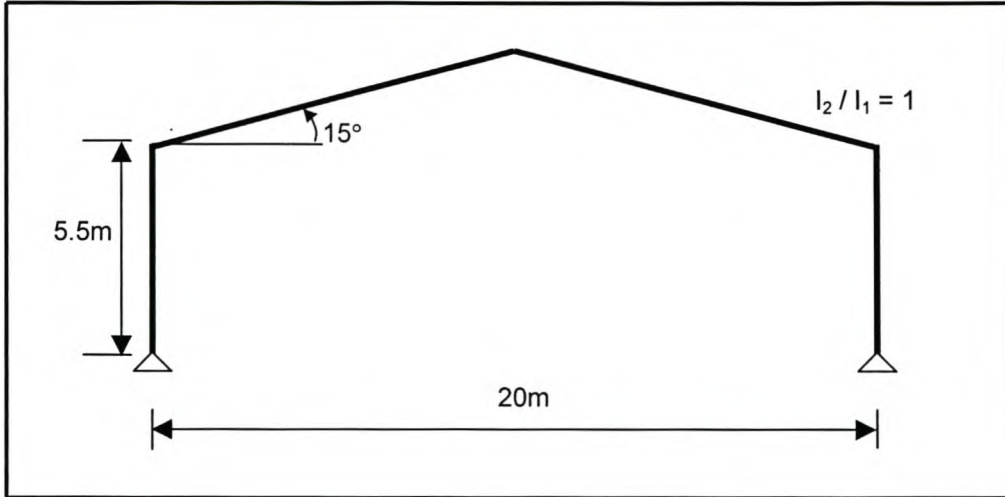


**Figure 27. Model for EUDL for Maximum Moment in the Roof Element**

For the sake of brevity, this section is not put forward in as much detail as Section 5.1.1, and the reader should be aware that the same principles apply to this section in deriving the formulae.

### Validity of the Linear Interpolation

The results of the PROKON analysis are now compared with those of Equations (38) & (39) (see Appendix F for the PROKON analyses output). The comparison is presented in Table 30.

**Table 30. Comparison of Computer Analyses with calculated Values for $M_{roof}$**

| | Theoretical number of workers $n_t$* | Practical number of workers $n_p$* | Adjusted weight per worker = $0.9 \times n_t/n_p$ (kN) | $M_{roof}$ through PROKON analyses (kN.m) | $M_{roof}$ through Equations (38) & (39) (kN.m) | Ratio of $M_{prokon}$ to $M_{equations}$ |
|---|---|---|---|---|---|---|
| $X_1$ | 3.4 | 3 | 1.02 | 6.76 | 7.09 | 1.05 |
| $X_2$ | 5.2 | 5 | 0.94 | 9.54 | 10.29 | 1.08 |
| $X_3$ | 8 | 8 | 0.9 | 12.96 | 14.28 | 1.10 |

*The number of workers is in terms of the number of workers on a 20m-spanning frame

It is found that for the maximum moment on the roof Equation (38) over-estimates $M_{roof}$, and therefore also $w_{roof}$, by 5 to 10%. This is in contrast to Equation (27) where the moment at column eaves is under-estimated. The reason for the one being over-estimated and the other under-estimated is that $M_{eaves}$ and $M_{roof}$ are opposite in sign, meaning the one is a negative moment and the other positive. The reason for the error in estimation being larger in this instance is that the interpolation is done over a wider range of values namely from 3.2 to 1, whereas in Equation (27) it is done merely from 1.52 to 1.

### *Alternative Configurations of Workers on the Frame*

Again, the effect of the worker configuration as shown in Figure 26 is measured against that of the original configuration. For the maximum moment on the roof and eight workers on the roof (see Appendix F for PROKON analyses output):

$M_{roof}$ for original configuration      = 14.3 kN.m

$M_{roof}$ for alternative configuration      = 14.4 kN.m

$M_{roof}$ for the alternative configuration is the same as $M_{roof}$ for the original (for all practical purposes). In the comparison shown above, $M_{roof}$ for the original configuration is calculated through Equations (38) & (39). Since $M_{roof}$ is already over-estimated by Equations (38) & (39) due to the linear interpolation, this means that the alternative configuration is actually more conservative. However, the over-estimation error due to the linear interpolation of Equations (38) & (39) provides for this alternative configuration. Therefore Equation (39) remains as is. The above also applies to the $x_1$ and $x_2$ - values (see Appendix F).

### 5.1.3.2 Conclusions from the Critical Appraisal of the Conversion Methodology

Although the linear interpolation overestimates $M_{roof}$ by 9%, $M_{roof}$ as calculated through Equations (38) & (39) is equal to $M_{roof}$ as calculated for the alternative positioning of workers on the frame. Therefore $w_{roof}$, as calculated through Equation (39), remains as is.

The spacing of the frames is now incorporated into Equation (39) to find

$$W_{roof} = \frac{0.9n(3.2 - 1.65n)}{s} \quad (kN/m^2) \qquad (40)$$

where s = spacing of frames (m)

As explained earlier, frame spacings of 4 and 5m are considered.

## 5.1.4 Probabilistic Modelling of the EUDL for the Maximum Moment in the Roof Element

The probabilistic model of the EUDL for the maximum moment on the roof is developed in exactly the same way as the EUDL for the moment at the column eaves. Therefore the methodology applied and the assumptions made in Section 5.1.2 are not repeated here, and only the results are shown at important stages of the development.

The first two probability moments is again determined in two ways and the results compared for verification:

- The first two moments through Equation (40).
- The first two moments by first converting the $x_1$ to $x_3$ - values to $kN/m^2$ - values.

### The First Two Probability Moments through Equation (40)

The results of the derivation are shown in Table 31. It is noted that no major discrepancies were identified between $\sigma_{x2-x3}$ and $\sigma_{x1-x3}$ (see Appendix G).

### Table 31. First Two Moments through Equation (40)

|  | s = 5m | s = 4m |
|---|---|---|
| E(w) (kN/m²) | 0.132 | 0.165 |
| $\sigma_w$ (kN/m²) | 0.030 | 0.038 |

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***The First Two Probability Moments by first converting the $x_1$ to $x_3$ - values to kN/m² - values***

The $x_1$, $x_2$ and $x_3$ - values are converted to kN/m² - values using the computer analyses as explained in Section 5.1.3.1 (also refer to Appendix G). The results are:

$$x_1 = 0.45 \text{ kN/m}$$
$$x_2 = 0.66 \text{ kN/m}$$
$$x_3 = 0.91 \text{ kN/m}$$

The second moment is now determined directly from the "distance" between these values as explained in Section 5.1.2, and divided by the frame spacing s to obtain these values in terms of kN/m². The results are shown in Table 32 for s = 5m and s = 4m.

**Table 32. First Two Moments through first converting to EUDL for $M_{roof}$**

|  | s = 5m | s = 4m |
|---|---|---|
| E(w) (kN/m²) | 0.132 | 0.165 |
| $\sigma_w$ (kN/m²) | 0.027 | 0.034 |

### 5.1.5 Interpretation of Results, and Conclusion

The probabilistic models for Maximum Variable 1 as obtained through the two methods for the moment at column eaves and the moment on the roof are compared in Table 33.

**Table 33. Comparison of Probabilistic Models for the EUDL's for $M_{eaves}$ and $M_{roof}$**

| Frame Spacing | Probability Parameter | Moment at Column Eaves | | Maximum Moment on the Roof | |
|---|---|---|---|---|---|
| | | Through Equation (28) | Through first converting to kN/m² - values | Through Equation (40) | Through first converting to kN/m² – values |
| 5m | $1^{st}$ moment (kN/m²) | 0.072 | 0.072 | 0.132 | 0.132 |
| | $2^{nd}$ moment (kN/m²) | 0.02 | 0.019 | 0.030 | 0.027 |
| 4m | $1^{st}$ moment (kN/m²) | 0.089 | 0.089 | 0.165 | 0.165 |
| | $2^{nd}$ moment (kN/m²) | 0.025 | 0.023 | 0.038 | 0.034 |

As is evident from Table 33, for both the moment at column eaves and the maximum moment on the roof there exists a discrepancy between the $2^{nd}$ moment obtained

through Equations (28) & (40) and the $2^{nd}$ moment obtained through first converting to $kN/m^2$ - values in that the latter is smaller than the former. The reason for this is as follows:

If one considers Equation (28) (and Equation (40)) it is obvious that the uncertainty in w ($kN/m^2$) $\sigma_w$ is not constant over the range of possible realisations of $n = n^*$, due to $w = f(n)$ being non-linear in n. On closer examination of Equation (28) one also observes that $\sigma_w$ decreases as n increases. A physical interpretation of this phenomenon is that if there are *few* workers on the roof positioned over the ridge and one worker were to be added this would increase the moment at column eaves or on the roof by more than would be the case if there were *many* workers on the roof and one worker were to be added. This damping effect is a direct result of the non-linearity of $w = f(n)$. So, for $n = n^*$ where $n^*$ is *large* the uncertainty in w would be *small*. To find $\sigma_w$ through Equation (28) an assumption therefore had to be made on what $n^*$ would be, and it was decided to use $n^* = E(n)$, i.e. the expected value of n. Where the uncertainty is determined through first converting to $kN/m^2$ - values, $\sigma_w$ is obtained by knowing the number of standard deviations $x_3$ is away from $x_2$. Since $x_3$ and $x_2$ are in terms of w ($kN/m^2$), the damping effect is already incorporated in that the "distance" between $x_3$ and $x_2$ is less than the "distance" between $n_3$ and $n_2$ (n = number of workers). Thus, determining the uncertainty from $x_2$ to $x_3$ implies that $n^* > E(n)$. As it is this upper region that is of interest, particularly when assessing failure probabilities, the uncertainty obtained through first converting to $kN/m^2$ - values prevails as representative of the second moment of Maximum Variable 1. Implicitly this means that one would expect $n^*$ to be greater than $E(n)$ when the structure fails.

A second observation from Table 33 is that the first two probability moments obtained for the maximum moment on the roof are $\pm$ 100% greater than for the moment at column eaves. The reason for this being that $M_{roof,p}$ is $3.2 \times M_{roof,w}$ whilst $M_{eaves,p} = 1.52 \times M_{eaves,w}$ where $M_w$ is the moment caused by w ($kN/m$) distributed over the span of the frame L, and $M_p$ is the moment caused by wL (kN) concentrated at the ridge of the roof. This shows that the effect of workmen congregating at the ridge of the roof has a much more accentuated effect on the maximum moment on the roof than it does on the moment at column eaves. The first two moments obtained for the maximum moment on the roof is therefore accepted as the more conservative representation of Maximum Variable 1 and will be used subsequently.

The methodology implemented in establishing the probabilistic model does not take into account the effect of axial forces developed in the columns. Since the columns are to be designed for combined axial compression and bending, this is an effect that should be considered. The worst case for axial compression in a column would be when the workers are all congregated directly above the column. This situation would result in an axial force F of twice the magnitude as would be when the total weight of the workers is distributed over the full span L. However, recall that the situation where workers are congregated at the roof ridge results in a maximum moment in the roof element $M_{roof}$ of factor 3.2 larger than would be when the total weight of the workers is distributed over the full span L. Therefore, Equation (40) results in an axial compressive force in the columns of 3.2/2 = 1.6 times larger than would be for the worst scenario when the workers congregate directly above the column. Conservative allowance is therefore made for axial forces in the columns through the methodology implemented in Section 5.1.3.

The probabilistic model for Maximum Variable 1 is now presented in Figures 28 & 29 for 5m and 4m spacing of frames respectively.
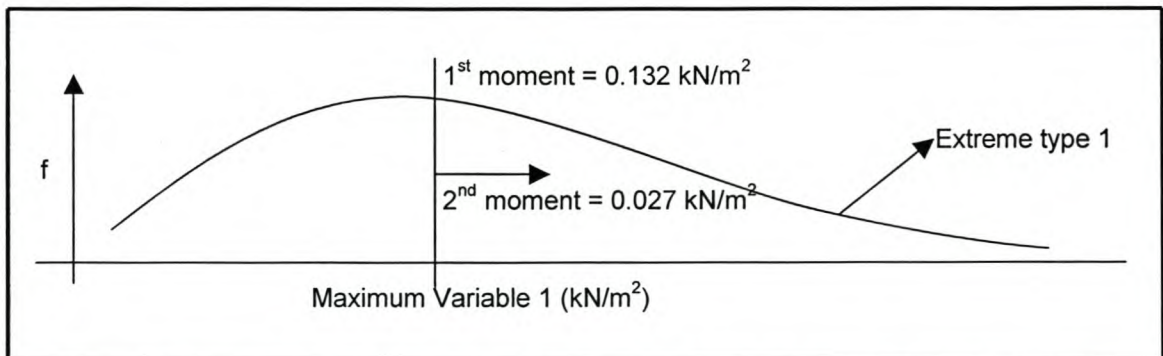


**Figure 28. Probabilistic Model for Maximum Variable 1 for 5m Frame Spacing**
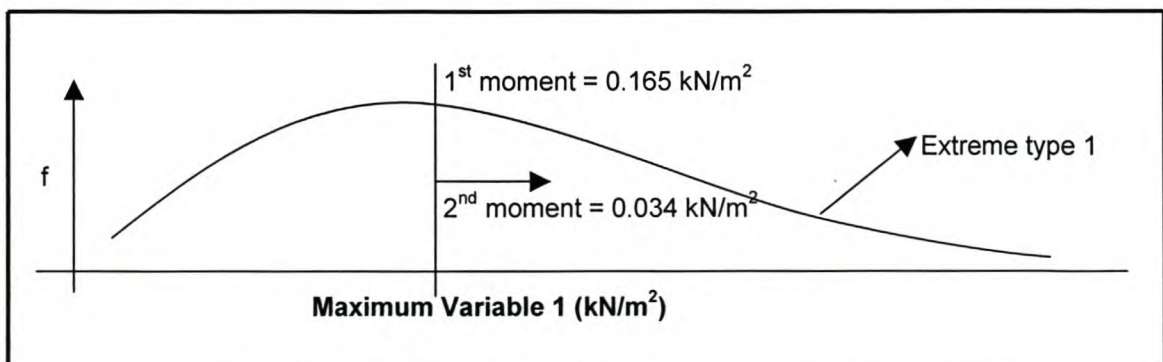


**Figure 29. Probabilistic Model for Maximum Variable 1 for 4m Frame Spacing**

## 5.2 Maximum Variable 2: Maximum Number of Construction Workers on a Purlin

This section is concerned with determining a probabilistic model for Maximum Variable 2. The first step towards this, is to convert the load mechanism to an EUDL as was done for Maximum Variable 1.

The two critical load effects are:
- The maximum positive moment at midspan.
- The maximum negative moment at the supports.

For both these load effects the number of workers on the purlin obtained from the expert survey are so positioned on the tributary area as to maximise these load effects.

From Table 26, the $x_1$, $x_2$ and $x_3$ - values for the number of workers on the tributary area of a purlin are:

$$x_1 = 1.6 \text{ workers}$$
$$x_2 = 2.2 \text{ workers}$$
$$x_3 = 3.2 \text{ workers}$$

Evidently, the number of workers on a purlin is not dependent on any geometric property of the purlin, as was the case for the workers on a frame. It may seem odd that $x_1$ to $x_3$ are not whole numbers, but it must be remembered that these are statistics obtained from the combined expert opinion. Figure 30 illustrates what is meant by the tributary area of a purlin (this is also what was understood by the experts as the tributary area of a purlin during the expert survey).

The number of workers on a purlin obtained from the experts are so positioned on this tributary area to maximise the load effects under consideration.
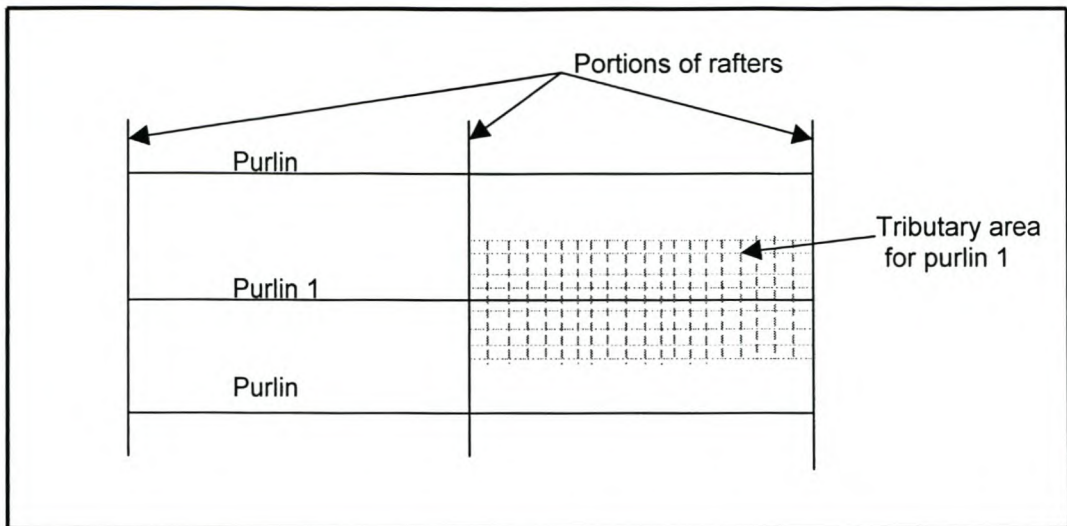
**Figure 30. Tributary Area for a Purlin**

## 5.2.1 An EUDL for the Maximum Positive Moment in the Purlin

In order to determine the maximum positive moment at midspan, a single span purlin is considered with the workers positioned near midspan. The methodology used in converting the load mechanism to an EUDL is as follows:

The $x_1$ to $x_3$ - values are rounded up or down in order to obtain an integer amount of workers. The weight of each of the integer number of workers is determined by increasing or decreasing the default 0.9 kN per worker pro-rata to the amount of rounding which has been done (see also Tables 27 & 30). These workers are now positioned on the purlin so as to maximise the positive moment at midspan and the EUDL which produces the same midspan moment is determined subsequently.

So for $x_1$ = 1.6 workers, this equates to two workers weighing 1.6(0.9)/2 = 0.72 kN. These two workers are now positioned at midspan of the purlin to maximise the midspan moment $M_{mid,p}$ as shown in Figure 31.
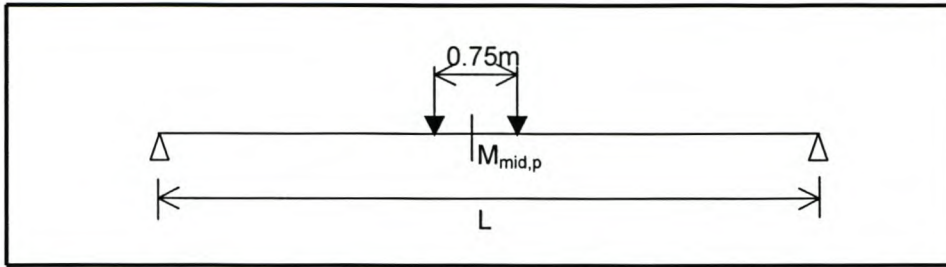
**Figure 31.   Positioning of Two Workers on Purlin to maximise the Midspan Moment**

The midspan moment induced by these workers on a single span purlin is given by:

$$M_{mid,p} = \frac{P(L - 0.75)}{2} \qquad \text{(kN.m)} \qquad (41)$$

where  P = 0.72 kN

L = purlin span = spacing of frames  (m)

The midspan moment induced by a uniformly distributed load $w_{x1}$ (kN/m$^2$) is:

$$M_{mid,w} = \frac{w_{x_1} s L^2}{8} \qquad \text{(kN.m)} \qquad (42)$$

with    s = spacing of the purlins  (m)

$w_{x1}$ can now be determined by stating $M_{mid,p} = M_{mid,w}$.  As for Maximum Variable 1, $w_{x1}$ is determined for two cases, one of which reflects a conservative situation, and the other a common situation.  The average situation transpires when L = 5m and s = 1.7m, and the conservative situation transpires when L = 4m and s = 1.4m.  The average value for the spacing of the purlins s = 1.7m is obtained from the construction site survey (see Table 18) and the extreme value s = 1.4m is obtained from consultation with experienced engineers, namely F Heyman, PJ de Villiers and IP de Villiers, all of whom have in excess of 30 years experience.  Note that the larger $w_{x1}$ - value again occurs when the tributary area is at it's minimum, i.e. when L and s are at their minimum values.  The reason for this is as stated in Section 5.1.1.2.

Positioning the workers transversely over the centre of the purlin would only produce a larger $M_{mid,p}$ when L is smaller than 4m and s is larger than 2m. As it is assumed that the frames would not be spaced closer than 4m apart and that the purlins would very rarely be spaced more than 2m apart, $M_{mid,p}$ for the configuration in Figure 31 therefore prevails as the most conservative.

For $x_2$ = 2.2 workers, this equates to two workers weighing 2.2(0.9)/2 = 0.99kN each. $w_{x2}$ can now be determined in exactly the same way as explained earlier for $w_{x1}$.

For $x_3$ = 3.2 workers, this equates to three workers weighing 3.2(0.9)/3 = 0.96 kN each. One can now proceed in the same way as for $w_{x1}$ and $w_{x2}$ by positioning the workers at midspan to find the maximum midspan moment and subsequent $w_{x3}$.

The results of the conversion of the $x_1$ to $x_3$ - values to EUDL's are shown in Table 34. Refer to PROBMOD in Appendix G for the calculations used in determining these values.

**Table 34. EUDL's for the Maximum Positive Moment in the Purlin**

|  | Span = 5m Spacing = 1.7m | Span = 4m Spacing = 1.4m |
|---|---|---|
| $w_{x1}$ (kN/m$^2$) | 0.29 | 0.53 |
| $w_{x2}$ (kN/m$^2$) | 0.40 | 0.58 |
| $w_{x3}$ (kN/m$^2$) | 0.54 | 0.76 |

**5.2.2 Probabilistic Modelling of the EUDL for the Maximum Positive Moment in the Purlin**

The probability first moment of the probabilistic model is taken as the $x_2$ - value, and the second probability moment can be calculated from knowing the number of standard deviations the quantiles are apart. Maximum Variable 2 is also assumed to be extreme type 1 distributed, as is Maximum Variable 1. By application of Equations (29) & (30), $\sigma_{x2-x3}$ and $\sigma_{x1-x3}$ are calculated (see Appendix G) and the results shown in Table 35.

**Table 35. Standard Deviation for the EUDL for the Maximum Positive Moment**

| | Span = 5m Spacing = 1.7m | Span = 4m Spacing = 1.4m |
|---|---|---|
| $\sigma_{x2-x3}$ (kN/m²) | 0.075 | 0.100 |
| $\sigma_{x1-x3}$ (kN/m²) | 0.077 | 0.106 |

The discrepancy between $\sigma_{x2-x3}$ and $\sigma_{x1-x3}$ is marginal and $\sigma_{x2-x3}$ is selected as representative for reasons stated previously. The first two probability moments of the EUDL for the maximum positive moment in the purlin are presented in Table 36.

**Table 36. First Two Moments for the EUDL for the Maximum Positive Moment**

| | Span = 5m Spacing = 1.7m | Span = 4m Spacing = 1.4m |
|---|---|---|
| E(w) (kN/m²) | 0.40 | 0.58 |
| $\sigma$(w) (kN/m²) | 0.075 | 0.100 |

### 5.2.3 An EUDL for the Maximum Negative Moment in the Purlin

For this load effect a double span purlin as shown in Figure 31 is considered. The number of workers conveyed by the experts apply to the tributary area of one span of the purlin only, so that it may be assumed that for a double span purlin the same number of workers are on the adjacent span. Note that this is a very conservative assumption. Such a configuration would maximise the negative moment at the internal support.

So, for $x_1$ = 2 workers weighing 0.72 kN each, the load is applied in the manner presented in Figure 32.
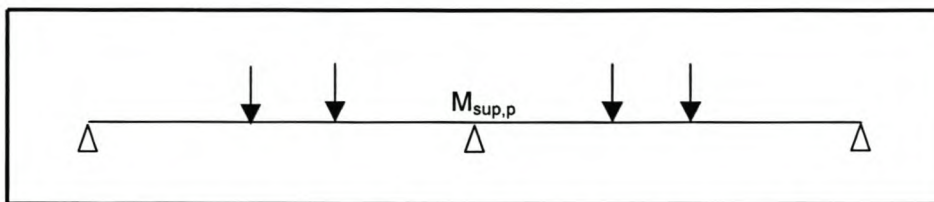


**Figure 32. Positioning of Workers on Purlin to Maximise the Support Moment**

The maximum support moment induced by the imposed loading in Figure 32 is given by (formula from the STEEL DESIGNERS MANUAL):

$$M_{sup,p} = \frac{P}{8L^2}\left(\begin{array}{l}(0.423L - 0.375)^2 (2.577L + 0.375) \\ + (0.423L + 0.375)^2 (2.577L - 0.375)\end{array}\right) - 0.846PL \qquad (43)$$

where  P = 0.72 kN

L = purlin span = spacing of frames.

The support moment induced by a uniformly distributed load $w_{x1}$ (kN/m$^2$) is:

$$M_{sup,w} = 0.125 \times w_{x1} \times s \times L^2 \quad (kN.m) \qquad (44)$$

where  s = spacing of the purlins

$w_{x1}$ can now be calculated by stating $M_{sup,p} = M_{sup,w}$.

Proceeding in the same way one can now calculate $w_{x2}$ and $w_{x3}$. The results of the conversion of the $x_1$ to $x_3$ - values to EUDL's are shown in Table 37 (see also Appendix G).

**Table 37. EUDL's for the Maximum Negative Moment in the Purlin**

|  | Span = 5m Spacing = 1.7m | Span = 4m Spacing = 1.4m |
|---|---|---|
| $w_1$ (kN/m$^2$) | 0.26 | 0.39 |
| $w_2$ (kN/m$^2$) | 0.35 | 0.53 |
| $w_3$ (kN/m$^2$) | 0.48 | 0.70 |

**5.2.4 Probabilistic Modelling of the EUDL for the Maximum Negative Moment in the Purlin**

By application of Equations (29) & (30), $\sigma_{x2\text{-}x3}$ and $\sigma_{x1\text{-}x3}$ are calculated and the results shown in Table 38 (see Appendix G).

**Table 38. Standard Deviation for the EUDL for the Maximum Negative Moment**

|  | Span = 5m Spacing = 1.7m | Span = 4m Spacing = 1.4m |
|---|---|---|
| $\sigma_{x2\text{-}x3}$ (kN/m$^2$) | 0.070 | 0.094 |
| $\sigma_{x1\text{-}x3}$ (kN/m$^2$) | 0.070 | 0.099 |

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The discrepancy between $\sigma_{x2\text{-}x3}$ and $\sigma_{x1\text{-}x3}$, is again minimal and $\sigma_{x2\text{-}x3}$ is selected as representative of the second moment for the reasons stated previously. Therefore the first two probability moments of the EUDL for the negative moment are given in Table 39.

**Table 39. First Two Moments for the EUDL for the Maximum Positive Moment**

|  | Span = 5m<br>Spacing = 1.7m | Span = 4m<br>Spacing = 1.4m |
|---|---|---|
| E(w) (kN/m²) | 0.35 | 0.53 |
| σ(w) (kN/m²) | 0.07 | 0.094 |

## 5.2.5 Interpretation of Results, and Conclusion

Comparison of Tables 36 & 39 shows that the values obtained for the maximum positive moment at midspan are higher than those for the maximum negative moment at the supports. In comparing these two load effects one has to take into consideration whether or not lateral support is provided for the compression flange in both instances. Unless the resistance conditions for both these load effects are alike no comparison can be made.

For the positive moment at midspan lateral support is provided by the roof cladding for the full portion of the purlin in positive curvature. The negative moment at the supports is laterally supported by the roof element. Although lateral support is not given for the full portion of the purlin in negative curvature, this portion is very small since the negative moment reduces to zero over a very short length of the span. This is shown in Figure 33.
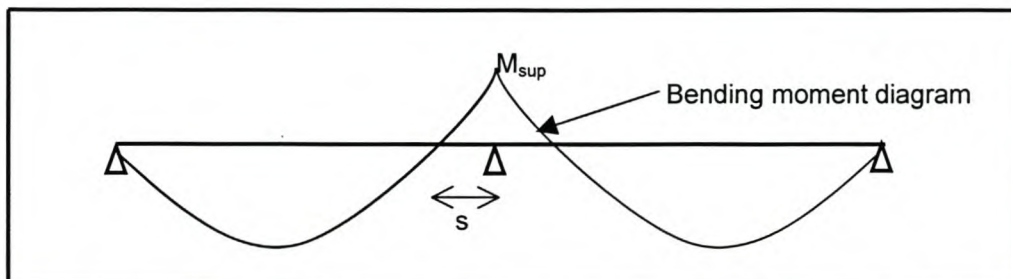


**Figure 33. Bending Moment Diagram for a double span Purlin**

Due to the distance s being relatively small the purlin will yield as a result of the design moment being exceeded rather than lateral torsional buckling of the bottom

flange. This is the same yielding mechanism as for the midspan moment and therefore the comparison is warranted.

The EUDL resulting from the maximum positive moment at midspan is subsequently selected as the more conservative and the $x_1$ to $x_3$ - values as given in Table 36 subsequently prevails.

The probabilistic model for Maximum Variable 2 is now presented in Figures 34 & 35 for 5m and 4m spacing of frames respectively.
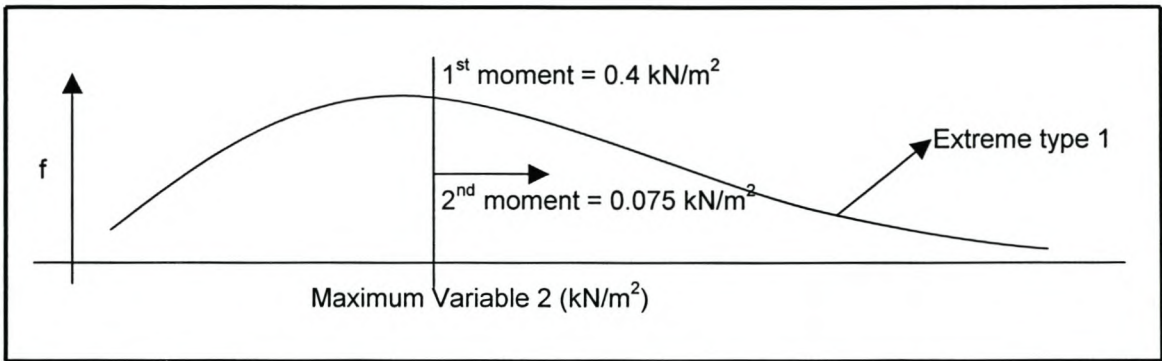


**Figure 34. Probabilistic Model for Maximum Variable 2 for 5m Frame Spacing, 1.7m Purlin Spacing**
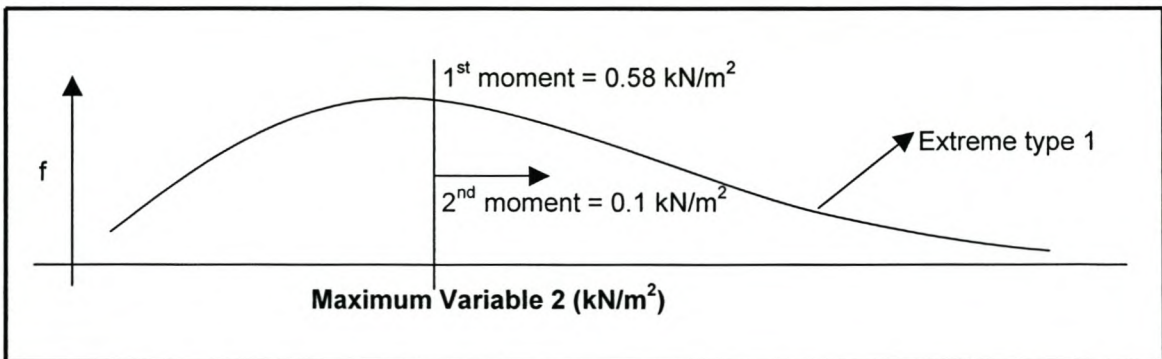


**Figure 35. Probabilistic Model for Maximum Variable 2 for 4m Frame Spacing, 1.4m Purlin Spacing**

## 5.3  Maximum Variable 3: Maximum Number of Bays' Cladding Stacked on a Frame

This section is concerned with determining a probabilistic model for Maximum Variable 3.  From the observations made during the site survey the manner in which the cladding is stacked on the frames is as follows:

Each sheet of cladding is the length of half a roof pitch, or in other words half the span length of the frames.  The sheets are stacked parallel to the span of the roof element on both pitches of the roof, from where it is spread out over the roof in a direction perpendicular to the span.  This is illustrated in Figure 36.
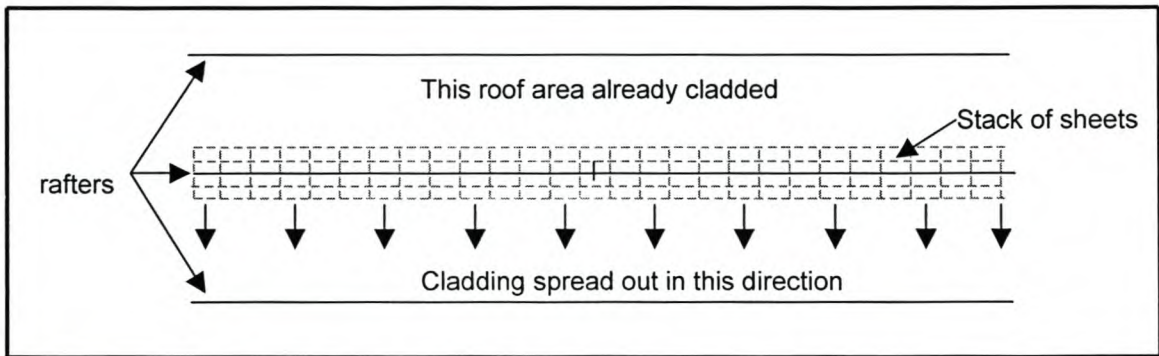


**Figure 36. Plan View of the Instalment of the Roof Cladding**

The cladding is stacked in this way without exception and therefore there is no uncertainty associated with the positioning of the stack of cladding on the frame.  The method applied previously where EUDL's are obtained for different load effects will therefore not be carried out here.  Recall that this variable is in terms of the number of bays' cladding stacked on a rafter.  Once the cladding is in place, the weight supported by a frame is that of one bay and is treated as dead load since there is low uncertainty in this.  The imposed load carried by the frame is the number of bays' cladding in excess of one bay stacked on the frame, i.e.

$$w = (n - 1) \times w_c \qquad (kN/m^2) \qquad (45)$$

where  n = number of bays cladding stacked on a frame

$w_c$ = the weight of cladding per square meter.

The weight of cladding $w_c$ ranges from 7 to 10 kg/m$^2$ for steel sheeting, and 15 kg/m$^2$ for fibre-cement sheeting. It would be overly conservative to assume that the maximum amount of over-stacking will occur with fibre-cement sheeting, which is the heavier of the two types of sheeting and not used as commonly. The situation is again favourable for the application of *Turkstra's Rule.*

The two possible combinations are:

- The maximum over-stacking combined with the weight of steel sheeting.
- The average over-stacking combined with the weight of fibre-cement sheeting.

The above combinations will yield conservative but not unrealistic results.

### 5.3.1 Maximum Over-Stacking of Steel Sheets

The $x_1$ to $x_3$ - values for Maximum Variable 3 are as follows:

$$x_1 = 1.3 \text{ bays' cladding}$$
$$x_2 = 2.2 \text{ bays' cladding}$$
$$x_3 = 3.4 \text{ bays' cladding}$$

These can now be converted to kN/m$^2$ - values by application of Equation (45) (see Appendix G):

$$w_1 = 0.03 \text{ kN/m}^2$$
$$w_2 = 0.12 \text{ kN/m}^2$$
$$w_3 = 0.24 \text{ kN/m}^2$$

For the above conversion the weight of steel cladding was taken as 10 kg/m$^2$, which is the upper limit of the range of weights of steel sheets. This conservatism is acceptable due the fact that the maximum over-stacking of 10 kg/m$^2$ steel sheets is not an unrealistically unlikely occurrence.

The first derivative of Equation (45) with respect to n is independent of n as a result of Equation (45) being linear in n. This means that the standard deviation of w namely $\sigma_w$ is constant over the range of possible values of n. Therefore there would be no difference in the value of the second probability moment if it were to be

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obtained from the first derivative of Equation (45) or from the "distances" between the quantiles $w_1$ to $w_3$.

Assuming that Maximum Variable 3 is extreme type 1 distributed and applying the same procedure as in Section 5.1.2 the first two probability moments are calculated. The second moment obtained reflects only the uncertainty in the *number* of bays' cladding and not the uncertainty in the weight of the cladding. The coefficient of variation associated with the dead weight of the sheets is taken as 10%. This uncertainty is combined with the aforementioned uncertainty through the sum of squares to find the total uncertainty. The first two probability moments are:

$$E(w) = 0.12 \quad kN/m^2$$
$$\sigma_w = 0.063 \quad kN/m^2$$

Although not shown here, it was again found that the discrepancy between $\sigma_{w2-w3}$ and $\sigma_{w1-w3}$ is insignificant (see Appendix G).

### 5.3.2 Average Over-Stacking of Fibre-Cement Sheets

The average over-stacking is combined with the weight of fibre-cement sheeting. The probabilistic model for the average number of bays' cladding stacked on a frame is obtained directly from the observations made during the site survey. It is important to recognise that this model is not the same as for Seed Variable 3. For Seed Variable 3 only the sample uncertainty $\sigma_{sample}$ is used, which represents the uncertainty in the *average value* of the variable (see Section 4.2.1). However, since specific realisations of the amount of over-stacking are relevant here, the total uncertainty $\sigma$ is to be obtained. This is done as follows:

$$\sigma = \sqrt{\sigma^2_{sample} + \sigma^2_{inherent}} \tag{46}$$

where $\sigma_{inherent}$ = the inherent variability and is given by Equation (17b)

$\sigma_{sample}$ = the sample uncertainty and is given by Equation (17c)

Subsequently the probability parameters for the number of bays' cladding stacked on the frame are as follows (see Appendix G):

<div style="margin-left: 4em;">

Distribution Type:    Lognormal

First Moment:    1.36   bays' cladding

Second Moment:    0.62   bays' cladding

</div>

The first two probability moments are now converted to $kN/m^2$ - values through Equation (45), with $w_c$ = 15 $kg/m^2$, and the uncertainty increased to allow for the variability of the dead weight of the sheets. The results are:

$$E(w) = 0.05\ kN/m^2$$

$$\sigma_w = 0.095\ kN/m^2$$

### 5.3.3 Interpretation of Results, and Conclusion

The probabilistic models obtained in Sections 5.3.1 & 5.3.2 are represented in Figures 37 & 38.
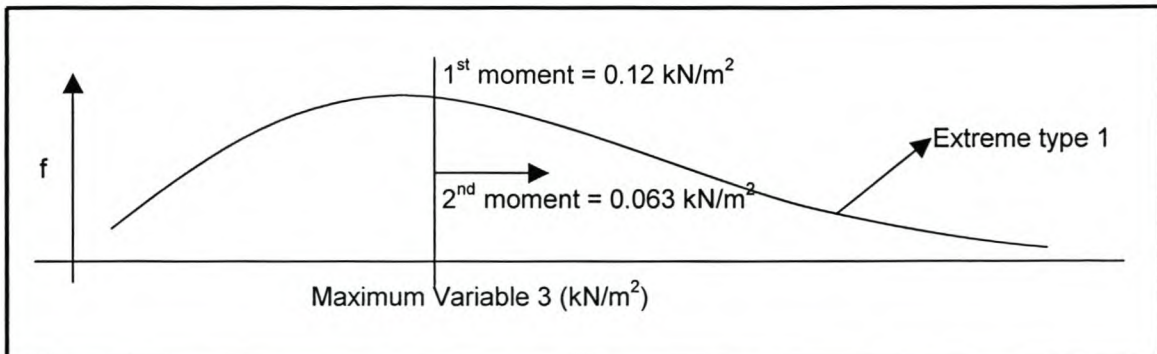


**Figure 37. Probabilistic Model for Maximum Variable 3, Maximum Over-Stacking of Steel Sheets**

The first moment of the *Maximum Over-Stacking of Steel Sheets* is significantly larger than that of the *Average Over-Stacking of Fibre-Cement Sheets*. However, the second moment of the *Average Over-Stacking of Fibre-Cement Sheets* is much larger than that of *Maximum Over-Stacking of Steel Sheets*. This, together with the fact that the two combinations are represented by different types of distributions (the one being extreme type 1 and the other lognormal), complicates the comparison. It is not directly possible to ascertain which one is the more conservative and therefore both models will be used in the subsequent reliability study. In the reliability context it will become clear as to which one is the more conservative.
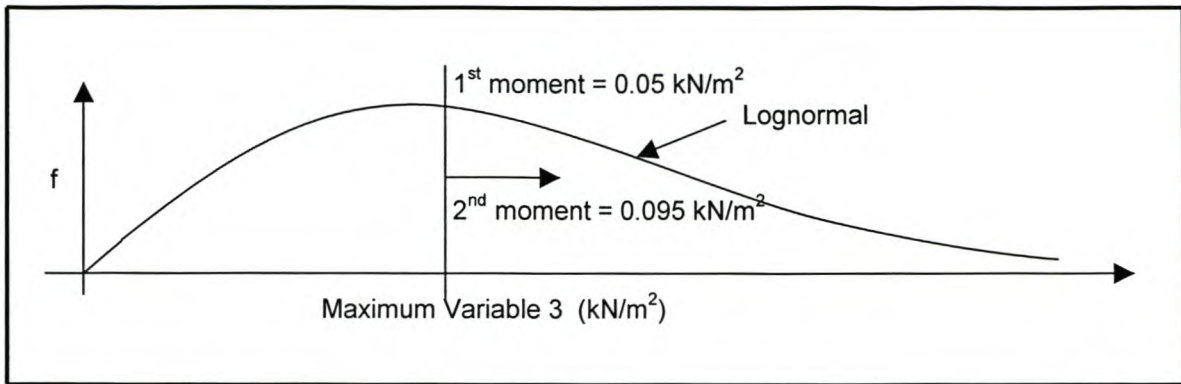
**Figure 38. Probabilistic Model for Maximum Variable 3, Average Over-Stacking of Fibre-Cement Sheets**

### 5.3.4 The Stacking of Roof Cladding on Purlins

Considering Table D1 in Appendix D, it is apparent that the unanimous opinion of the experts is that the SABS loading code is not to provide for the stacking of cladding on purlins. The stacking on the frames may be justified by recognising that this provides for easy installation of the sheeting, as well as the fact that the frames have a much higher load carrying capacity than the purlins do. Stacking of roof cladding on purlins has no benefit for the erection process and may be regarded as negligence from the contractor. Therefore this load mechanism is not modelled as an imposed load for which the SABS loading code has to cater for. The implications of this on the outcome of the investigation are discussed further in Section 8.2.

### 5.4 Maximum Variable 4: Maximum number of Maintenance Workers on a Frame

The derivation of the probabilistic model of Maximum Variable 4 is done strictly in accordance with that of Maximum Variable 1. The procedure is not repeated here and only the final results are given (see Appendix G for the calculations).

The $x_1$ to $x_3$ - values for Maximum Variable 4 are:

$$x_1 = 1.6 \text{ workers / 20m spanning frame}$$
$$x_2 = 2.5 \text{ workers / 20m spanning frame}$$
$$x_3 = 3.3 \text{ workers / 20m spanning frame}$$

In comparison with the number of construction workers on a frame (Maximum Variable 1) the values shown above are much smaller.

The same methodology as for Maximum Variable 1 is now implemented to firstly convert the quantiles to EUDL's, and secondly establish the probabilistic model for the two spacings of frames considered. The results are shown in Figures 39 & 40.
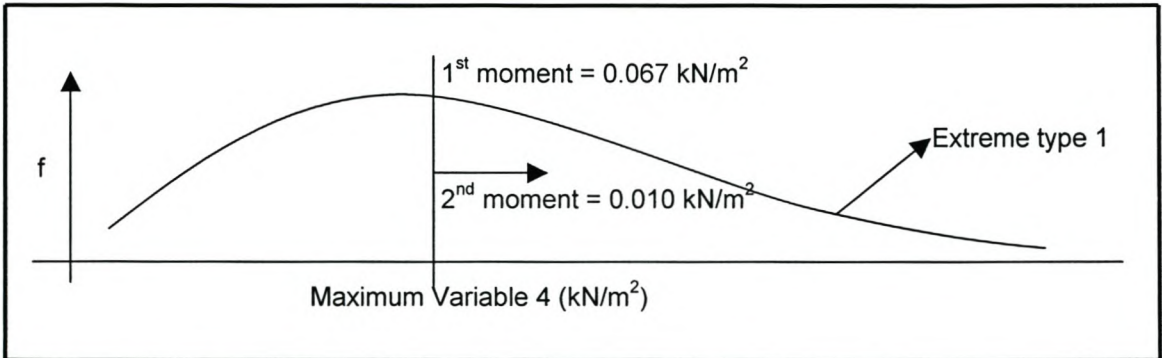


**Figure 39. Probabilistic Model for Maximum Variable 4 for 5m Frame Spacing**
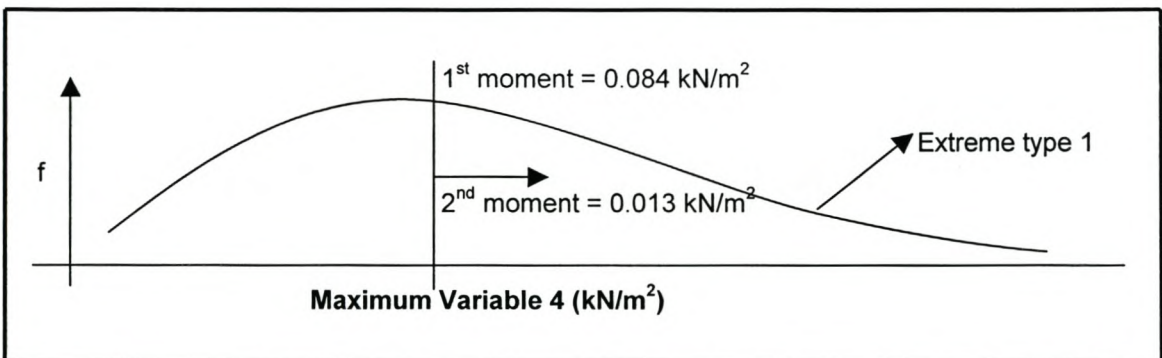


**Figure 40. Probabilistic Model for Maximum Variable 4 for 4m Frame Spacing**

## 5.5 Maximum Variable 5: Maximum number of Maintenance Workers on a Purlin

The derivation of the probabilistic model of Maximum Variable 5 is done strictly in accordance with that of Maximum Variable 2. The procedure is not repeated here and only the final results are given (see Appendix G for calculations).

The $x_1$ to $x_3$ - values for Maximum Variable 5 are:

$$x_1 = 1.2 \text{ workers}$$
$$x_2 = 1.5 \text{ workers}$$
$$x_3 = 1.7 \text{ workers}$$

In comparison with the number of *construction* workers on a purlin (Maximum Variable 2) the values shown above are much smaller. The same methodology as for Maximum Variable 2 is now implemented to firstly convert the quantiles to EUDL's, and secondly establish the probabilistic model for the two tributary areas considered. The results are shown in Figures 41 & 42:
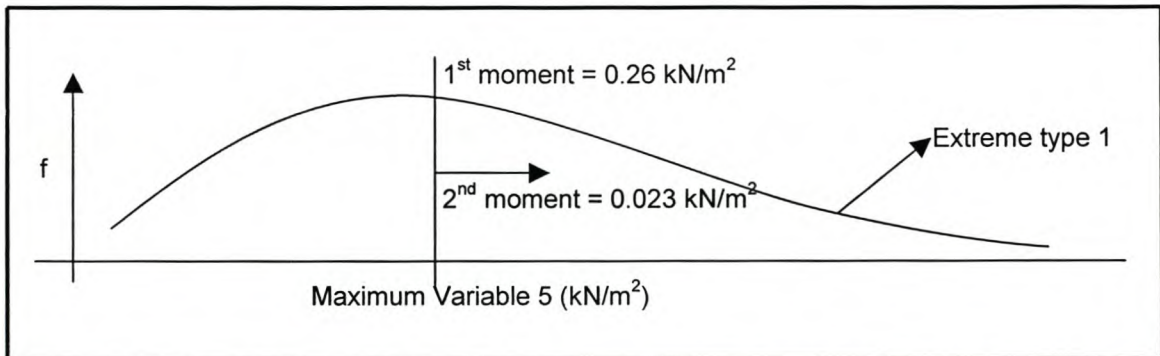


**Figure 41. Probabilistic Model for Maximum Variable 5 for 5m Frame Spacing, 1.7m Purlin Spacing**
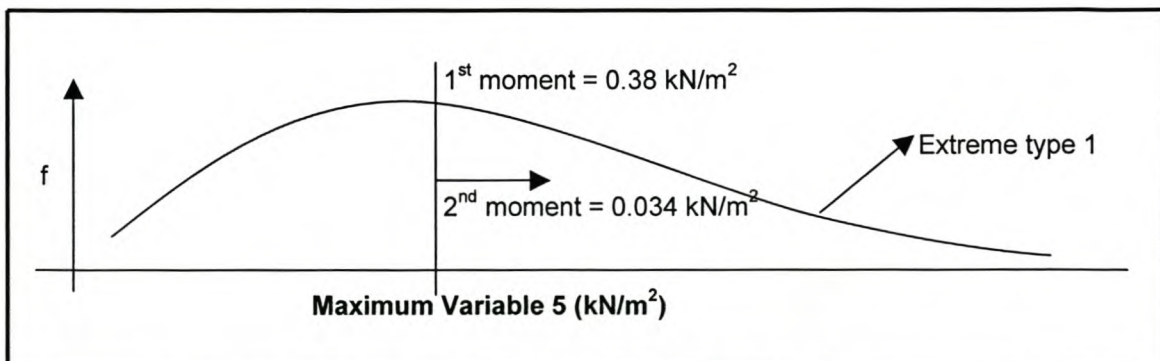


**Figure 42. Probabilistic Model for Maximum Variable 5 for 4m Frame Spacing, 1.4m Purlin Spacing**

## CHAPTER 6: RESULTING PROBABILISTIC LOAD MODELS

In the foregoing chapter, a substantial amount of probabilistic models were developed for different imposed load mechanisms and building geometries. The evolution and establishment of the different load models are summarised in the flow-charts presented in Figure 43 for the construction load, and Figure 44 for the maintenance load.

The load models for the *average loads* as presented in Figures 43 & 44 are derived from the observations made during the construction site survey. The first two moments for the load models are established in the same way as is explained in Section 5.3.2, where the uncertainty is obtained from the sum of the inherent variability and the sample uncertainty of the load. It is important to recognise that the load models for the average loads differ from that of the seed variables in that only the *sample uncertainty* is incorporated for the seed variables as explained in Section 4.2.1, whilst for the average load models the *total uncertainty* (sample as well as inherent uncertainty) pertains. Note that the *average load* for the *stacking of roof cladding* is obtained by combining the average number of bays' cladding stacked on the frames with the weight of steel cladding = 0.1 kN/m$^2$ (which is considered to be the most common type of cladding used). Refer to Appendix G for the derivation of the *average load models.*

At this stage it is important to understand the relationship between the average and 95% maximum load models as shown in Figures 43 & 44. The mean value of the 95% maximum load model could have been determined through extrapolation of the average load model as obtained through the construction site survey (i.e. the 95% - value of the average load model). However, this investigation is based on the principle that a more accurate representation of the 95% maximum value and the uncertainty associated therewith is obtained by directly establishing the 95% - value, and *not* by determining the "implied" 95% value from the average load model. Therefore, the mean value of a maximum load model is the average taken over the maximum values emerging from a number of sets of 20 buildings each (i.e. the average of the one-in-20 building maximum values). The uncertainty lies in the fact that for different sets of 20 buildings, different maximum values would emerge.
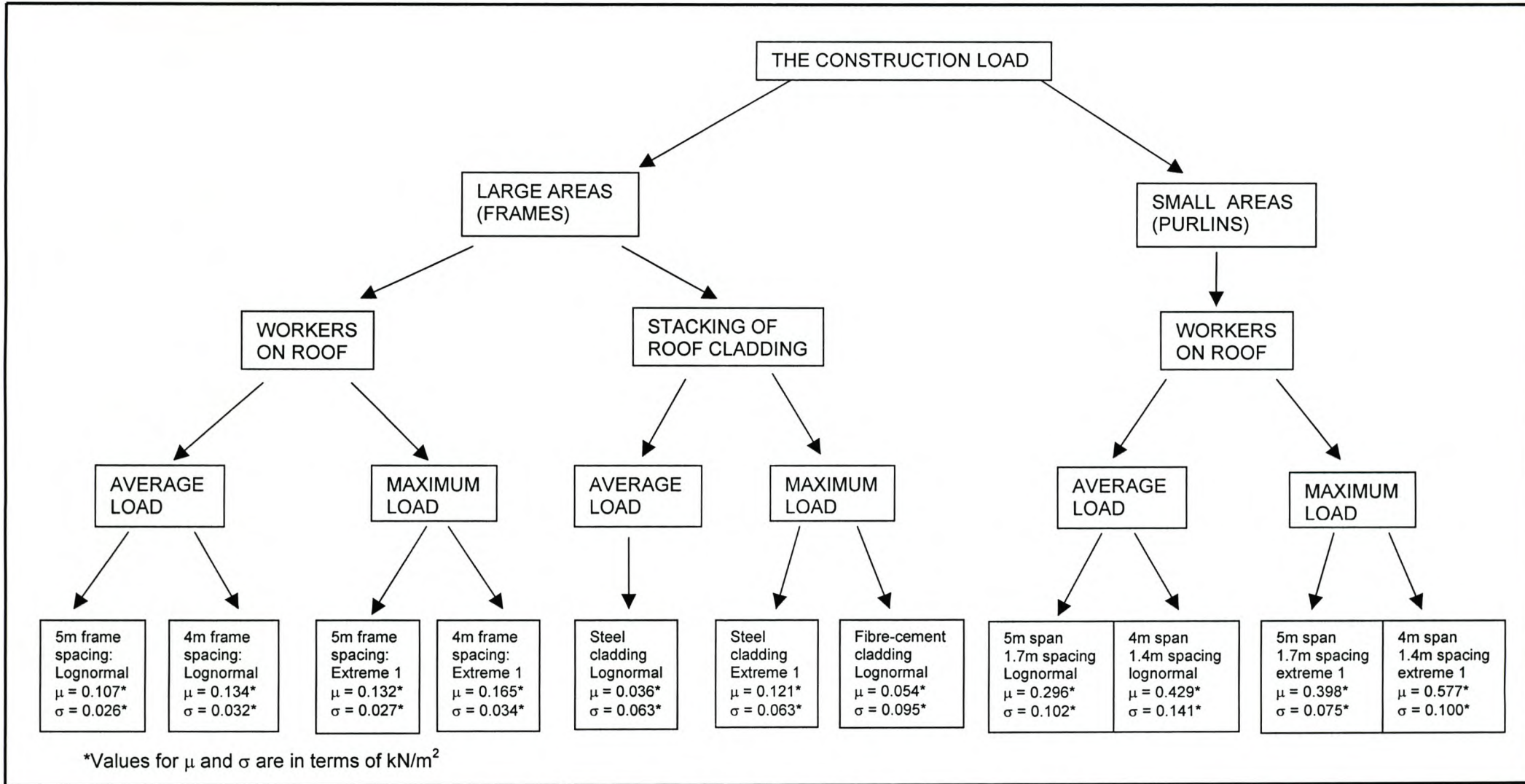
**Figure 43. Probabilistic Models for the Construction Load Mechanisms**
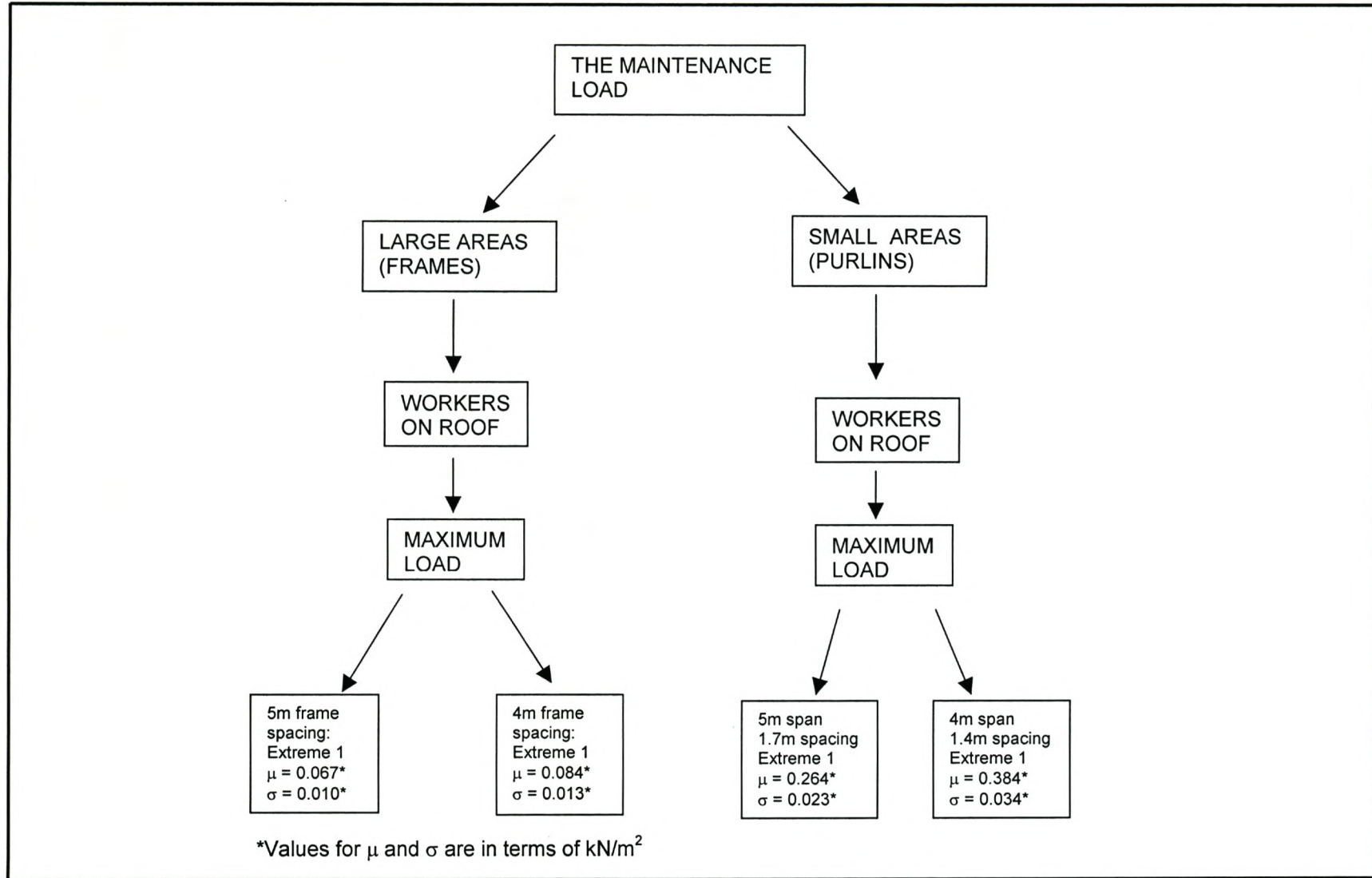
**Figure 44. Probabilistic Models for the Maintenance Load Mechanisms**

It would be advantageous if the extensive collection of load models shown in Figures 43 & 44 are reduced to a more concise collection in order to represent load models that can be implemented in *general*, i.e. that would cover a wider spectrum of building geometries and load mechanisms. The load models are to be reduced to the following:

- The load due to construction workers for large tributary roof areas.
- The load due to stacked materials for large tributary roof areas.
- The load due to construction workers for small tributary roof areas.
- The load due to maintenance workers for large tributary roof areas.
- The load due to maintenance workers for small tributary roof areas.

Furthermore, it would be advantageous to have a single load model representing the construction load for large tributary roof areas. The establishment of such a model is done in Section 6.6.

## 6.1 The Load due to Construction Workers for Large Tributary Roof Areas

To define the probabilistic model for the load due to construction workers on large areas, it is necessary to consolidate the different load models which constitutes this load model. Starting at the lowest level shown in Figure 43, the load models for the 5m and 4m - frame spacings are to be combined into one.

The two frame spacings considered may be reduced to one representative model through one of two options:

- *Option 1*. Combine the first two moments by linear interpolation between them. Therefore, $\mu = \mu_{5m} \times 5/\text{spacing}$ and $\sigma = \sigma_{5m} \times 5/\text{spacing}$, so that $\mu$ (spacing = 5m) = $\mu_{5m}$ and $\mu$ (spacing = 4m) = $\mu_{4m}$ and likewise for the second moments.

- *Option 2*. Selecting one of two frame spacings as representative. The 4m frame spacing relates to the smaller tributary area for the frames and is therefore the more conservative of the two models.

*Option 1* would result in a load model for which the first two moments are functions of the frame spacing. This makes the load model case sensitive. However, a more general load model is preferred which would sufficiently cover most cases found in

practice. Therefore, in proposing a general load model, *Option 2* would be the more viable option.

Using the maximum load model obtained for the 4m-frame spacing for the purpose of code calibration and probabilistic design would ensure that the minimum levels of reliability are maintained for most design cases.

The probabilistic model of the imposed load due to construction workers for large tributary roof areas is shown in Figure 45.
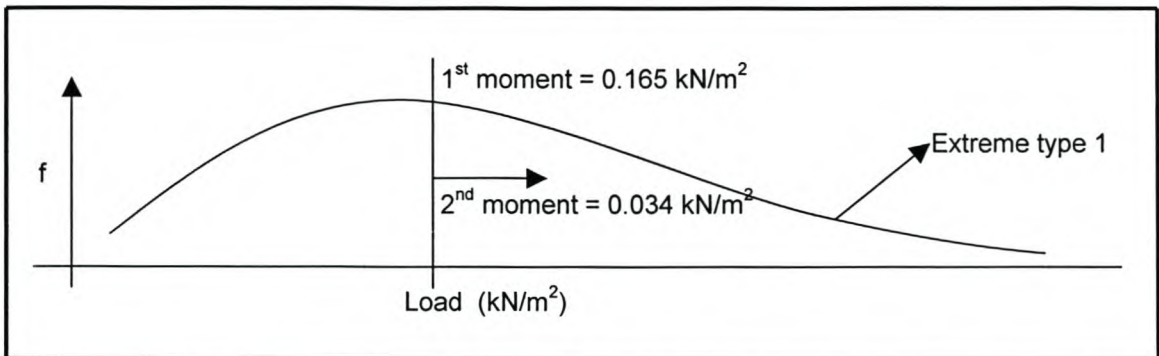


**Figure 45. Probabilistic Model for the Load due to Construction Workers for Large Tributary Roof Areas**

## 6.2 The Load due to Stacked Materials for Large Tributary Roof Areas

Again, the *maximum* load model for this load mechanism is to apply. In selecting between the maximum load for steel cladding and that for fibre-cement cladding (as determined in Sections 5.3.1 & 5.3.2 respectively), one of two options can again be utilised:

- *Option 1*. The most conservative load model is to be used. The conservatism of each model is determined by comparing the 95% characteristic values. The 95% - value for the steel cladding load model it is 0.239 $kN/m^2$ and for the fibre-cement cladding load model is 0.187 $kN/m^2$ (refer to Appendix G). The steel cladding model is therefore the most conservative, having the larger 95% characteristic value.

- *Option 2*. Simply use the steel cladding load model. The reason being that this is by far the more common type of cladding used in practice.

Both of the options in the above yield the same result – which is that the steel cladding load model is to be used. Therefore, in proposing a load model representative of the imposed load due to stacked materials for the purpose of code calibration and probabilistic design, the maximum load model for steel cladding is proposed.

The probabilistic model of the imposed load due to stacked materials for large tributary roof areas is shown in Figure 46.
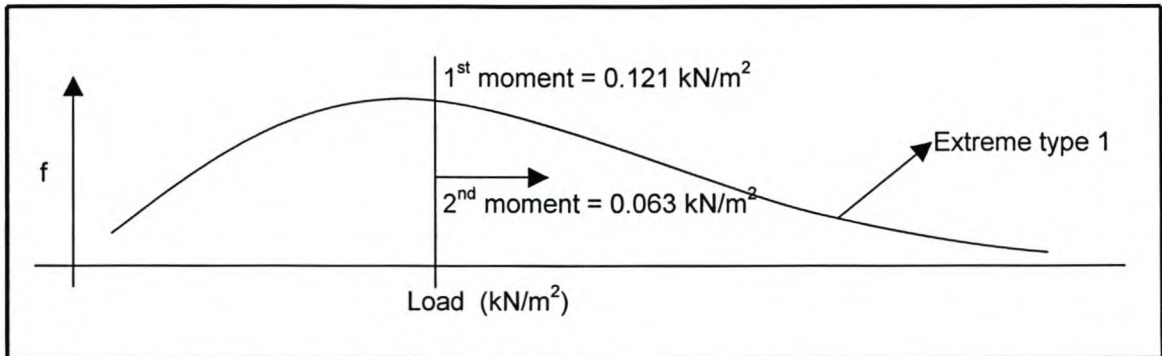


**Figure 46. Probabilistic Model for the Load due to Stacked Materials for Large Tributary Roof Areas**

## 6.3 The Load due to Construction Workers for Small Tributary Roof Areas

Referring to Figure 43, it is not clear whether the maximum load model is actually more conservative than the average load model as determined through the construction site survey. The one model has a larger first moment and the other a larger second moment. The 95% - value for the maximum model for the larger tributary area is 0.537 kN/m$^2$ and for the average model is 0.486 kN/m$^2$ (refer to Appendix G). Although the 95% - values for the two models are rather close, the value for the maximum model is larger, which suggests that the maximum load model is indeed the maximum model.

In consolidating between the two areas considered, the approach of taking the more conservative route is again applied. Therefore the load model for the smaller area (1.4 x 4m) is used as the representative load model.

The probabilistic model of the imposed load due to construction workers for small tributary roof areas is shown in Figure 47.
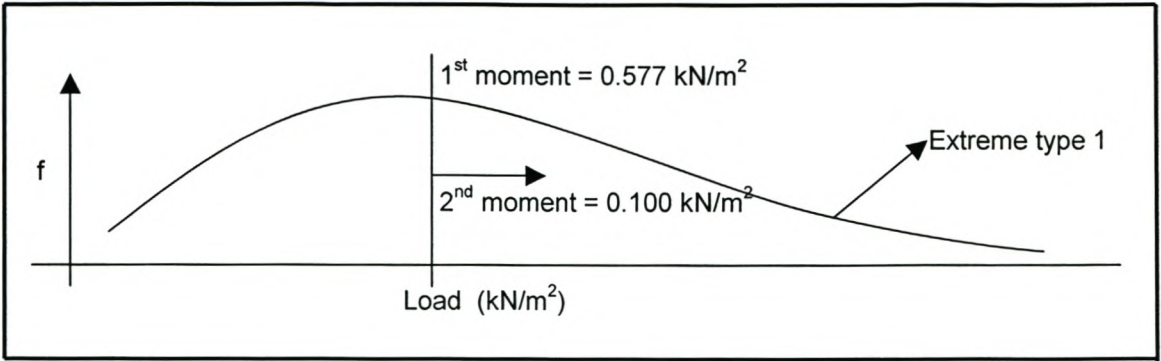
**Figure 47. Probabilistic Model for the Load due to Construction Workers for Small Tributary Roof Areas**

## 6.4 The Load due to Maintenance Workers for Large Tributary Roof Areas

Applying the same methodology as in Section 6.1, the following load model is obtained:
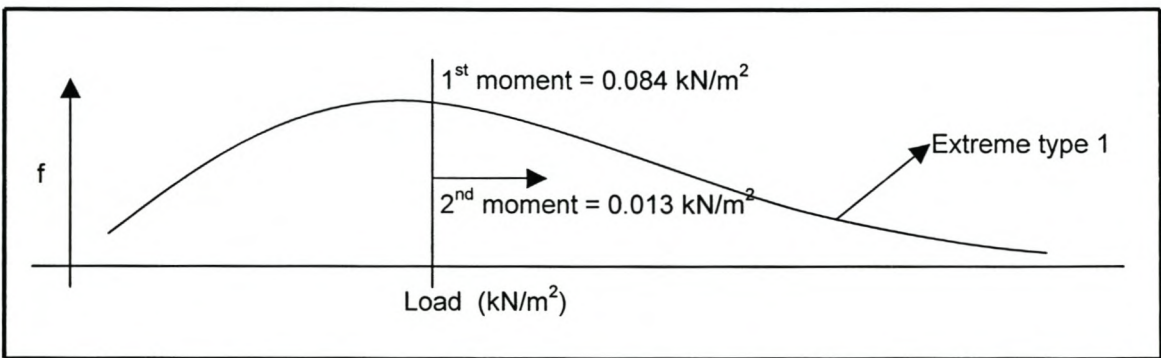


**Figure 48. Probabilistic Model for the Load due to Maintenance Workers for Large Tributary Roof Areas**

## 6.5 The Load due to Maintenance Workers for Small Tributary Roof Areas

Since no load survey was conducted for this load mechanism, the only load model available is that obtained from the expert survey. The subsequent model is presented in Figure 49, again for the smaller tributary roof area.
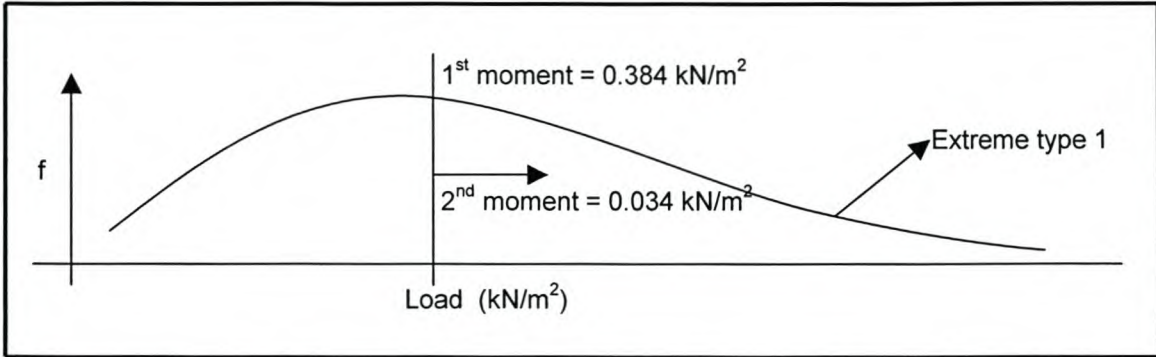


**Figure 49. Probabilistic Model for the Load due to Maintenance Workers for Small Tributary Roof Areas**

## 6.6 The Construction Load for Large Tributary Roof Areas

The loads due to the stacking of roof cladding and construction workers on the frame take place simultaneously on the roof. Subsequently, the purpose of this section is to consolidate the load models for the stacking of roof cladding and that of workers on the roof for large tributary areas (Sections 6.1 & 6.2) into one.

From Section 6.1, the load model for the maximum number of workers is that for the 4m-frame spacing. From Section 6.2, the load model chosen to represent the maximum over-stacking is that for the maximum number of bays of steel cladding. The events of maximum number of workers on the roof and maximum over-stacking of roof cladding are unlikely occurrences. Therefore, to assume that the maximum over-stacking and maximum number of workers would take place on the same building would be unrealistic and overly conservative. The approach applied is to combine the maximum event of the one variable with the average (and more common situation) of the other variable. Note that this is a version of *Turkstra's Rule*, the only difference being that the dimension for application of Turkstra's Rule is *time* (maximum lifetime load combined with arbitrary-point-in-time load), while for this instance the dimension is *case* (per building). Also, the construction loads are assumed to be once-off

occurrences and therefore independent of time, and only dependent on building. The following two combinations apply:

- The maximum number of workers (4m-frame spacing) and average over-stacking of steel cladding.
- The average number of workers (4m-frame spacing) and maximum over-stacking of steel cladding. Note that by using the 4m-frame spacing load model for the average number of workers, some conservatism is built into this combination.

## 6.6.1 Maximum Number of Construction Workers combined with Average Over-stacking of Roof Cladding

The load due to the maximum number of construction workers on the frame is to be combined with the load due to the average amount of over-stacking of cladding on the frame. The probabilistic model for the average over-stacking has been obtained from the construction site survey (see Appendix G) and is shown in Figure 43. The weight of the sheets is taken as 0.1 $kN/m^2$, which is the upper limit of the range of weights for steel sheets – a conservative assumption. The probability parameters for the two variables to be combined are shown in Table 40.

**Table 40. Parameters of the Variables for the First Combination**

|  | Maximum Workers on Frame | Average Over-Stacking of Cladding on Frame |
|---|---|---|
| Distribution Type | Extreme type 1 | Lognormal |
| 1st Moment ($kN/m^2$) | 0.165 | 0.036 |
| 2nd Moment ($kN/m^2$) | 0.034 | 0.063 |

The combination is based on the first-order second moment formulation developed by *Cornell* and *Ang & Cornell* (ANG and TANG (1984)). Introduce the performance function:

$$g(X) = c^* - (MW + AS) \tag{47}$$

where  MW    = the load due to the Maximum Workers on the frame

AS    = the load due to the Average over-Stacking of cladding on the frame

$c^*$    = a realisation of the combination $C_{MW+AS}$ = MW + AS

Different limit state equations g(X) = 0 can now be obtained for different values of c*, with each limit equation implying that MW + AS =c*. g(X) > 0 implies that (MW + AS) < c*, which is referred to as the "safe state" in reliability terms. The probability that g(X) > 0 can now be determined for each c* through first-order second moment interpretation, and so doing the cumulative probabilities F((MW + AS) ≤ c*) are obtained at each c*. MW + AS is subsequently denoted as $C_{MW+AS}$. The applied procedure is set forth in the following:

### The Cumulative Distribution of $C_{MW+AS}$

The probability that $C_{MW+AS}$ is smaller than c* is given by

$$F(C_{MW+AS} \leq c^*) = \Phi(\beta) \tag{48}$$

$$\text{where} \quad \beta = \frac{\mu_g}{\sigma_g} \tag{49}$$

$$\mu_g = c - (\mu_{MW} + \mu_{AS})$$

$$\sigma_g = \sqrt{\sigma_{MW}^2 + \sigma_{AS}^2}$$

Φ = cumulative probability function for a standardised normal random variable

Equation (48) only applies if the random variables MW and AS are uncorrelated normal variates. MW and AS are uncorrelated but since they are non-normal, equivalent normal distributions have to be obtained at the design points for these variates. The design points are the most likely (or expected) realisations of MW and AS ($x_{MW}$* and $x_{AS}$*) which will induce the limit state g(X) = 0. Since β is dependent on the equivalent normal distributions of the variates MW and AS, which are in turn dependent on $x_{MW}$* and $x_{AS}$*, implicitly this means that β is dependent of $x_{MW}$* and $x_{AS}$*. The x* - values are dependent on β through the following relationship:

$$x_i^* = \mu_X^N + \alpha_i \beta \sigma_X^N \tag{50}$$

where $\mu_X^N$ & $\sigma_X^N$ = the first two moments for the equivalent normal variates

$\alpha_i$ = the direction cosine for MW (i=1) and AS (i=2).

This inter-dependency of β on x* and in turn x* on β means that $x_{MW}$*, $x_{AS}$* and β is to be determined through performing a number of iterations of Equations (49) & (50) until

sufficient convergence is obtained. The equivalent normal distribution at x* is determined by finding $\mu_X^N$ and $\sigma_X^N$ for which the cumulative probability at x*, namely $F_N(x^*)$ is equal to the cumulative probability $F_X(x^*)$ of the original variate.

The following relations apply:

$$\mu_X^N = x^* - \sigma_X^N \Phi^{-1}\left[F_X(x^*)\right] \tag{51}$$

$$\sigma_X^N = \frac{\phi\left\{\Phi^{-1}\left[F_X(x^*)\right]\right\}}{f_X(x^*)} \tag{52}$$

where $\Phi^{-1}$ = the inverse of the cumulative probability function of the standardised normal distribution

$\phi$ = probability density function of the standardised normal distribution

Equations (51) & (52) are used to solve for $\sigma_{MW}^N$ and $\mu_{MW}^N$, as well as $\sigma_{AS}^N$ and $\mu_{AS}^N$ whereby the equivalent normal distributions of MW and AS are defined through the first two moments.

As an initial step, $x_{MW}^*$ and $x_{AS}^*$ are chosen as $\mu_{MW}$ and $\mu_{AS}$ respectively and Equations (52), (51), (49) and (50) are applied in that order to find $\beta$ and the "new" $x_{MW}^*$ and $x_{AS}^*$ - values. Using these "new" $x_{MW}^*$ and $x_{AS}^*$ - values and repeating the process a different $\beta$ - value is obtained. This process is repeated until sufficient convergence is obtained for $\beta$ and the design point. The probability that $C_{MW+AS}$ is smaller than c* is now calculated through Equation (48).

A spreadsheet programme COMBAN was developed to evaluate $F(C_{MW+AS} < c^*)$ for different c* - values (refer to Appendix I). Since it is essentially the upper region, i.e. the larger values of $C_{MW+AS}$ that is of interest, the c* - values should be so chosen as to reflect this. The c* - values are chosen as 0.18, 0.2, 0.25, 0.3, 0.35, 0.4, 0.45 and 0.5 kN/m². Comparison with the sum of the two first moments of MW and AS = 0.165 + 0.036 = 0.201 kN/m² shows that the selected c* - values tend towards the upper region of $C_{MW+AS}$.

The results of the combination for the above c* - values are shown in Table 41. $F(C_{MW+AS} < c^*)$ is evaluated at each value of c* through Equation (48). Refer to Appendix I for the numerical calculations.

### Table 41. Point Cumulative Probabilities of $C_{MW+AS}$

|  | c* =0.18 | c* = 0.2 | c* =0.25 | c* = 0.3 | c* =0.35 | c* = 0.4 | c* =0.45 | c* = 0.5 |
|---|---|---|---|---|---|---|---|---|
| F($C_{MW+AS}$ < c*) | 0.49832 | 0.72998 | 0.90968 | 0.95755 | 0.97645 | 0.98555 | 0.99051 | 0.99345 |

* c* - values are in terms of $kN/m^2$

The point cumulative probabilities obtained in Table 41 are used to compare $C_{MW+AS}$ with the combination of the load due to the maximum over-stacking and the average number of workers on the frame. The latter load combination is established in the subsequent section and the comparison is drawn in Section 6.6.3.

### 6.6.2 Average Number of Construction Workers combined with Maximum Over-stacking of Roof Cladding

The load due to the average number of construction workers on the frame is to be combined with the load due to the maximum amount of over-stacking of cladding on the frame. This combination is subsequently denoted as $C_{AW+MS}$. The probabilistic model for the average number of construction workers AW has been obtained from the construction site survey (see Appendix G) and is shown in Figure 43. The statistics for two variables to be combined are put forward in Table 42.

### Table 42. Parameters of the Variables for the Second Combination

|  | Average Workers on Frame AW | Maximum Over-Stacking of Cladding on Frame MS |
|---|---|---|
| Distribution Type | Lognormal | Extreme type 1 |
| 1st Moment $(kN/m^2)$ | 0.134 | 0.121 |
| 2nd Moment $(kN/m^2)$ | 0.032 | 0.063 |

Proceeding in the same way as for the first combination (Section 6.6.1), one can now obtain the combined cumulative distributions for $C_{AW+MS}$ through first-order second moment formulation. The point cumulative probabilities are calculated at the selected c* - values as in Section 6.6.1, and the results shown in Table 43.

### Table 43. Point Cumulative Probabilities of $C_{AW+MS}$

|  | c* =0.18 | c* = 0.2 | c* =0.25 | c* = 0.3 | c* =0.35 | c* = 0.4 | c* =0.45 | c* = 0.5 |
|---|---|---|---|---|---|---|---|---|
| F($C_{AW+MS}$ < c*) | 0.00297 | 0.03658 | 0.50457 | 0.90579 | 0.99009 | 0.99920 | 0.99994 | 0.99999 |

* c* - values are in terms of $kN/m^2$

### 6.6.3 Comparison of Results and Selection of a Representative Model

The purpose of this section is to select the most conservative of $C_{MW+AS}$ and $C_{AW+MS}$ as determined in Sections 6.6.1 & 6.6.2 respectively. The model so selected is then to serve as the representative model for the construction load for large tributary roof areas.

The cumulative probabilities obtained for the two combinations are compared in Table 44 at the selected c* - values.

**Table 44. Point Cumulative Probabilities of $C_{MW+AS}$ & $C_{AW+MS}$**

|  | c* =0.18 | c* = 0.2 | c* =0.25 | c* = 0.3 | c* =0.35 | c* = 0.4 | c* =0.45 | c* = 0.5 |
|---|---|---|---|---|---|---|---|---|
| $F(C_{MW+AS} < c*)$ | 0.49832 | 0.72998 | 0.90968 | 0.95755 | 0.97645 | 0.98555 | 0.99051 | 0.99345 |
| $F(C_{AW+MS} < c*)$ | 0.00297 | 0.03658 | 0.50457 | 0.90579 | 0.99009 | 0.99920 | 0.99994 | 0.99999 |

\* c* - values are in terms of kN/m²

From Table 44 one may conclude that $C_{AW+MS}$ has the larger first moment of the two (0.25 kN/m² vs 0.18 kN/m²), and that $C_{MW+AS}$ has the larger second moment. The 95% characteristic value for $C_{MW+AS}$ is 0.29 kN/m², and for $C_{AW+MS}$ is 0.315 kN/m², which suggests that $C_{AW+MS}$ is the more conservative of the two. However, for the range of cumulative probabilities of 0.97 and upwards, $C_{MW+AS}$ yields the larger values, as is shown in Figure 50.
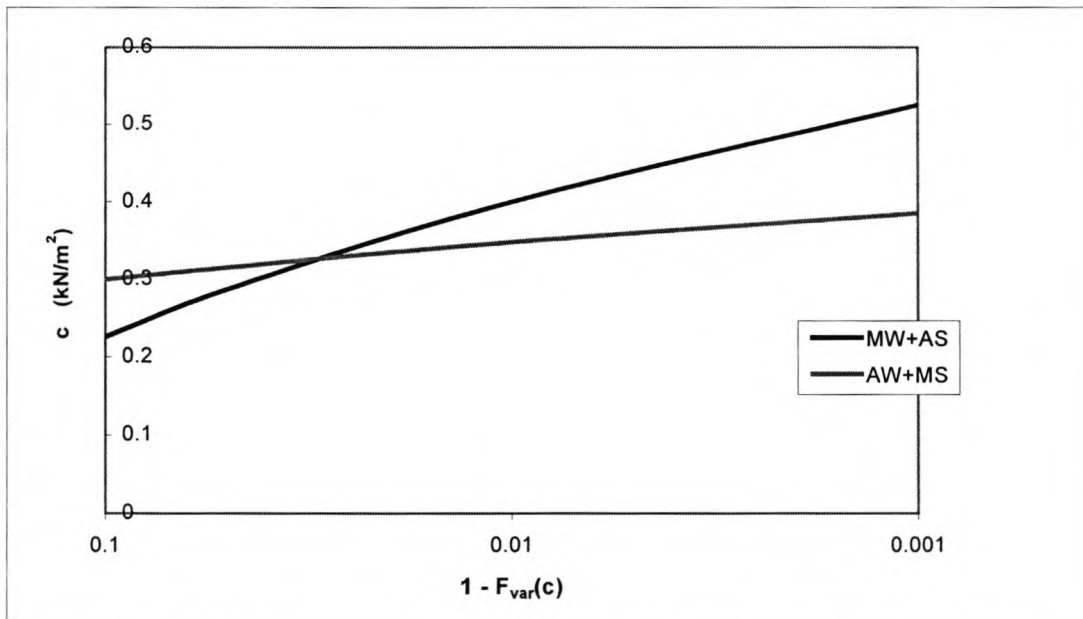


**Figure 50. Comparison of $C_{MW+AS}$ & $C_{AW+MS}$ for the upper range of values**

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It is this upper range of values that is of importance for reliability analyses and $C_{MW+AS}$ therefore prevails as the more conservative of the two. This is verified through the following:

When conducting a reliability analysis one would typically aim for a minimum level of reliability of $\beta = 3$. Introduce the linear performance function:

$$g = R - D - L \qquad (53)$$

where  R = the random variable for the resistance

D = the random variable for the dead load

L = the random variable for the imposed load

The average value of R can be calculated which will ensure that $\beta = 3$. By substituting L in Equation (53) with either the combination $C_{MW+AS}$ or $C_{AW+MS}$, different values for R are obtained which will enforce this reliability. It is argued that the combination which results in the largest R, is the most conservative. The effect of the dead load is accounted for by performing the analysis for three instances of the dead load representing the range of reasonable values for the dead load, i.e. the minimum value, the common value and the maximum value.

The statistics for the variables to be used in the analyses are shown in Tables 45 & 46 for the analysis with $C_{MW+AS}$ and $C_{AW+MS}$ respectively. It is assumed that R, D and L are independent variates. The values chosen for D at which to evaluate R are 0.20, 0.35 and 0.50 $kN/m^2$, constituting a range for D of 0.20 to 0.50 $kN/m^2$ (including the dead load of the roof cladding and weight of services). The coefficients of variation (c.o.v.'s) are taken as 0.15 for R and 0.1 for D. The values for the c.o.v.'s are obtained from the NBS 577 (1980).

**Table 45. Statistics for the Reliability Analyses with $C_{MW+AS}$**

| | R | D | $C_{MW+AS}$ | |
| --- | --- | --- | --- | --- |
| | | | Maximum Workers MW | Average over-Stacking AS |
| Distribution | Lognormal | Lognormal | Extreme type 1 | Lognormal |
| C.o.v.'s | 0.15 | 0.10 | 0.206 | 1.75 |
| $1^{st}$ moment $(kN/m^2)$ | To be determined | 0.20, 0.35, 0.50 | 0.165 | 0.036 |
| $2^{nd}$ moment $(kN/m^2)$ | $0.15 \times 1^{st}$ moment | $0.1 \times 1^{st}$ moment | 0.034 | 0.063 |

**Table 46. Statistics for the Reliability Analyses with $C_{AW+MS}$**

| | R | D | $C_{AW+MS}$ | |
| --- | --- | --- | --- | --- |
| | | | Average Workers AW | Maximum over-Stacking MS |
| Distribution | Lognormal | Lognormal | Lognormal | Extreme type 1 |
| C.o.v.'s | 0.15 | 0.10 | 0.239 | 0.521 |
| $1^{st}$ moment $(kN/m^2)$ | To be determined | 0.20, 0.35, 0.50 | 0.134 | 0.121 |
| $2^{nd}$ moment $(kN/m^2)$ | $0.15 \times 1^{st}$ moment | $0.1 \times 1^{st}$ moment | 0.032 | 0.063 |

A spreadsheet programme RELAN was developed to perform the reliability analyses (see Appendix J). The results of the analyses are shown in Table 47 for $\beta = 3$.

**Table 47.  Resistance's to maintain $\beta = 3$ over range of Dead Load Values**

| Dead Load $(kN/m^2)$ | Resistance required for $C_{MW+AS}$ $(kN/m^2)$ | Resistance required for $C_{AW+MS}$ $(kN/m^2)$ |
| --- | --- | --- |
| 0.20 | 1.00 | 0.88 |
| 0.35 | 1.17 | 1.10 |
| 0.50 | 1.35 | 1.33 |

From Table 47 it is observed that $C_{MW+AS}$ requires the larger resistances over the range of dead loads to maintain a reliability level of $\beta = 3$, and hence it is concluded that $C_{MW+AS}$ is the more conservative variable.  As expected, the difference between the resistances required for the two combinations reduces as the dead load increases. This is due to the effect of the different imposed load variables being suppressed by the larger contribution of the dead load.  In proposing an imposed roof load model to be used for probabilistic design, $C_{MW+AS}$ would yield the more conservative results when aspiring for a target $\beta = 3$.

### 6.6.4 The Effect of the Lognormal Distribution function.

When the aforementioned reliability analyses (see Section 6.6.3) were conducted, an anomaly was discovered.  This anomaly lies in the effect of the lognormal distribution on the reliability where the first moment is small and the second moment large, i.e. the coefficient of variation is large (even larger than 1).

When the reliability analysis was performed for $C_{MW+AS}$, it was recognised that for a local increase in the first moment of AS from 0.036 to 0.13 $kN/m^2$, the reliability of the

system (with $\mu_R$ = 1.00 kN/m² and $\mu_D$ = 0.20 kN/m²) increases from β = 3 to β = 3.15, whereafter β starts to decrease again for further increase in the first moment of AS. This is contra-intuitive since one would expect the reliability to constantly decrease as any of the load variables, i.e. variables with negative direction cosines, increases. Modelling of AS as an extreme type 1 or as a normal random variable does not show this effect, which suggests that there is a discrepancy in the mechanism which involves the calculation of the reliability when the lognormal distribution is used for variables with high coefficients of variation.

This discrepancy is explained by recognising the constraint placed on the lognormal distribution of $f_{LN}(0)$ = 0, i.e. the probability density function has to take on a value of zero when the lognormal variable has a zero value. This constraint forces the shape of the distribution to change when the first moment increases. The effect of this altered shape of the distribution is felt most severely in the upper tail region of the variable (the larger values), and it is in this region where the design value falls when conducting a reliability analysis with β = 3. This upper region is even more relevant when the contributions of the other load variables is small in terms of inducing the failure state, in other words the lognormal variable has by far the dominating direction cosine in producing failure. Therefore, this anomaly is negated as the other load variables increase or the resistance decreases, i.e. the "distance" between the average values of the resistance and the sum of the loads decreases. The use of a *shifted lognormal distribution* where the shape of the distribution is kept the same and the first moment increased would not result in such an anomaly.

Furthermore, modelling of AS as an extreme type 1 or a normal variable, and everything else kept constant as in Table 45 with $\mu_R$ = 1 kN/m² and $\mu_D$ = 0.2 kN/m² yields β - values of 3.61 and 3.68 respectively. This is significantly higher than β = 3 for the case where AS is lognormally distributed, which further suggests that the lognormal distribution in this instance produces overly conservative results in reliability analyses. It is however noted that this over-conservatism due the lognormal distribution is suppressed as the dead load increases. The amount of over-conservatism is measured by comparison with the imposed load model for the maximum amount of over-stacking MS on the roof (see Figure 46).

The statistics for this variable is:

Distribution function:  extreme type 1

1st moment:               0.121 kN/m$^2$

2nd moment:              0.063 kN/m$^2$

This maximum variable is used instead of AS in Table 45 and the required resistance is calculated to meet the target $\beta$ = 3 over the range of dead load values.   The combination of the imposed load when the maximum over-stacking load MS is used, is denoted as $C_{MW+MS}$.  The results are compared with that of the required resistance when $C_{MW+AS}$ is used (as calculated in the previous section), and shown in Figure 51.
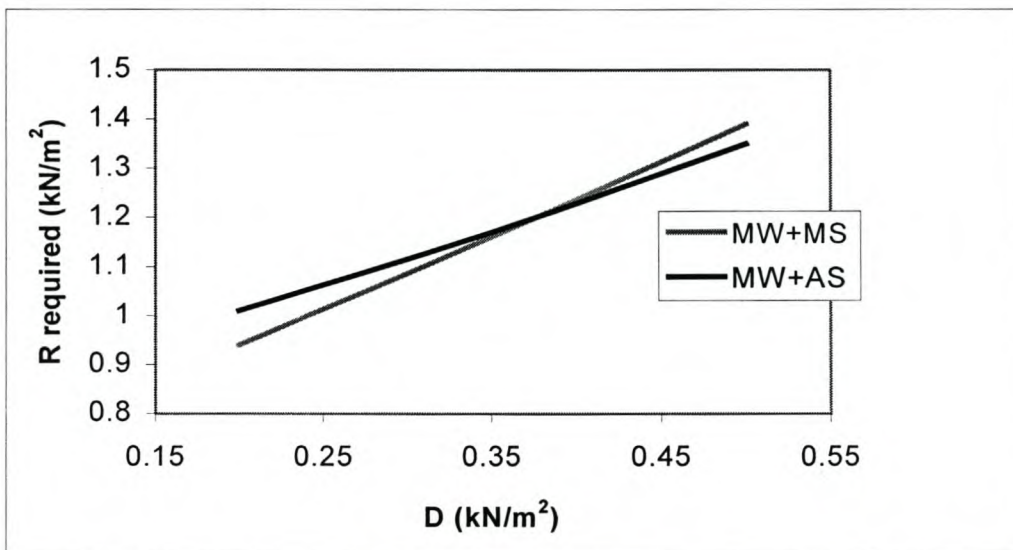


**Figure 51. Comparison of the required Resistance to meet $\beta$ = 3 for $C_{MW+AS}$ and $C_{MW+MS}$**

Figure 51 shows that for dead load values smaller than 0.38 kN/m$^2$ ($\approx$ 0.35 kN/m$^2$, i.e. the common value), $C_{MW+AS}$ requires a larger resistance to meet $\beta$ = 3.   This is in contrast to what one would expect from considering the statistics for AS and MS where it is expected that $C_{MW+MS}$ would result in the more conservative values over the full range of dead loads.   It is therefore concluded that for values of the dead load lower than the common value, $C_{MW+AS}$ is overly conservative.

The question now arises of how valid it is to model the average value for the cladding stacked on the roof as a lognormal distribution.  The values for the first two moments of 0.036 and 0.063 kN/m$^2$ respectively, as measured from the construction site survey, combined with the lognormal distribution produces conservative results in reliability

analyses for smaller than average dead loads. However, there is no rationale for changing the distribution to any other one. The argument for using the lognormal distribution is that it realistically models the load since negative values for the load due to stacking of cladding is not possible. Also, as can be seen it produces the most conservative results.

Everything considered, the maximum load due to workers on the roof combined with the average over-stacking of roof cladding results in a load model which is conservative but can generally be regarded as representative. Attention is drawn to the fact that for lower values of the dead load (i.e. for shorter span buildings) this model may tend to provide overly conservative results when performing probabilistic design.

The load combination $C_{MW+AS}$ is proposed as representative of the construction load for large tributary roof areas, and is subsequently denoted as $L_{C,large\ A}$.

### 6.6.5 Generalisation into a Known Probability Function

It would be advantageous if the construction load for large tributary roof areas $L_{C,large\ A}$ could be modelled in terms of a known probabilistic model. Up to now, this load is represented by a series of point cumulative probabilities at chosen values of the load variable as shown in Table 41.

Since the construction load for large tributary areas is essentially a combination of an extreme type 1 and a lognormal random variable (independent) it is assumed that one of these two distributions would best agree with the total construction load for large tributary areas. The complication is that the one dominates in terms of the first moment and the other in terms of the second moment. The method by which it is measured which distribution best agrees with the load is by plotting the cumulative probabilities on extreme type 1 and lognormal probability papers.

### *Extreme Type 1 Probability Paper*

The hypothesis $H_0$: that the distribution function of $L_{C,large\ A}$ is that of an extreme type 1 distribution; is stated. This validity of $H_0$ is tested by means of extreme type 1

probability paper. If $L_{C,large\ A}$ were to be an extreme type 1 random variable, the cumulative probability function $F_{LC,large\ A}(c)$ is given by:

$$F_{LC,large\ A}(c) = \exp(-\exp(\alpha(c - u)))\qquad(54)$$

where u and $\alpha$ = the two parameters of the extreme type 1 distribution

Equation (54) is now expressed in terms of the argument z so that $F_{LC,large\ A}(z)$ is independent of u and $\alpha$:

$$F_{LC,large\ A}(z) = \exp(-\exp(z))\qquad(55)$$

$$\text{where}\ z\ = \alpha(c - u)\qquad(56)$$

Equation (56) defines a linear relation between c and z. From Table 41, $F_{LC,large\ A}(c)$ is known at the selected c* - values. Substituting $F_{LC,large\ A}(c)$ into Equation (55) implies a certain value for z. Therefore, the relation $c \rightarrow F_{LC,large\ A}(c) \rightarrow z$ suggests an indirect relation between c and z via $F_{LC,large\ A}(c)$. The more linear this relation between c and z is, i.e. the more the values of z plotted against those of c would resemble a straight line, the closer the distribution of $L_{C,large\ A}$ is to an extreme type 1 distribution. The values of c and z are plotted in Figure 52, and a linear trend line is fitted through them. The deviation from this trend line is measured in terms of the $R^2$ - value, which is equal to unity if the relation between c and z is perfectly linear.
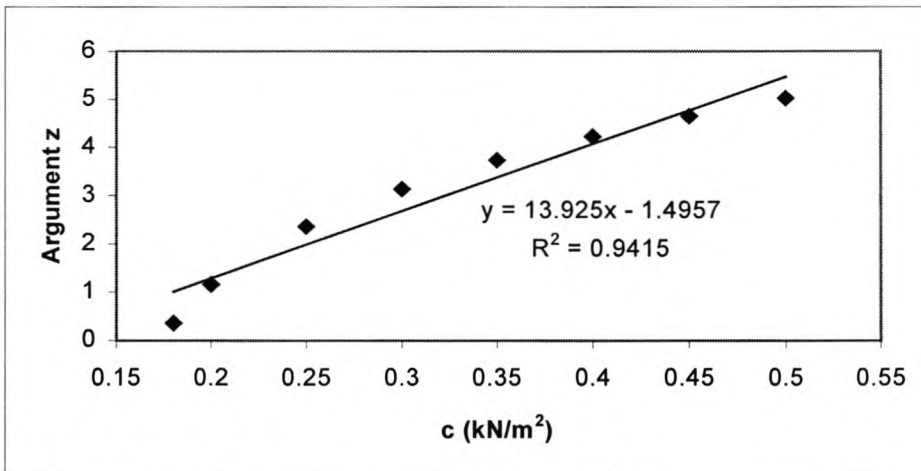


**Figure 52. Extreme Type 1 Probability Paper for $L_{C,large\ A}$**

From Figure 52 it can be seen that the point values at which the cumulative probabilities are calculated and through which the linearisation is done are taken from the average value (0.18 kN/m$^2$) to the 99% characteristic value. It is this upper "half"

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of the distribution that is of importance for probabilistic design purposes and code calibration as it is in this range that the realisation of $L_{C,large\ A}$ which induces failure will take place.

***Lognormal Probability Paper***

The hypothesis $H_1$: that the distribution function of $L_{C,large\ A}$ is that of a lognormal distribution; is stated. This validity of $H_1$ is tested by means of lognormal probability paper. This is done in a similar fashion as was done for the extreme type 1 paper. For the lognormal distribution we have (formulae for the lognormal distribution are obtained from VAN DEVENTER PJU (2000)):

$$F_{L_{C,large\ A}}(c) = \Phi\left[\frac{\log_e(c) - \mu^N}{\sigma^N}\right]$$

(57)

where  $\Phi[\ ]$     = the cumulative probability function of the normal distribution.

   $\mu^N$     = the equivalent normal first moment

   $\sigma^N$     = the equivalent normal second moment

Equation (57) can be rearranged in terms of the inverse of $\Phi$, so that

$$\Phi^{-1}\left[F_{L_{C,large\ A}}(c)\right] = \frac{\log_e(c) - \mu^N}{\sigma^N}$$

(58)

Equation (58) defines a linear relation between $\log_e(c)$ and $\Phi^{-1}[F_{LC,large\ A}(c)]$. If it is found that this relation is perfectly linear then it is accepted that the distribution of $L_{C,large\ A}$ is not against the hypothesis $H_1$. The values of $\log_e(c)$ versus $\Phi^{-1}[F_{LC,large\ A}(c)]$ are plotted in Figure 53, and a linear trend line is fitted through them. The deviation from this trend line is measured in terms of the $R^2$ - value, which is equal to unity if the relation is perfectly linear. For ease of interpretation the values of c are in terms of kg/m$^2$ so that $\text{Log}_e(c)$ is larger than zero.

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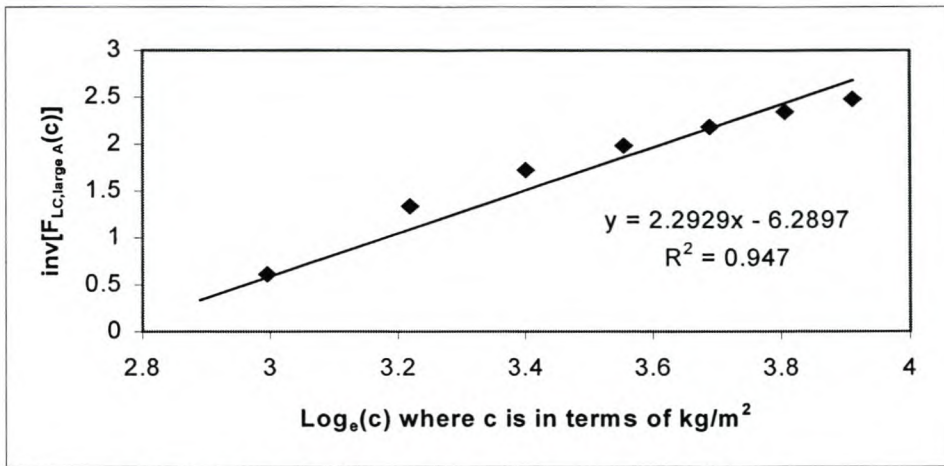
**Figure 53. Lognormal Probability Paper for $L_{C,large\ A}$**

### Interpretation and Conclusions

Since $R^2$ is closer to unity for the lognormal distribution than for the extreme type 1 it is stated that the distribution of $L_{C,large\ A}$ is not against the hypothesis $H_1$ and it is accepted that $L_{C,large\ A}$ can be modelled as a lognormal random variable. The reason for $L_{C,large\ A}$ being closer to a lognormal variable than an extreme type 1 variable lies in the fact that the second moment for AS ~ lognormal is almost 100% greater than the second moment for MW ~ extreme type 1, which forces $L_{C,large\ A}$ to behave more as a lognormal variable for the upper range of it's values.

If one considers the $R^2$ - values in Figures 52 & 53 it is observed that they do not differ significantly and that they are both relatively close to unity. Therefore, there would be no significant deviation in either modelling $L_{C,large\ A}$ as an extreme type 1 or a lognormal variable and $L_{C,large\ A}$ is therefore insensitive to being modelled as any one of these two distributions.

$\mu^N$ and $\sigma^N$ in Equation (58) can now be calculated from the equation of the trend line shown in Figure 53 by comparison with Equation (58). From the comparison it is obvious that $\sigma^N = 1/2.293 = 0.44$ kg/m$^2$ (0.0043 kN/m$^2$) and $\mu^N = 6.29\sigma^N = 2.74$ kg/m$^2$ (0.0274 kN/m$^2$). The first two moments for $L_{C,large\ A}$ ~ lognormal can now be calculated from these two parameters via the following relations:

$$\mu = \exp[\mu^N + 0.5(\sigma^N)^2] \tag{59}$$

$$\sigma = \sqrt{\exp\left(2\mu^N + 2\left(\sigma^N\right)^2\right) - \exp\left(2\mu^N + \left(\sigma^N\right)^2\right)} \tag{60}$$

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The resulting statistics for the probabilistic model of $L_{C,large A}$ are shown in Figure 54.
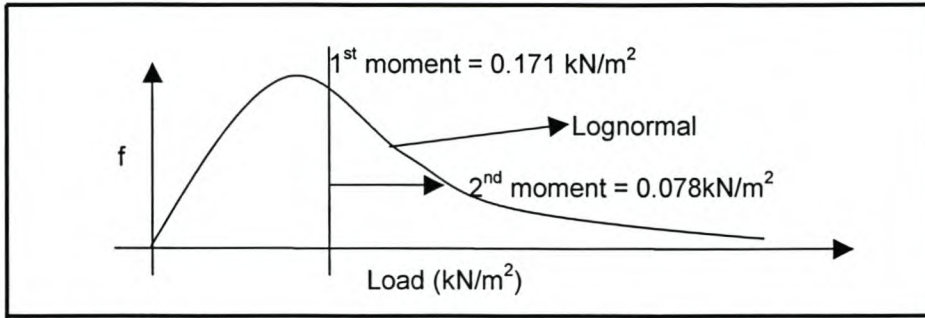


**Figure 54. Probabilistic Model for the Construction Load for Large Tributary Roof Areas**

When considering the model obtained for $L_{C,large A}$ one observes that the values of the first two moments seem odd in the sense that the first moment is smaller than the sum of the first moments of MW and AS, and the second moment seems relatively high resulting in a high coefficient of variation c.o.v. = 0.46. This is explained by recognising that over the range of c* - values as chosen in Table 41, the lognormal distribution function with these two moments best approximates the distribution of the cumulative probabilities of $L_{C, large A}$ = MW + AS. A comparison of the cumulative probability function for $L_{C, large A}$ obtained through the generalisation to lognormal and the original cumulative probabilities obtained in Table 41 is shown for the upper "half" of the variable in Table 48.

**Table 48. Comparison of Cumulative Probabilities for the Original and the Generalised Model**

| $F_{LC,large A}$ (%) | $L_{C, large A}$ - value for original (kN/m$^2$) | $L_{C, large A}$ - value for generalisation (kN/m$^2$) |
|---|---|---|
| 50 | 0.180 | 0.155 |
| 60 | 0.187 | 0.173 |
| 70 | 0.196 | 0.195 |
| 80 | 0.210 | 0.224 |
| 90 | 0.246 | 0.272 |
| 95 | 0.288 | 0.318 |
| 97 | 0.335 | 0.353 |
| 99 | 0.435 | 0.428 |

From Table 48 it can be seen that the values are close for the lower percentiles of 50 to 80%, whereafter the generalisation overestimates $L_{C, large A}$ for the percentiles from 80% to 95%, and finally the values converge again for the 99% cumulative probability. This is in accordance with Figure 52 where it is shown that the deviation from the

trend line is largest over the middle section. The values of the first two moments are inconsequential, as long as there exists close agreement between the cumulative probability percentiles obtained from the generalisation and the original for this range of upper realisations.

In establishing a probabilistic model to be solely used for reliability analyses and probabilistic design one could possibly find the first two moments in such a way that it closely approximates the region for $F_{LC,large\ A}(c)$ = 95 to 99% as it is in this region where the design point would fall. However, being more accurate in this region would mean that the model is less accurate in the region for $F_{LC,large\ A}(c)$ = 50 to 95%. Therefore, in prescribing a model for the construction load on large tributary areas to be used for general purposes, such as in combination with the extreme value of another combination in applying the adaptation of Turkstra's rule, the load model as found in Figure 54 prevails. Furthermore, it is noted that for the cumulative probabilities of 95 to 99% the generalised model deviates from the original by an average of 5% overestimation which is in any event rather insignificant.

**CHAPTER 7: EVALUATION OF THE PROVISIONS MADE BY THE SABS 0160 - 1989.**

This section is concerned with evaluating the provisions made by the SABS 0160-1989 (SABS) for the imposed roof load of inaccessible roofs in terms of the load mechanisms investigated in this study. Mainly, the SABS is evaluated in terms of the level of reliability catered for through its design provisions.

The SABS loading code does not specifically stipulate what the status is in terms of its nominal prescribed imposed load intensities, i.e. what amount of conservatism is built into these prescribed values. For the purpose of this section, it is assumed that the 95% maximum load models, as established in this investigation, are the rational equivalent of the nominal prescribed load intensity of the SABS 0160-1989, and are subsequently implemented in this manner.

## 7.1 Large Tributary Roof Areas

SABS 0160-1989 Clause 5.4.4.3 defines the imposed roof load for inaccessible roofs through the following equation:

$$L_n = 0.3 + \frac{15 - A_t}{60} \qquad \text{for } A_t < 15 \text{ m}^2 \qquad (61)$$

$$= 0.3 \qquad \text{for } A_t \geq 15 \text{m}^2$$

$$= 0.5 \qquad \text{for } A_t \leq 3 \text{m}^2$$

where $L_n$ = the nominal imposed roof load in kN/m$^2$

$A_t$ = the tributary area for the member under consideration in m$^2$.

Equation (61) defines a relation between $L_n$ and $A_t$ whereby $L_n$ decreases linearly as $A_t$ increases. In so doing, the SABS 0160-1989 (subsequently referred to as SABS) takes into account the fact that the probable number of workers or equipment per m$^2$ increases as the area decreases.

The tributary roof areas for frames of industrial buildings are invariably larger than 15 m$^2$ and therefore the 0.3 kN/m$^2$ as prescribed by the SABS 0160-1989 is relevant to this evaluation. For large areas, the performance of the SABS is measured in terms of its provision for the construction and maintenance loads.

## 7.1.1 The Construction Load

$L_{C,large\ A}$ as determined in Section 6.6 pertains to this section. The evaluation of SABS 0160-1989 is done on the basis of two criteria:

- What percentile is the 0.3 kN/m$^2$ characteristic load value as prescribed by the SABS? In other words, if one accepts that $L_{C,large\ A}$ is the probabilistic model describing the imposed construction roof load for large areas, what is the cumulative probability $F_{LC,large\ A}(0.3)$?

- What is the level of reliability obtained using the proposed nominal load value of the SABS and the limit state design philosophy.

### SABS 0160-1989 characteristic value

Considering the cumulative probability function of $L_{C,large\ A}$ = MW + AS as shown in Table 48 ($L_{C,large\ A}$ original), it is found that the 0.3 kN/m$^2$ as prescribed by the SABS is the 96% characteristic value. Code calibration is done on the basis that the nominal load values are the 95% characteristic values. Therefore the 0.3 kN/m$^2$ as prescribed by the SABS, being the 96% - value, is reasonable.

### Reliability obtained through Implementation of SABS 0160-1989 Design Criteria

The limit state function of the SABS 0160 for the relevant load cases is

$$\phi R_n \geq 1.2 D_n + 1.6 L_n \tag{62}$$

where $\phi$        = resistance factor for steel = 0.9 (SABS 0162)

     1.2     = partial load factor for the dead load D

     1.6     = partial load factor for the imposed load L

     $D_n$     = the nominal dead load value

     $L_n$     = the nominal imposed roof load prescribed by SABS 0160

The performance function relevant to this situation is

$$g = R - D - L_{C,\text{large A}} \qquad (63)$$

where $L_{C,\text{large A}} = MW + AS$

$\qquad$ MW $\qquad$ = the load due to workers on the roof (see Section 6.6.1)

$\qquad$ AS $\qquad$ = the load due to the stacking of roof cladding (see Section 6.6.1)

$g < 0$ represents the failure state and $g > 0$ the safe state. The probability of $g < 0$ can now be determined through first-order second moment formulation. $R_n$ is calculated from Equation (62) for $L_n = 0.3$ kN/m$^2$ and $D_n = 0.20$, 0.35 and 0.50 kN/m$^2$ which again constitutes the range of reasonable dead load values for light industrial buildings. The performance function is subsequently evaluated with the statistics pertaining to the load and resistance variables shown in Table 49.

**Table 49. Statistics for the Variables used to evaluate the Level of Reliability provided for by the SABS for Large Tributary Roof Areas**

| | D | MW | AS | R |
|---|---|---|---|---|
| Distribution type | Lognormal | Extreme type 1 | Lognormal | Lognormal |
| C.o.v. | 0.100* | 0.206 | 1.75 | 0.15* |
| 1$^{st}$ moment (kN/m$^2$) | 0.20, 0.35, 0.50 | 0.165 | 0.036 | From Eq. (62) |
| Average / nominal value | 1.05* | N/A | N/A | 1.05* |

*C.o.v's and average to nominal ratios obtained from NBS 577 (1980)

Note that the average value for R calculated from Equation (62) is done taking into account the ratios of average to nominal values for the variables as follows:

$$0.9\frac{R_{avg}}{1.05} = 1.2\frac{D_{avg}}{1.05} + 1.6 \times 0.3 \qquad (64)$$

The $D_{avg}/D_n$ - ratio of 1.05 is non-conservative and suggests that the designer under-estimates the dead load. The $R_{avg}/R_n$ - ratio of 1.05, in turn, suggests conservatism in this regard from the designer.

The first-order second moment reliability analyses are now performed for the three chosen values of $D_{avg}$. The reliability obtained for the three cases is presented in Figure 55, for the case where $L_{C,\text{large A}} = MW + AS$ and where $L_{C,\text{large A}}$ is equal to the generalised variable (see Section 6.6.5). The spreadsheet programme RELAN is again used for the analyses (refer to Appendix J).

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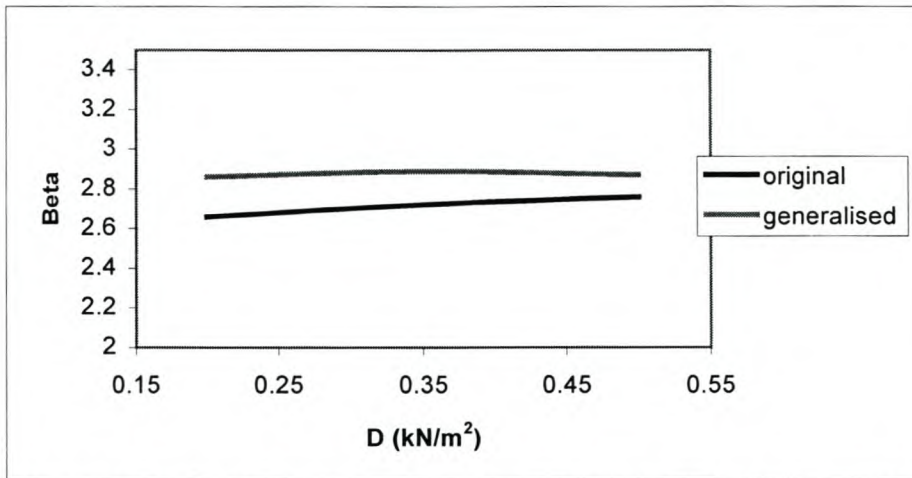
**Figure 55. Reliability obtained through SABS provisions for Large Tributary Roof Areas**

As is evident from Figure 55, the generalised variable representing the construction load for large areas with a single distribution function overestimates the reliability index $\beta$ obtained from the original model by 4 to 7%. Taking into account that the original model is interpreted to be rather conservative, this deviation is acceptable.

Considering the level of reliability obtained over the range of dead loads it is evident that the reliability is not significantly sensitive to the magnitude of the dead load, staying in the vicinity of $\beta = 2.7$. The minimum level of reliability that the SABS aims to achieve with it's proposed load and resistance factors and nominal load values is $\beta = 3$.

Although this study suggests that the required level of reliability is currently not achieved for the said conditions, it may be argued that the consequences of collapse during the construction period is not as severe as when the building is in occupational use and therefore a lesser level of reliability is acceptable. Also, $\beta = 3$ applies for buildings over their 50 year lifetimes. As the construction loads only occur over a relatively short period of time, it could be argued that a smaller target reliability applies. However, establishing such a target level of reliability for the construction period is a regulatory task for code authorities, and currently there exists no rationale for accepting $\beta = 2.7$. In other words, although it may be anticipated that a level of reliability of smaller than $\beta = 3$ is acceptable, the amount by which it may be smaller (if indeed it *may* be smaller) is not known and therefore $\beta = 2.7$ is not acceptable.

Recall that the model chosen to represent the load due to workmen on the roof is that for 4m - frame spacing, relating to the smaller tributary area and therefore the more conservative model (see Section 6.1). The model for the 5m - frame spacing is subsequently used in combination with the average load due to the over-stacking of roof cladding AS, and the reliability so obtained through the SABS is re-evaluated for this case. It is found that the SABS provisions cater for a level of reliability in the vicinity of $\beta$ = 2.8 over the range of dead load values (refer to Appendix J). This is not significantly larger than when the model for the 4m - frame spacing is used (where $\beta$ = 2.7). The reason for the reliability not being significantly influenced is that the second moment of AS is by far the dominating parameter in determining the failure probability (i.e. the dominating direction cosine is that of AS) and this suppresses the influence of changes in the load due to workmen. It can subsequently be concluded that using the model for the 4m - frame spacing does not produce overly conservative results.

A combination of many parameters may be varied in order to achieve the desired level of reliability. For instance the resistance factor, the partial load factors or the proposed nominal imposed roof load value may be varied. It is not advisable to adjust the resistance factor or the partial imposed load factor of 1.6 to 2 (which is required to achieve $\beta$ = 3) since this has an influence on a large amount of design cases and would result in overly conservative resistances. An increase of the SABS proposed roof load for large areas from 0.3 kN/m$^2$ to 0.4 kN/m$^2$ would ensure that $\beta$ = 3 is achieved and would be a more viable option since this would limit the conservatism to the specific case of imposed roof loads for inaccessible roofs. This, however, is not necessarily the best (or only) option and the subsequent treatment (if any) of the non-conservatism of the SABS with regard to imposed roof loads needs to be assessed by code authorities.

It is also of interest to note that although the 0.3 kN/m$^2$ of the SABS 0160-1989 is the 95% characteristic value of the construction load model, this does not warrant that the desired level of reliability of $\beta$ = 3 is achieved.

### 7.1.2 The Maintenance Load

Referring to Figure 48 in Section 6.4, the maintenance load for large tributary roof areas includes only the load imposed by the maintenance workers on the roof. It is

important to realise that the maintenance load, as determined through this investigation, excludes the process of reinstallation of the roof sheeting, which tends more towards the model established for construction loads.

It can be anticipated from the magnitude of the first two moments of this variable that in terms of reliability this load does not play a significant role. The SABS prescribed nominal load value of 0.3 kN/m$^2$ is the 99.9% characteristic value of the load model for the maintenance workers on the frames. The level of reliability catered for through the provisions made by the SABS is $\beta = 5$, which equates to a failure probability of 3.1E-05 % (refer to Appendix J).

The numbers obtained in the above are purely academic and it can be safely concluded that the SABS conservatively provides for the roof load due to maintenance workers on large areas.

## 7.2 Small Tributary Roof Areas

Small tributary roof areas pertain to the tributary areas for the purlins of light industrial steel buildings. The tributary area for the purlins which was used to calculate the load model from is equal to 4 x 1.4m, which is the smaller and more conservative area of the two considered. Substituting this area into Equation (61) one obtains the prescribed SABS load value of 0.46 kN/m$^2$ for a tributary area $A_t = 5.6$ m$^2$.

For small tributary areas, the performance of the SABS 0160-1989 is again measured in terms of its provision for the construction load and the maintenance load.

### 7.2.1 The Construction Load

The evaluation of the SABS is performed in terms of the characteristic value and the reliability obtained through the provisions made for design.

*SABS 0160-1989 characteristic value*

The 0.46 kN/m$^2$ - value of the SABS is the 8% characteristic value of the load model for the construction load for small tributary roof areas, i.e. even far below the expected

value. This is totally off par with the 95% - value on which code calibration is based. The 95% characteristic value of the load model determined through this study is 0.76 kN/m$^2$, which is factor 1.7 larger than the SABS value! This suggests that the SABS 0160-1989 is highly (and disconcertingly) non-conservative in providing for construction loads on small tributary areas.

Attention is drawn to the fact that the above load model is based on a purlin with a span of 4m and spacing of 1.4m which represents the conservative configuration found in practice (see Section 6.3). The load model for the more common situation of a span of 5m and a spacing of 1.7m is now used to re-evaluate the SABS prescribed value (refer to Figure 43 for the statistics for this model). For a tributary area $A_t$ = 1.7 x 5m = 8.5m$^2$, the SABS prescribed load value is 0.41 kN/m$^2$, which is the 63% characteristic value. Although this is closer to the 95% value of 0.55 kN/m$^2$ for this more common situation, it still suggests that the SABS 0160-1989 is substantially non-conservative. A more thorough conclusion will be made after the reliability obtained through SABS design provisions is made known.

### Reliability obtained through Implementation of SABS 0160-1989 Design Criteria

The reliability is assessed in the same way as was done in Section 7.1 through first-order second moment formulation. Since small tributary areas pertain to those for the purlins, the range of dead loads applicable to purlins is to be considered. Since the purlin's own weight contribution is small in relation to the weight of the cladding, the dead load is substantially dependent on the type of cladding, i.e. steel sheets or fibre-cement. For steel sheets the range of dead load values is taken as 0.10 to 0.12 kN/m$^2$ and for fibre-cement it is taken as 0.15 to 0.18 kN/m$^2$, including self-weight. The statistics pertaining to the load and resistance variables are summarised in Table 50.

**Table 50. Statistics for the Variables used to evaluate the Level of Reliability catered for by the SABS for Small Tributary Roof Areas**

| | D | Load due to construction workers | R |
|---|---|---|---|
| Distribution type | Lognormal | Extreme type 1 | Lognormal |
| C.o.v. | 0.10* | 0.17 | 0.15* |
| 1$^{st}$ moment (kN/m$^2$) | 0.1, 0.12, 0.15, 0.18 | 0.577 | From Eq. (62) |
| Average / nominal value | 1.05* | N/A | 1.05* |

*C.o.v's and average to nominal ratios obtained from NBS 577 (1980)

The first order second moment reliability analyses are now performed for the four chosen values of $D_{avg}$ (see Appendix J). The reliability obtained is presented in Figure 56.
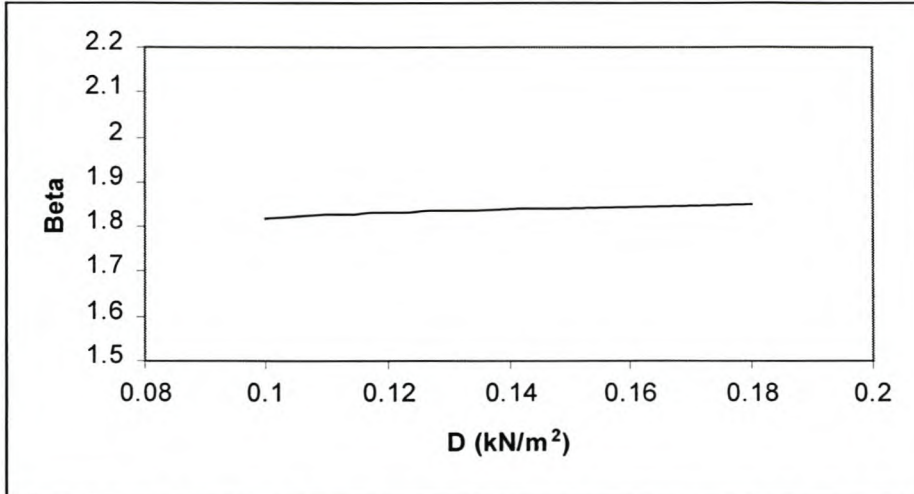


**Figure 56. Reliability obtained through SABS provisions for Small Tributary Roof Areas**

The level of reliability obtained is $\beta \approx 1.84$, which equates to a probability of failure of 3.3%. This is substantially lower than the required minimum of $\beta = 3$. For the more common situation (purlin span = 5m, spacing = 1.7m) it is found that $\beta = 2.6$ with a probability of failure of 0.4%, which is still well below the target level of $\beta = 3$.

It could be argued that the consequences of failure of a purlin during construction very seldom involve total collapse of the purlin and loss of life and that the cost of correcting the failure by replacing the purlin, or otherwise, when the contractor is in any event established on site is negligible. From this point of view a lesser level of reliability may be acceptable. However, a reliability level of $\beta = 1.84$ is unacceptably low under most circumstances and there would have to be very strong arguments to justify such a low level of reliability.

For a tributary area $A_t = 5.6m^2$, the SABS prescribed imposed roof load value would have to be increased to 0.65 kN/m$^2$ to meet the target level of reliability of $\beta = 3$.

### 7.2.2 The Maintenance Load

Using the model for the maximum maintenance load as determined from the expert survey (see Section 6.5) one finds that the 0.46 kN/m$^2$ prescribed load value of the SABS is the 97% characteristic value, which is on par.

The level of reliability obtained through SABS design criteria over the range of dead load values considered in the previous section is $\beta = 3.5$ ($\pm$ 2% for the two extremes of the dead load). It can therefore be concluded that the SABS conservatively provides the maintenance load for small tributary roof areas.

### 7.3 Conclusions

Three main conclusions can be made from this section:

- The SABS 0160-1989 conservatively provides for imposed roof loads due to maintenance activities on the roof (subject to the exclusion of replacement of roof sheeting).

- The SABS 0160-1989 is non-conservative in its provision for imposed roof loads due to construction activities for members with large tributary areas.

- The SABS 0160-1989 is highly non-conservative in its provision for imposed roof loads due to construction activities for members with small tributary areas.

Certain "mitigating circumstances" apply to the reliability during construction. This is due to the consequences of failure in terms of monetary cost and loss of life being relatively small during the construction period, especially for purlins, where small tributary roof areas apply. Since bending is the primary limit state considered, failure would typically be in a non-brittle, non-catastrophic manner, and, for the purlins especially, redundancies are present in the form of roof cladding acting as tension connectors between purlins and frames which would prevent the purlin from actually collapsing and therefore minimising the probability of loss of life. Although the argument in the above suggests that a reliability level of lower than $\beta = 3$ may be acceptable, it certainly does not justify $\beta = 2.7$ and especially $\beta = 1.8$. Determining an "acceptable" target reliability level is a regulatory task for code authorities. It involves

structural safety and economic analysis for a large number of structural examples. In any event, it is highly unlikely that a reliability level of $\beta = 1.8$ would ever be acceptable. Table 51 presents target reliabilities for ultimate limit states according to the Probabilistic Model Code (JCSS 2000).

**Table 51. Different Target Reliability Levels as proposed by JCSS (2000) for a 50-year Lifetime of the Structure**

| Relative cost of Safety measure | Consequences of failure | | |
|---|---|---|---|
| | Minor | Moderate | Large |
| Large | $\beta = 1.7$ | $\beta = 2.0$ | $\beta = 2.6$ |
| Normal | $\beta = 2.6$ | $\beta = 3.2$ | $\beta = 3.5$ |
| Small | $\beta = 3.2$ | $\beta = 3.5$ | $\beta = 3.8$ |

If it is assumed that the consequences of failure during construction are *minor* (for the reasons stated in the above) and that the relative cost of safety measure is *normal*, one obtains a target $\beta$ of 2.6 (from Table 51). To make the building more safe would involve amongst others an increase of the z - modulus of the sections for the purlins and frames since the bending limit state is the primary consideration. This is what primarily determined the original cost of the building and such an increase in steel would certainly have a relatively large effect on the cost of the building. If it were only the connections that needed to be strengthened (i.e. thicker plates, more bolts etc.), this would have an insignificant effect on the cost of the building. Therefore the assumption that the relative cost of safety measure is normal (as opposed to small) is justifiable.

An important aspect to take note of when comparing the $\beta$ - values found in this study to those of Table 51, is that the values in Table 51 are "average" levels of reliability while those obtained in this study are minimum levels of reliability. "Average" levels of reliability are interpreted by recognising that over the spectrum of design cases for steel, concrete etc. these $\beta$ - values are the average of what should be catered for through generalised codified provisions. This means that, due to the generalisation of design codes, certain cases have marginally lower levels of reliability. This study on the other hand, calculates the level of reliability for worst case scenarios, i.e. $\beta = 2.7$ & 1.8 can be considered to be *minimum* levels of reliability. This is justified through the many conservative assumptions that have been made during the course of this study in establishing the load models. Analogue to this study it can confidently be stated that for the majority of light industrial steel buildings that are designed to SABS specifications, under any conditions (building geometry, dead load, etc.), a level of

reliability of more than $\beta = 2.7$ & 1.8 applies for the frames and purlins respectively during the construction period.

The target $\beta$ of 2.6 obtained from Table 51 justifies to a certain extent the reliability level of $\beta = 2.7$ for large areas during construction, but the $\beta$ of 1.8 for small areas is still highly non-conservative. It is however stressed that $\beta = 2.6$, as obtained from Table 51, results from the *author's* interpretation of Table 51 (and assuming that Table 51 is legitimate).

For the SABS to meet the required minimum level of reliability of $\beta = 3$ over the range of tributary areas the following is proposed for maintenance and construction loads:

- For the maintenance load it is found that the SABS 0160-1989 meets the required minimum $\beta = 3$ over the range of tributary areas. If the SABS value is to be adjusted, it should be reduced so as to prevent uneconomical designs in providing for maintenance loads. However, such reduction is not recommended since this investigation is specific to light industrial steel buildings (of portal frame construction) and there are other applications of inaccessible roofs where such reduced values may lead to unsafe designs. Also, the load mechanism of hail and snow is not accounted for in the investigation (see Section 8.3) and therefore reduction of current SABS values may result in inadequate provision for hail and snow loads.

- For the construction load, an increase in the prescribed load value of the SABS 0160-1989 is necessary to meet the required minimum $\beta = 3$. The prescribed uniform loads required to meet the target $\beta = 3$ exactly are shown in Table 52, for three different tributary areas. The current SABS prescribed values for the three areas are also shown.

**Table 52. Imposed Roof Load Values Required to Meet $\beta = 3$**

| Tributary area (m²) | Value required to meet $\beta = 3$ (kN/m²) | SABS 0160-1989 prescribed load value (kN/m²) |
|---|---|---|
| 5.6 | 0.65 | 0.46 |
| 8.5 | 0.46 | 0.41 |
| Large areas (>50) | 0.40 | 0.30 |

For the SABS to maintain the minimum $\beta$ of 3 for construction loads over the range of tributary areas, Equation (61) is to be altered to the following:

$$L_n = 0.4 + \frac{15 - A_t}{30} \qquad \text{for } A_t < 15 \text{ m}^2 \qquad (65)$$

$$= 0.4 \qquad \text{for } A_t \geq 15 \text{m}^2$$

$$= 0.8 \qquad \text{for } A_t \leq 3\text{m}^2$$

where  $L_n$ = the nominal imposed roof load in kN/m$^2$

$A_t$ = the tributary area for the member under consideration in m$^2$.

Equation (65) results in $L_n = 0.71$ kN/m$^2$ where $A_t = 5.6$ m$^2$, and $L_n = 0.62$ kN/m$^2$ where $A_t = 8.5$ m$^2$.  Although these values are higher than the required minimum values of 0.65 and 0.46 kN/m$^2$ (see Table 52), Equation (65) is so constructed that it is compatible with the current SABS format where the values remain constant for $A_t > 15$m$^2$ and $A_t < 3$m$^2$.  Also, Equation (65) results in convenient or "user friendly" values at these junctures, whilst at the same time rendering itself to simple and easy application.  This extra conservatism is therefore built into Equation (65) to allow for convenient codified application.

Equation (65) can now be used to represent the adjusted SABS prescribed load value.  Comparison with the prescribed loads of other loading codes (as was done in Section 1.2, Figure 1) and with the current SABS 0160-1989 load value is shown in Figure 57.  The values required to meet $\beta = 3$ at the three tributary areas (as shown in Table 52) are also shown in Figure 57.

Comparison of Figures 57 & 1, shows that the SABS prescribed load is adjusted in the direction of the other loading codes' values.  However, the adjusted SABS value is still substantially lower than particularly those of the ASCE code and the Eurocode.  Only for very small tributary areas (< 5m$^2$) does the adjusted SABS value become on par with Eurocode and relatively close to the ASCE code value (but still lower than the ASCE code value).
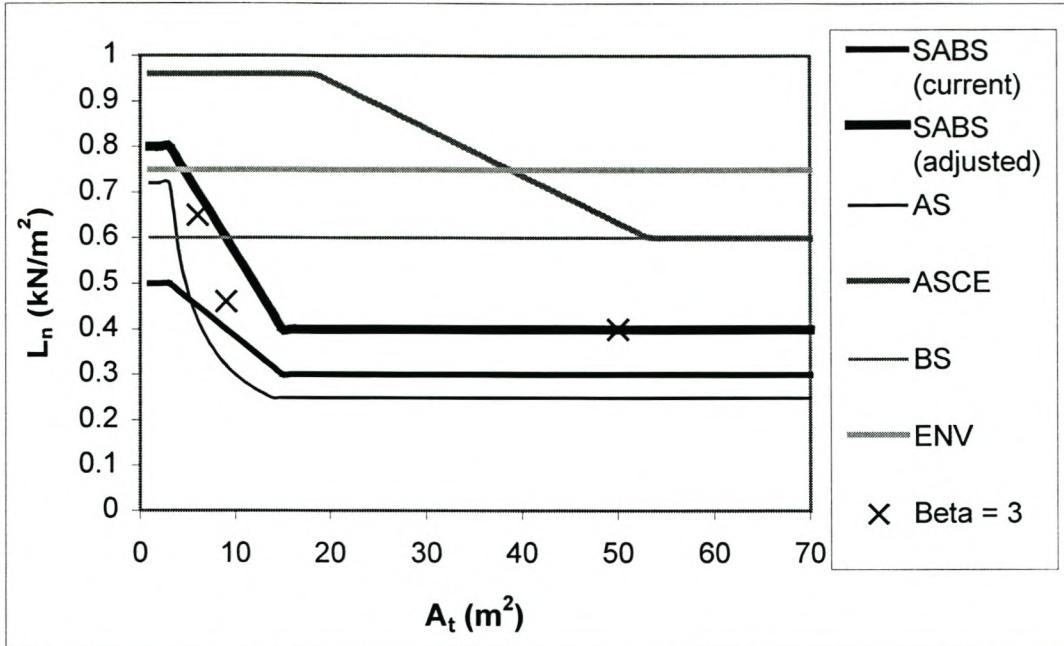
**Figure 57. Imposed Roof Loads dependent on Tributary Area, with Adjusted SABS Load**

As an alternative to increasing the prescribed uniformly distributed load of the SABS, larger specified concentrated loads may be introduced to provide for small areas. The advantages and disadvantages of this approach are to be assessed in a separate investigation.

## CHAPTER 8: COMPARISON OF LOAD MODELS AND CRITICAL ASSESMENT OF LOAD MECHANISMS

### 8.1 Comparison of Load Models with Other Codified Provisions

This section compares the load models founded in this study to those in the EUROPEAN PRESTANDARD PrEN 1991-1-6 Part 1.6: General actions - Actions during execution, and the Probabilistic Model Code (JCSS 2000).

### 8.1.1 European Prestandard PrEN 1991-1-6 Part 1.6

This part of the European Prestandard applies to the investigation due to the fact that it addresses construction loads in a systematic way and incorporates quantitative provisions for design. The construction loads addressed include:

1. Working personnel, staff and visitors, with small site equipment.

2. Storage of movable items (e.g. building and construction materials, precast elements, and equipment).

3. Non permanent equipment in position for use during execution (e.g. formwork panels, scaffolding, falsework, travelling forms, launching girders and nose).

4. Moveable heavy equipment, usually wheeled or tracked, (e.g. cranes, lifts, vehicles, lifttruck, power installation, jacks, heavy lifting devices).

5. Accumulation of waste materials (e.g. surplus construction materials, demolition materials).

6. Parts of the structure under execution before the final design actions take effect (e.g. additional loads due to concrete being fresh, loads and reverse load effects due to particular process of construction such as assemblage, loads from lifting operation).

Construction loads relevant to this investigation are Numbers 1, 2 and 3 in the above.

*Working personnel, staff and visitors, with small site equipment*

The recommended value of the code is 1.0 kN/m$^2$. This value is the 99.99 % characteristic value of the load model for workers on large areas and the 99.98 % characteristic value of the load model for workers on small areas. It is concluded that the 1.0 kN/m$^2$ does not in actuality apply to inaccessible roofs, and in particular is more relevant to easily accessible construction areas where concrete works occur. Therefore, this comparison is not warranted.

*Storage of moveable items*

The code proposes a uniformly distributed load of 0.2 kN/m$^2$ for bridges. This value is the 98 % characteristic value of the load model for cladding stacked on the rafters. Again, this is considered to be not applicable to the investigation.

*Non permanent equipment in position for use during execution*

These loads are to be defined for the particular project using information given by the supplier. This criterium is in accordance with what was assessed from the philosophy-of-design questions put forward to engineers during the expert survey, where the opinion is that these effects should be designed for each specific case and that a generalised codified provision would result in overly conservative designs for most cases (see Appendix D1). These loads generally occur locally on a building (equipment suspended from the roof) and to design the whole of the building for such localised loads would be uneconomical.

### 8.1.2 JCSS Probabilistic Model Code (JCSS 2000)

The imposed load models provided in this code all pertain to imposed *floor* loads for various occupancy types. There are no probabilistic load models for imposed loads of inaccessible roofs.

### 8.1.3 Conclusions

It is concluded that, due to the lack of information, no formal investigation yielding quantitative results has previously been conducted on imposed loads for inaccessible roofs.

## 8.2 Commentary on Construction Loads

A contentious issue that needs to be addressed at this stage is the engineer's responsibility in terms of designing for the construction phase of the building. In this study, the imposed loads produced during construction activities on the roof of the structure are regarded as loads for which the structure should be designed in its final (finished) form. This is in accordance with the SABS 0160-1989 where the prescribed imposed roof load does provide for the effects of workmen and stacked materials during construction. However, this issue is not resolved since there are strong arguments supporting the philosophy that it is the contractor's responsibility to erect the building and to "make it stand" in it's final form, and it is the engineer's responsibility to ensure that the building can withstand the loads it is subjected to during it's lifetime, i.e. during the time it serves it's purpose.

It may be economically unsuitable and unpractical for building contractors to provide additional support during construction, and if it is the case that the imposed roof load produced during the lifetime of the building is large enough that it may just as well account for the loads during construction, this would warrant construction loads being provided for through the prescribed imposed roof load. However, this would require that the code specifically stipulates for which construction loads it provides for. There are many types of construction loads and many degrees of completion of the structure that influence the resistance of the building. All of this complicates the matter of which construction loads are suitable for codified provision and which are not.

Basically, there are three viewpoints from which the proper treatment of construction loads should be considered, namely philosophical, practical and economical. These three considerations should be used as the basis for performing a cost - benefit analysis for this issue, and ultimately coming to a rational conclusion.

It is the opinion of the author that codified provision should be made for construction loads "within reason". The concept of "within reason" will always be a subjective one. The author's perception of "within reason" is by means of answering the following question: "What is necessary to install the roof cladding after the frames have been erected?" Workmen on the frames are certainly necessary, stacking of roof cladding on the frames is necessary from a practical standpoint, stacking of cladding on the purlins is not necessary and so forth. By the nature of the loads imposed during construction, these loads are to an extent subject to human control. This, in a sense, negates the attribute of inherent randomness that has been associated with the magnitudes of the construction load variables in this study. The study is founded on the principle that human behaviour in so far the loads imposed during construction is totally random. Also, by not incorporating the situation where cladding is stacked on the purlins as part of the reliability analyses for purlins, in effect it is assumed that random human behaviour is allowed within certain constraints – these constraints being what is perceived to be "within reason". Random behaviour within constraints is a contradictory concept. However, it is a concept that performs adequately in terms of codified provision which is not overly conservative. So, within this constraint, totally random behaviour is assumed for the purpose of reliability analyses.

## 8.3 Hail and Snow Loads

As stated in Section 3.5.5, the measurement and subsequent quantification of the load due to hail and snow accumulating on the roof is a meteorological exercise to be conducted separately and *independently* from the expert survey. The load mechanisms quantified through this investigation can be assumed to not take place simultaneously with hail or snow precipitation on the roof. The reason being that both the load mechanisms occur only for very short time intervals and therefore the probability of the time intervals overlapping is very small. Thus, a combination of say, the maximum number of maintenance workers on the roof with the load due to snow or hail, would be overly conservative.

It is of interest to note that upon enquiry from the weather bureau for records of hail and snow precipitation, it was discovered that no such information exists. The only available data is that of time and place of hail or snow precipitation, and some isolated documented cases where damage was caused to cladding due to hail impact forces.

## CHAPTER 9: SUMMARY AND CONCLUSIONS

A critical evaluation of provisions for imposed loads in the South African Loading Code for design of structures, SABS 0160-1989, by comparison with other codes was performed earlier by RETIEF, DUNAISKI and DE VILLIERS (2001). A representative set of four loading codes was selected for use as a basis in the evaluation of the SABS loading code. They are the BS 6399 Part 1-1996 (BS Code), the ENV 1991-2-1:1995 Part 2-1 (Eurocode), the AS 1170.1-1989 Part 1 (AS Code) and the ASCE 7-95 (ASCE Code), with particular emphasis on the Eurocode and the ASCE code as the two main contenders for an international reference code.

The evaluation revealed the SABS loading code to be generally non-conservative in its provisions for imposed loads for a range of general and specialist occupancy classes. The SABS provision for imposed loads for inaccessible roofs was found to be substantially non-conservative in comparison with the other codes. A comprehensive literature investigation yielded no information on imposed roof loads or any load survey data on the subject. An investigation into the imposed load for inaccessible roofs was therefore required in order to establish a scientific rationale through which the codified design values may be measured effectively. Due to the lack of information and the large uncertainties involved in the imposed roof load, stochastic treatment of the loads was implemented.

Therefore, the aim of the investigation was to establish the probabilistic load models which describe the imposed loads on inaccessible roofs. These models were then to be compared to existing load models and used to ascertain the level of safety catered for by SABS design provisions.

Ideally, one would have wanted to determine a general model covering the whole spectrum of imposed loads on inaccessible roofs and applicable to all types of buildings. However, such an approach is bound to result in inaccurate approximations of reality and for certain load cases and building types, gross deviations are to be expected.

Rather, the approach applied was to select a type of building that can be regarded as a generic example of buildings to which these loads apply, and to discretize the load into the various sub-mechanisms that translate into the imposed roof load. The

representative building selected for the investigation is a low-rise light industrial steel building, as shown in Figure 2. The load mechanisms identified are:

1. Workers on the roof during construction

2. Workers on the roof during repair, cleaning and maintenance

3. The stacking of roof cladding during construction

4. Machinery and equipment used during construction, repair and maintenance

5. Machinery supported by the roof during the installation of services or for any other purpose during the lifetime of the building

6. Hail and snow accumulating on the roof

Mechanisms 1, 2, and 3 where then quantified, either through physical load surveys, or through conducting an expert survey for those variables which are not observable. The use of expert opinion as a resource for information is not readily accessible in terms of yielding scientifically defendable results. However, owing to the nature of the load mechanisms translating into the imposed roof load, there was no other alternative but to draw on the knowledge of experts. Through consultation with experts, Mechanism 4 was found to be inconsequential in magnitude to warrant further investigation. Mechanism 5 was found to be non-quantifiable and that the magnitude for such loads are to be determined for each specific case. Mechanism 6 could not be modelled due to lack of information on hail and snow precipitation in South Africa.

The probabilistic models for Mechanisms 1,2 and 3 where then translated into load effects by taking into account the physical process resulting in the load effects. By applying these mechanisms in such a way as to maximise the said load effects, equivalent uniformly distributed loads (EUDL's) were calculated for each mechanism. The probabilistic models obtained in terms of the EUDL's pose an easily accessible format through which existing load models and codified provisions could be evaluated.

Depending on the load carrying member considered, the size of the tributary area for the member and whether the load model represents the average or maximum value for the variable, a substantial amount of permutations of the load models for the mechanisms resulted. They are shown in Figures 43 & 44. For convenience of application, the number of load models was reduced through conservative assumptions and consolidation of certain models into one. The resulting generalised probabilistic load models are shown in Table 53.

**Table 53. Summary of Load Models established through the Investigation**

| Load model | Distribution type | Average value (kN/m²) | Standard deviation (kN/m²) |
|---|---|---|---|
| Construction load on large areas (frames) | Lognormal | 0.17 | 0.08 |
| Construction load on small areas (purlins) | Extreme type 1 | 0.58 | 0.10 |
| Maintenance load on large areas (frames) | Extreme type 1 | 0.08 | 0.01 |
| Maintenance load on small areas (purlins) | Extreme type 1 | 0.38 | 0.03 |

These load models were then utilised to evaluate the SABS provisions in terms of how the 95% characteristic values of these models compare to the prescribed load intensity by the SABS 0160-1989, and what level of reliability is catered for by SABS ultimate limit-state design criteria. Code calibration is based on attaining a minimum target level of reliability of $\beta = 3$. The results are shown in Table 54.

**Table 54. Evaluation of SABS Provisions for Imposed Roof Loads**

| Load model | 95% characteristic value (kN/m²) | SABS 0160-1989 nominal proposed load value (kN/m²) | Minimum level of reliability catered for by SABS design criteria |
|---|---|---|---|
| Construction load on large areas (frames) | 0.29 | 0.3 | $\beta = 2.7$ |
| Construction load on small areas (purlins) | 0.76 | 0.46 | $\beta = 1.8$ |
| Maintenance load on large areas (frames) | 0.11 | 0.3 | $\beta = 5.0$ |
| Maintenance load on small areas (purlins) | 0.45 | 0.46 | $\beta = 3.5$ |

It is concluded that the SABS conservatively provides for maintenance loads on the roof, while the reliability for construction loads is non-conservative for large tributary areas and highly non-conservative for small areas. Certain mitigating circumstances have been identified for failure during construction, which to a certain extent justifies a level of reliability of $\beta$ smaller than 3 for construction loads. However, $\beta = 1.8$, for construction loads on small tributary roof areas, can under no circumstances be acceptable.

Whilst adjusting the prescribed load value of the SABS so as to meet the target $\beta = 3$ over the range of tributary areas does render the SABS value to a more favourable comparison with that of other loading codes, the adjusted SABS value is still

significantly non-conservative in relation to the other codes, in particular the Eurocode and the ASCE code.

As stated earlier, an investigation into the literature revealed that there are no imposed load models for inaccessible roofs available. The load models established in this investigation provides the basis for reviewing and adjusting provisions in the SABS 0160-1989. Incorporation of the load models into the JCSS Code will also solve a deficiency in this international code for probabilistic structural design.

## CHAPTER 10: REFERENCES

[1]     ANG AHS and TANG WH (1984).  *Probability Concepts in Engineering Planning and Design, Vol. II: Decision, Risk, and Reliability.*  John Wiley & Sons, New York.

[2]     ANSI/ASCE 7-95: AMERICAN SOCIETY OF CIVIL ENGINEERS *Minimum Design Loads for Buildings and Other Structures.*

[3]     AS 1170.1-1989: AUSTRALIAN STANDARD, Part 1: *Dead and live loads and load combinations.*

[4]     BS 6399 Part 1-1996: BRITISH STANDARD *Loading for buildings Part 1. Code of practice for dead and imposed loads.*

[5]     BS 6399 Part 3-1988: BRITISH STANDARD *Loading for buildings Part 3. Code of practice for imposed roof loads.*

[6]     COOKE RM (1991).  *Experts in Uncertainty: Opinion and Subjective Probability in Science.*  Oxford University Press, New York.

[7]     DE VILLIERS PJ, RETIEF JV and DUNAISKI PE (2000).  *Imposed Loading: Comparison of imposed loading as prescribed by various codes.*  Report to SAICE Working Group on SA Loading Code.  Document F5.2.  January 2000.

[8]     ELLINGWOOD B, GALAMBOS TV, MACGREGOR JC and CORNELL CA (1980).  *Development of a Probability Based Load Criteria for American National Standard A58,* NBS Special Publication No. 577, National Bureau of Standards, US Department of Commerce, Washington, DC.

[9]     ENV 1991-2-1-1995: RATIFIED EUROPEAN TEXT, *Part 2-1: Actions on structures – Densities, Self-weight and Imposed Loads.*

[10]    GUTTMAN I and WILKS SS (1965).  *Engineering Statistics.*  John Wiley & Sons, New York.

[11] INTERNATIONAL ORGANIZATION FOR STANDARDISATION (1998). *General principles on reliability of structures.* International Standard ISO 2394-1998.

[12] JCSS (2000) *Joint Committee on Structural Safety Probabilistic Model Code. Part 2 Load Models.* 11th Draft.

[13] PREN 1991-1-6: EUROPEAN PRESTANDARD, *Part 1.6: General Actions – Actions during execution*

[14] RETIEF JV, DUNAISKI PE and DE VILLIERS PJ (2000). *Evaluation of SABS 0160-1989 Minimum Imposed Loads: Comparison with other codes.* Report to SAICE Working Group on SA Loading Code. Document H 5.2. October 2000.

[15] SABS 0160-1989: SOUTH AFRICAN STANDARD *Code of Practice. The general procedures and loadings to be adopted in the design of buildings.* South African Bureau of Standards, Pretoria.

[16] SABS 0162-1-1993: SOUTH AFRICAN STANDARD *Code of Practice. The structural use of steel. Part 1: Limit-states design of hot-rolled steelwork,* South African Bureau of Standards, Pretoria.

[17] SOUTH AFRICAN NATIONAL CONFERENCE ON LOADING (1998). *Towards the development of a unified approach to design loading on civil and industrial structures for South Africa.*

[18] STEEL DESIGNERS MANUAL (1960). *2nd Edition, Gray CS, Kent LE, Mitchell WA, Godfrey GB.* Prepared for the British Steel Producers' Conference in conjunction with the British Iron and Steel Federation.

[19] TER HAAR TR and RETIEF JV (1996*). Questionnaire on Serviceability Criteria for Steel Structures.* University of Stellenbosch. Stellenbosch.

[20] TER HAAR TR and RETIEF JV (1997). *Utilising the Classical Method for Calibrating Experts in Steel Structures.* University of Stellenbosch. Stellenbosch.

[21]  VAN DEVENTER PJU (2000). *Advanced Engineering Statistics.* Unpublished Course Notes MT02. University of Stellenbosch. Stellenbosch.

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**APPENDIX A: SENSTUDY**

The spreadsheet programme SENSTUDY calculates the bending moments at the eaves of the columns and the roof ridge for different values of the circumstance parameters as explained in Section 2.1. The type of building used is a portal frame structure as shown in Figure 2.

Refer to the attached diskette for SENSTUDY. A printout of SENSTUDY is presented on the following page.

# SENSTUDY

| | | | |
|---|---|---|---|
| Roof Slope = | 0.18 | Roof Angle (rad) = | 0.174533 |
| k = $l_2 / l_1$ = | 1 | Dn (kN/m²) = | 0.35 |
| | | Ln (kN/m²) = | 0.3 |

| | |
|---|---|
| L1 (grav) = | 0.9 |
| $q_z$ (wind) = | 0.848 |

Note: Values for M/L² are in terms of kN/m per meter spacing of the frames

| | | | | | | | L1 = 1.2 Dn + 1.6 Ln | | | | L2 = 0.9 Dn + 1.3 Wn | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | vertical on roof | | | | horisontal on walls | | | hor | vertical on roof | | | | | | | | | |
| h/L | C1 wall | C2 wall | C1 roof | C2 roof | 1+d/(2v) | N | He/WL | M/L² at Col Eaves | x/L | M/L² at Roof Ridge | He/Wh | M/L² at Windward Eaves | M/L² at Leeward Eaves | M/L² | He/WL | M/L² | M/L² at Leeward Eaves | M/L² at Windward Eaves | Va/L due to uplift | Va/L due to sway on walls | Va/L due to sway on roof | Sum Va/L | Ha/L | M/L² at Roof Ridge |
| 0.090 | 0.700 | 0.225 | 1.200 | 0.400 | 1.980 | 14.151 | 0.633 | -0.051 | 0.444 | 0.015 | 0.166 | 0.006 | 0.003 | 0.003 | 0.633 | 0.032 | 0.034 | 0.031 | 0.394 | 0.004 | -0.010 | 0.387 | 0.382 | 0.004 |
| 0.095 | 0.700 | 0.225 | 1.200 | 0.400 | 1.928 | 13.665 | 0.609 | -0.052 | 0.446 | 0.016 | 0.169 | 0.006 | 0.003 | 0.004 | 0.609 | 0.033 | 0.035 | 0.032 | 0.394 | 0.005 | -0.011 | 0.387 | 0.371 | 0.005 |
| 0.100 | 0.700 | 0.225 | 1.200 | 0.400 | 1.882 | 13.238 | 0.586 | -0.053 | 0.448 | 0.017 | 0.173 | 0.007 | 0.003 | 0.004 | 0.586 | 0.033 | 0.035 | 0.032 | 0.394 | 0.005 | -0.011 | 0.388 | 0.361 | 0.005 |
| 0.125 | 0.700 | 0.225 | 1.200 | 0.400 | 1.705 | 11.719 | 0.492 | -0.055 | 0.457 | 0.021 | 0.186 | 0.011 | 0.005 | 0.005 | 0.492 | 0.035 | 0.036 | 0.034 | 0.394 | 0.008 | -0.013 | 0.389 | 0.324 | 0.009 |
| 0.150 | 0.700 | 0.225 | 1.200 | 0.400 | 1.588 | 10.808 | 0.422 | -0.057 | 0.463 | 0.024 | 0.197 | 0.015 | 0.008 | 0.006 | 0.422 | 0.036 | 0.037 | 0.036 | 0.394 | 0.011 | -0.015 | 0.390 | 0.300 | 0.011 |
| 0.175 | 0.700 | 0.225 | 1.200 | 0.400 | 1.504 | 10.220 | 0.368 | -0.058 | 0.468 | 0.027 | 0.205 | 0.020 | 0.011 | 0.007 | 0.368 | 0.036 | 0.036 | 0.038 | 0.394 | 0.016 | -0.017 | 0.392 | 0.286 | 0.014 |
| 0.200 | 0.700 | 0.225 | 1.200 | 0.400 | 1.441 | 9.821 | 0.325 | -0.058 | 0.471 | 0.029 | 0.211 | 0.026 | 0.014 | 0.008 | 0.325 | 0.037 | 0.035 | 0.040 | 0.394 | 0.020 | -0.019 | 0.395 | 0.277 | 0.016 |
| 0.225 | 0.700 | 0.225 | 1.200 | 0.400 | 1.392 | 9.544 | 0.290 | -0.059 | 0.474 | 0.032 | 0.217 | 0.033 | 0.018 | 0.009 | 0.290 | 0.037 | 0.034 | 0.042 | 0.394 | 0.026 | -0.021 | 0.399 | 0.274 | 0.017 |
| 0.250 | 0.700 | 0.225 | 1.200 | 0.400 | 1.353 | 9.349 | 0.261 | -0.059 | 0.477 | 0.034 | 0.222 | 0.041 | 0.023 | 0.010 | 0.261 | 0.037 | 0.032 | 0.044 | 0.394 | 0.032 | -0.023 | 0.403 | 0.273 | 0.019 |
| 0.275 | 0.700 | 0.225 | 1.200 | 0.400 | 1.321 | 9.212 | 0.237 | -0.059 | 0.479 | 0.036 | 0.226 | 0.049 | 0.028 | 0.011 | 0.237 | 0.037 | 0.029 | 0.046 | 0.394 | 0.039 | -0.025 | 0.407 | 0.275 | 0.021 |
| 0.300 | 0.700 | 0.225 | 1.200 | 0.400 | 1.294 | 9.118 | 0.216 | -0.058 | 0.481 | 0.037 | 0.229 | 0.059 | 0.033 | 0.012 | 0.216 | 0.037 | 0.026 | 0.049 | 0.394 | 0.046 | -0.027 | 0.413 | 0.279 | 0.022 |
| 0.325 | 0.700 | 0.225 | 1.200 | 0.400 | 1.271 | 9.055 | 0.199 | -0.058 | 0.482 | 0.039 | 0.232 | 0.069 | 0.039 | 0.013 | 0.199 | 0.037 | 0.023 | 0.052 | 0.394 | 0.054 | -0.029 | 0.419 | 0.285 | 0.023 |
| 0.350 | 0.700 | 0.225 | 1.200 | 0.400 | 1.252 | 9.017 | 0.183 | -0.058 | 0.484 | 0.041 | 0.235 | 0.079 | 0.045 | 0.014 | 0.183 | 0.036 | 0.020 | 0.055 | 0.394 | 0.062 | -0.031 | 0.426 | 0.292 | 0.025 |
| 0.375 | 0.700 | 0.225 | 1.200 | 0.400 | 1.235 | 8.998 | 0.170 | -0.057 | 0.485 | 0.042 | 0.238 | 0.091 | 0.052 | 0.015 | 0.170 | 0.036 | 0.016 | 0.058 | 0.394 | 0.072 | -0.033 | 0.433 | 0.300 | 0.026 |
| 0.400 | 0.700 | 0.225 | 1.200 | 0.400 | 1.220 | 8.995 | 0.158 | -0.057 | 0.486 | 0.043 | 0.240 | 0.103 | 0.060 | 0.016 | 0.158 | 0.036 | 0.011 | 0.062 | 0.394 | 0.082 | -0.035 | 0.441 | 0.309 | 0.027 |
| 0.425 | 0.700 | 0.225 | 1.200 | 0.400 | 1.207 | 9.005 | 0.148 | -0.056 | 0.490 | 0.045 | 0.242 | 0.117 | 0.068 | 0.017 | 0.148 | 0.036 | 0.007 | 0.066 | 0.394 | 0.092 | -0.036 | 0.449 | 0.319 | 0.028 |
| 0.450 | 0.700 | 0.225 | 1.200 | 0.400 | 1.196 | 9.025 | 0.138 | -0.056 | 0.488 | 0.046 | 0.244 | 0.130 | 0.076 | 0.017 | 0.138 | 0.035 | 0.002 | 0.070 | 0.394 | 0.103 | -0.038 | 0.459 | 0.329 | 0.030 |
| 0.475 | 0.700 | 0.225 | 1.200 | 0.400 | 1.186 | 9.054 | 0.130 | -0.055 | 0.489 | 0.047 | 0.246 | 0.145 | 0.085 | 0.018 | 0.130 | 0.035 | -0.004 | 0.075 | 0.394 | 0.115 | -0.040 | 0.468 | 0.340 | 0.031 |
| 0.500 | 0.700 | 0.225 | 1.200 | 0.400 | 1.176 | 9.090 | 0.122 | -0.055 | 0.489 | 0.048 | 0.247 | 0.161 | 0.094 | 0.019 | 0.122 | 0.035 | -0.009 | 0.079 | 0.394 | 0.127 | -0.042 | 0.479 | 0.351 | 0.032 |
| 0.510 | 0.700 | 0.275 | 1.200 | 0.400 | 1.173 | 9.106 | 0.119 | -0.055 | 0.489 | 0.048 | 0.248 | 0.171 | 0.109 | 0.020 | 0.119 | 0.034 | -0.015 | 0.085 | 0.394 | 0.140 | -0.043 | 0.490 | 0.363 | 0.032 |
| 0.525 | 0.700 | 0.275 | 1.200 | 0.400 | 1.168 | 9.132 | 0.115 | -0.054 | 0.490 | 0.049 | 0.249 | 0.181 | 0.116 | 0.020 | 0.115 | 0.034 | -0.019 | 0.088 | 0.394 | 0.148 | -0.044 | 0.498 | 0.370 | 0.033 |
| 0.535 | 0.700 | 0.275 | 1.200 | 0.400 | 1.165 | 9.151 | 0.113 | -0.054 | 0.490 | 0.049 | 0.249 | 0.187 | 0.120 | 0.021 | 0.113 | 0.034 | -0.022 | 0.090 | 0.394 | 0.154 | -0.045 | 0.502 | 0.375 | 0.033 |
| 0.550 | 0.700 | 0.275 | 1.200 | 0.400 | 1.160 | 9.180 | 0.109 | -0.054 | 0.490 | 0.050 | 0.250 | 0.198 | 0.127 | 0.021 | 0.109 | 0.034 | -0.026 | 0.094 | 0.394 | 0.163 | -0.046 | 0.510 | 0.383 | 0.034 |
| 0.575 | 0.700 | 0.275 | 1.200 | 0.400 | 1.153 | 9.232 | 0.103 | -0.053 | 0.491 | 0.051 | 0.252 | 0.216 | 0.139 | 0.022 | 0.103 | 0.034 | -0.033 | 0.100 | 0.394 | 0.178 | -0.048 | 0.523 | 0.396 | 0.035 |
| 0.585 | 0.700 | 0.275 | 1.200 | 0.400 | 1.151 | 9.254 | 0.101 | -0.053 | 0.491 | 0.051 | 0.252 | 0.224 | 0.144 | 0.023 | 0.101 | 0.034 | -0.036 | 0.102 | 0.394 | 0.184 | -0.049 | 0.529 | 0.401 | 0.035 |
| 0.600 | 0.700 | 0.275 | 1.200 | 0.420 | 1.147 | 9.288 | 0.098 | -0.053 | 0.491 | 0.052 | 0.253 | 0.235 | 0.152 | 0.023 | 0.098 | 0.034 | -0.041 | 0.107 | 0.396 | 0.193 | -0.049 | 0.541 | 0.411 | 0.037 |
| 0.650 | 0.700 | 0.275 | 1.200 | 0.420 | 1.136 | 9.411 | 0.089 | -0.052 | 0.492 | 0.054 | 0.255 | 0.276 | 0.179 | 0.025 | 0.089 | 0.033 | -0.057 | 0.121 | 0.396 | 0.227 | -0.053 | 0.571 | 0.437 | 0.039 |
| 0.700 | 0.700 | 0.275 | 1.200 | 0.440 | 1.126 | 9.545 | 0.081 | -0.051 | 0.493 | 0.055 | 0.257 | 0.319 | 0.208 | 0.026 | 0.081 | 0.033 | -0.074 | 0.137 | 0.399 | 0.263 | -0.055 | 0.608 | 0.466 | 0.042 |
| 0.800 | 0.700 | 0.275 | 1.200 | 0.460 | 1.110 | 9.837 | 0.068 | -0.049 | 0.494 | 0.058 | 0.261 | 0.416 | 0.272 | 0.029 | 0.068 | 0.033 | -0.114 | 0.173 | 0.402 | 0.344 | -0.061 | 0.685 | 0.524 | 0.047 |
| 0.900 | 0.700 | 0.275 | 1.200 | 0.480 | 1.098 | 10.152 | 0.058 | -0.047 | 0.495 | 0.061 | 0.264 | 0.525 | 0.346 | 0.031 | 0.058 | 0.032 | -0.160 | 0.213 | 0.405 | 0.435 | -0.066 | 0.774 | 0.584 | 0.053 |
| 1.000 | 0.700 | 0.275 | 1.200 | 0.500 | 1.088 | 10.484 | 0.050 | -0.045 | 0.496 | 0.063 | 0.267 | 0.647 | 0.428 | 0.034 | 0.050 | 0.031 | -0.211 | 0.258 | 0.407 | 0.537 | -0.071 | 0.874 | 0.644 | 0.059 |

**APPENDIX B: Verification of SENSTUDY through Comparison with PROKON Analyses**

The validity of the results obtained from the spreadsheet programme SENSTUDY is evaluated through comparison with a PROKON analyses. The building used for the comparison is a portal frame building as shown in Figure 2 with the following parameters:

- Building span L = 20m
- Eaves height h = 6m
- Frame spacing s = 5m
- Roof angle $\theta$ = 10°
- Stiffness ratio $I_2/I_1$ = 1

The loads and load conditions for the building are as follows:

- Gravitational loads
  - Dead load $D_n$ = 0.35 kN/m$^2$ (including own weight)
  - Imposed load $L_n$ = 0.3 kN/m$^2$
- Wind Loads $W_n$
  - Wind pressure $q_z$ = 0.848 kN/m$^2$ (for Terrain Category 2)
  - External wind pressure coefficients
    - For the windward wall: $C_{1,wall}$ = 0.7
    - For the leeward wall: $C_{2, wall}$ = 0.225
    - For the windward half of the roof: $C_{1, roof}$ = 1.2
    - For the leeward half of the roof: $C_{2, roof}$ = 0.4

The load combinations used are the gravitational and wind load combinations:

- $Q_n = 1.2D_n + 1.6L_n$     (gravitational)
- $Q_n = 0.9D_n + 1.3W_n$     (wind)

The results of the comparison are summarised in Table B1. The values in Table B1 are to be checked with the output pages of SENSTUDY and PROKON provided hereafter. Note that the values for the load effects as determined through SENSTUDY are in terms of $M/L^2$ per meter frame spacing. Therefore, the values shown in Table B.1 are obtained by multiplying the values shown in SENSTUDY by 2000 (= 20$^2$ x 5).

**Table B.1. Comparison of SENSTUDY with PROKON Analyses**

| | | SENSTUDY (kN.m) | PROKON (kN.m) |
|---|---|---|---|
| Gravitational Load Combination | $M_{eaves}$ | -117 | -117 |
| | $M_{roof}$ | 74 | 74 |
| Wind Load Combination | $M_{eaves}$ at windward column | 98 | 96 |
| | $M_{eaves}$ at leeward column | 52 | 51 |
| | $M_{roof}$ | -44 | -46 |

As is evident from Table B1, the results obtained from SENSTUDY compare favourably with those of the PROKON analyses.

| PROKON | Job Number | | Sheet |
| --- | --- | --- | --- |
| Software Consultants (Pty) Ltd | Job Title | | |
| Internet: http://www.prokon.com | Client | | |
| E-Mail : mail@prokon.com | Calcs by | Checked by | Date |

## APPENDIX B

**X-Moments for Load Combination D+L** (gravitational load combination)
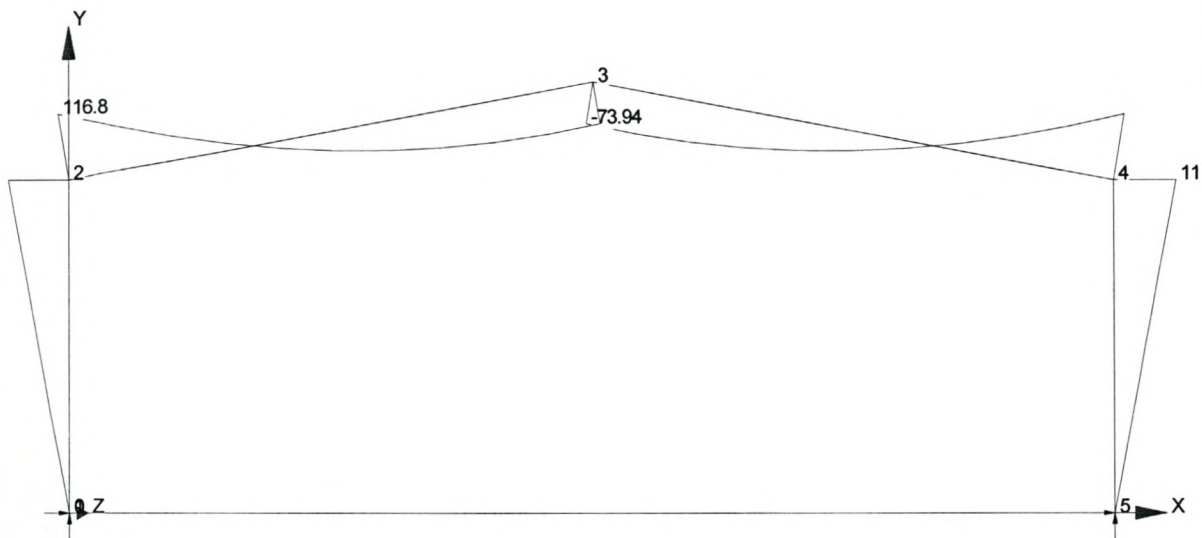


**X-Moments for Load Combination D+W** (wind load combination)
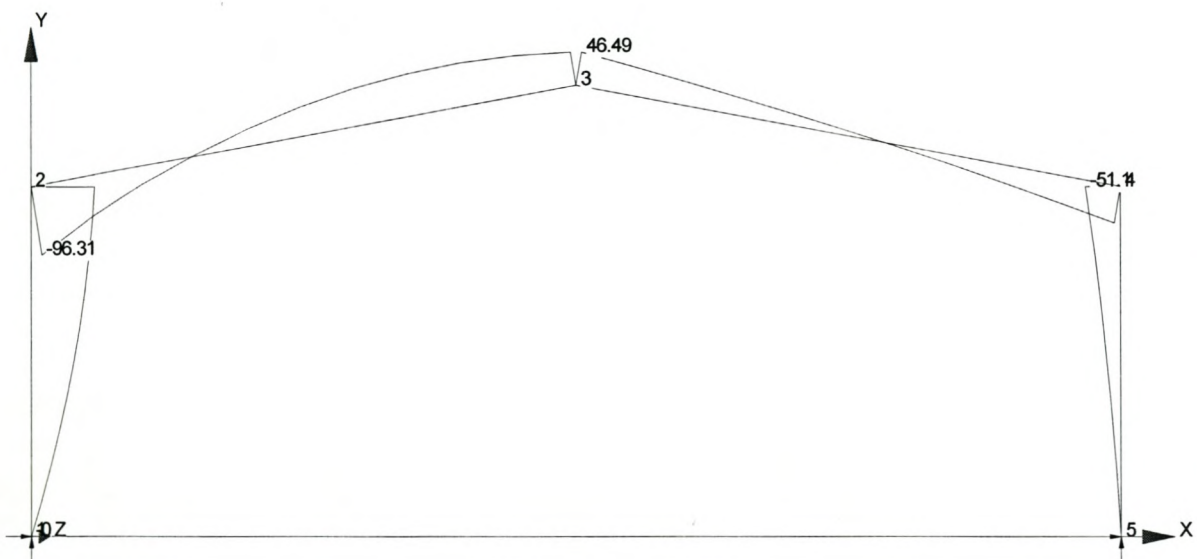
```
========= S p a c e - F r a m e   A n a l y s i s  -  P R O K O N =========
                     Ver W1.9.08 - 19 Nov 2002

TITLE :APPENDIX B

Data file : C:\JdeV\Appendix B.A03
Created on: 2/19/03

========================= NODAL POINT COORDINATES ============================

Node no. X-coord    Y-coord    Z-coord  Node no. X-coord    Y-coord    Z-coord
         m          m          m                 m          m          m

1        0.000      0.000      0.000    2        0.000      6.000      0.000
3       10.000      7.760      0.000    4       20.000      6.000      0.000
5       20.000      0.000      0.000

============================= ELEMENT DATA ===================================

Beam         Secn. type        Fixity    Length       ß
                                         m           (°)

1-2          COL                 00       6.000      0.00
2-3          RAFTER              00      10.154      0.00
3-4          RAFTER              00      10.154      0.00
4-5          COL                 00       6.000      0.00

=========================== SECTION PROPERTIES ===============================

Section : COL    Section designation: 356x171x45     I1
----------------
   A        Ixx       Iyy       J       Material
  m^2       m^4       m^4       m^4

 5.700E-3  121E-6   8.10E-6   160E-9   Steel:300W

Section : RAFTER Section designation: 356x171x45     I1
----------------
   A        Ixx       Iyy       J       Material
  m^2       m^4       m^4       m^4

 5.700E-3  121E-6   8.10E-6   160E-9   Steel:300W

=============================== MATERIALS ====================================

Designation              E       poisson    Density    Exp. coeff.
                         kPa                 kN/m^3
Steel:300W               206.0E6  0.30       77.00      11.70E-6

============================== SUPPORT DATA ==================================

                          Prescribed displacements
Node    Fixity    X         Y         Z        X-Rot     Y-Rot     Z-Rot
                  m         m         m        rad.      rad.      rad.

1       XY        0.00      0.00      0.00     0.00      0.00      0.00
5       XY        0.00      0.00      0.00     0.00      0.00      0.00

                          Spring constants
Node    Fixity    X         Y         Z        X-Rot     Y-Rot     Z-Rot
                  kN/m      kN/m      kN/m     kNm/rad   kNm/rad   kNm/rad



================================ LOADS =======================================

Load Case  Description

DEAD_L     Dead Load
LIVE_L     Live Load
WWALLS     Wind load on the walls
WROOF      Wind load on the roof

Own weight not added to any load case/combination
```

210

| Job Number | | Sheet |
|---|---|---|
| Job Title | | |
| Client | | |
| Calcs by | Checked by | Date |

```
========================== LOAD CASE DEAD_L =============================
Dead Load

*** BEAM ELEMENT LOADS ***

Element     Direction       P          a          Wl          Wr          dT
                            kN          m         kN/m        kN/m         °C

2-3            Y          0.00       0.00       -1.75       -1.75        0.00
3-4            Y          0.00       0.00       -1.75       -1.75        0.00

========================== LOAD CASE LIVE_L =============================
Live Load

*** BEAM ELEMENT LOADS ***

Element     Direction       P          a          Wl          Wr          dT
                            kN          m         kN/m        kN/m         °C

2-3            Y          0.00       0.00       -1.50       -1.50        0.00
3-4            Y          0.00       0.00       -1.50       -1.50        0.00

========================== LOAD CASE WWALLS =============================


*** BEAM ELEMENT LOADS ***

Element     Direction       P          a          Wl          Wr          dT
                            kN          m         kN/m        kN/m         °C

1-2            X          0.00       0.00        2.97        2.97        0.00
4-5            X          0.00       0.00        0.95        0.95        0.00

========================== LOAD CASE WROOF  =============================


*** BEAM ELEMENT LOADS ***

Element     Direction       P          a          Wl          Wr          dT
                            kN          m         kN/m        kN/m         °C

2-3            L          0.00       0.00        5.09        5.09        0.00
3-4            L          0.00       0.00        1.70        1.70        0.00

========================== LOAD COMBINATIONS ============================

Load Comb   Description

D+L         Gravitational combination
D+W         Wind load combination

Comb.    Load factor for each load case: Ultimate Limit State

         DEAD_L LIVE_L WWALLS WROOF

D+L      1.2    1.6    0.0    0.0
D+W      0.9    0.0    1.3    1.3

Comb.    Load factor for each load case: Serviceability Limit State

         DEAD_L LIVE_L WWALLS WROOF

D+L      1.0    1.0    0.0    0.0
D+W      1.0    0.0    1.0    1.0

========================= OUTPUT: LINEAR ANALYSIS ======================
============ BEAM ELEMENT END FORCES IN LOCAL ELEMENT AXES at ULS =========

Elem     Lcase     Axial    Y-Shear     M-xx      Axial     Y-Shear     M-xx
                    kN        kN         kNm        kN         kN        kNm

1-2      D+L       45.00     -19.47      0.00     -45.00      19.47     -116.80
         D+W      -41.29      27.63     -0.00      41.29      -4.48       96.31
2-3      D+L       26.97      40.94    116.80     -19.17       3.37       73.94
         D+W      -11.57     -39.89    -96.31      14.30     -11.76      -46.49
3-4      D+L       19.17       3.37    -73.94     -26.97      40.94     -116.80
         D+W      -17.45       6.17     46.49      14.72     -13.05       51.10
```

```
4-5        D+L       45.00      19.47     116.80     -45.00     -19.47      -0.00
           D+W      -15.40     -12.24     -51.10      15.40       4.80      -0.00

============================= STATISTICAL DATA ================================

Own weight of structure = 0.00 kN

No. of real numbers in Stiffness matrix = 249   (1992 bytes)
Time used to analyse = 0: 0:0.010 seconds
Total number of : Nodes              = 5
                  Beam Elements      = 4
                  Shell Elements     = 0
                  Supports           = 2
                  Section properties = 2
                  Load cases         = 4
                  Load combinations  = 2

============================== END OF OUTPUT =================================
```

**APPENDIX C: Quantitative Expert Opinions**

The quantitative expert opinions are presented in Table C.1 hereafter.

Table C.1. Experts' Opinions elicited from Expert Survey

| Expert | Seed Variable 1* | | | Seed Variable 2 | | | Seed Variable 3 | | | Maximum Variable 1* | | | Maximum Variable 2 | | | Maximum Variable 3 | | | Maximum Variable 4* | | | Maximum Variable 5 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 5%-value | 50%-value | 95%-value | 5%-value | 50%-value | 95%-value | 5%-value | 50%-value | 95%-value | 5%-value | 50%-value | 95%-value | 5%-value | 50%-value | 95%-value | 5%-value | 50%-value | 95%-value | 5%-value | 50%-value | 95%-value | 5%-value | 50%-value | 95%-value |
| H Loubscher | 3.67 | 6.67 | 9.67 | 0.99 | 1.00 | 1.01 | 1.05 | 1.50 | 1.95 | 3.67 | 6.67 | 9.67 | 1.98 | 2.00 | 2.02 | 1.05 | 1.50 | 1.95 | 3.67 | 6.67 | 9.67 | 0.99 | 1.00 | 1.01 |
| IP de Villiers | 2.10 | 3.00 | 3.90 | 1.05 | 1.50 | 1.95 | 1.10 | 2.00 | 2.90 | 2.10 | 3.00 | 3.90 | 1.05 | 1.50 | 1.95 | 1.10 | 2.00 | 2.90 | 2.10 | 3.00 | 3.90 | 1.05 | 1.50 | 1.95 |
| PJ de Villiers | 2.00 | 2.86 | 5.00 | 0.99 | 1.00 | 1.01 | 1.10 | 2.00 | 2.90 | 4.00 | 5.00 | 8.89 | 3.96 | 4.00 | 4.04 | 1.10 | 2.00 | 2.90 | 1.15 | 2.50 | 3.85 | 1.98 | 2.00 | 2.02 |
| F Heyman | 2.40 | 6.00 | 9.60 | 1.05 | 1.50 | 1.95 | 1.05 | 1.50 | 1.95 | 2.40 | 6.00 | 9.60 | 1.05 | 1.50 | 1.95 | 1.05 | 1.50 | 1.95 | 1.05 | 1.50 | 1.95 | 0.99 | 1.00 | 1.01 |
| W hugo | 4.80 | 12.00 | 19.20 | 0.99 | 1.00 | 1.01 | 1.15 | 2.50 | 3.85 | 4.80 | 12.00 | 19.20 | 0.99 | 1.00 | 1.01 | 1.15 | 2.50 | 3.85 | 2.10 | 3.00 | 3.90 | 0.99 | 1.00 | 1.01 |
| G Bastiaanse | 2.50 | 3.33 | 4.00 | 1.00 | 1.50 | 2.00 | 1.05 | 1.50 | 1.95 | 4.00 | 5.00 | 5.71 | 1.00 | 2.00 | 4.00 | 1.05 | 1.50 | 1.95 | 2.00 | 2.50 | 2.86 | 1.05 | 1.50 | 1.95 |
| W Jordaan | 2.50 | 2.86 | 3.33 | 0.99 | 1.00 | 1.01 | 1.05 | 1.50 | 1.95 | 2.86 | 4.00 | 5.00 | 1.98 | 2.00 | 2.02 | 1.05 | 1.50 | 1.95 | 2.00 | 2.50 | 3.33 | 1.98 | 2.00 | 2.02 |
| G Adema | 2.22 | 2.86 | 3.33 | 0.99 | 1.00 | 1.01 | 0.99 | 1.00 | 1.01 | 2.86 | 4.00 | 6.67 | 1.98 | 2.00 | 2.02 | 0.99 | 1.00 | 1.01 | 1.67 | 2.50 | 4.00 | 0.99 | 1.00 | 1.01 |
| A Davis | 3.33 | 4.00 | 5.00 | 0.99 | 1.00 | 1.01 | 1.00 | 1.50 | 2.00 | 3.33 | 5.00 | 10.00 | 2.97 | 3.00 | 3.03 | 2.00 | 2.50 | 3.00 | 2.00 | 2.50 | 3.33 | 1.98 | 2.00 | 2.02 |
| W Kleinhans | 2.86 | 3.33 | 4.00 | 0.99 | 1.00 | 1.01 | 1.10 | 2.00 | 2.90 | 2.86 | 5.00 | 10.00 | 1.98 | 2.00 | 2.02 | 1.10 | 2.00 | 2.90 | 2.86 | 3.33 | 4.00 | 1.98 | 2.00 | 2.02 |
| A Eckermans | 2.50 | 3.33 | 4.00 | 0.99 | 1.00 | 1.01 | 1.05 | 1.50 | 1.95 | 2.50 | 4.00 | 5.00 | 0.99 | 1.00 | 1.01 | 1.05 | 1.50 | 1.95 | 2.22 | 2.86 | 4.00 | 0.99 | 1.00 | 1.01 |
| F van Zyl | 1.45 | 5.50 | 9.55 | 1.15 | 2.50 | 3.85 | 1.15 | 2.50 | 3.85 | 1.45 | 5.50 | 9.55 | 1.15 | 2.50 | 3.85 | 1.15 | 2.50 | 3.85 | 1.05 | 1.50 | 1.95 | 1.10 | 2.00 | 2.90 |
| A Ellmer | 2.50 | 2.86 | 3.33 | 0.99 | 1.00 | 1.01 | 0.99 | 1.00 | 1.01 | 3.33 | 4.00 | 5.71 | 1.98 | 2.00 | 2.02 | 1.98 | 2.00 | 2.02 | 3.33 | 4.00 | 5.71 | 1.98 | 2.00 | 2.02 |
| E Houting | 2.08 | 5.83 | 9.58 | 1.05 | 1.50 | 1.95 | 1.10 | 2.00 | 2.90 | 2.08 | 5.83 | 9.58 | 1.05 | 1.50 | 1.95 | 1.10 | 2.00 | 2.90 | 1.47 | 2.67 | 3.87 | 0.99 | 1.00 | 1.01 |
| P Storey | 2.90 | 11.00 | 19.10 | 1.10 | 2.00 | 2.90 | 1.05 | 1.50 | 1.95 | 2.90 | 11.00 | 19.10 | 1.10 | 2.00 | 2.90 | 1.05 | 1.50 | 1.95 | 2.40 | 6.00 | 9.60 | 1.05 | 1.50 | 1.95 |
| D Payne | 4.95 | 5.00 | 5.05 | 0.99 | 1.00 | 1.01 | 0.99 | 1.00 | 1.01 | 6.60 | 6.67 | 6.73 | 1.98 | 2.00 | 2.02 | 1.98 | 2.00 | 2.02 | 3.96 | 4.00 | 4.04 | 0.99 | 1.00 | 1.01 |
| A Loins | 3.33 | 4.00 | 5.00 | 0.99 | 1.00 | 1.01 | 1.00 | 1.50 | 2.00 | 4.00 | 5.00 | 6.67 | 1.98 | 2.00 | 2.02 | 2.00 | 2.50 | 3.00 | 2.50 | 2.86 | 3.33 | 0.99 | 1.00 | 1.01 |
| J Jacobs | 3.96 | 4.00 | 4.04 | 1.00 | 1.50 | 2.00 | 0.99 | 1.00 | 1.01 | 3.96 | 4.00 | 4.04 | 1.00 | 2.00 | 3.00 | 0.99 | 1.00 | 1.01 | 3.96 | 4.00 | 4.04 | 1.00 | 2.00 | 3.00 |
| G McNeil | 8.00 | 9.00 | 10.00 | 0.99 | 1.00 | 1.01 | 1.00 | 2.00 | 3.00 | 8.00 | 9.00 | 10.00 | 0.99 | 1.00 | 1.01 | 2.00 | 3.00 | 4.00 | 6.00 | 7.00 | 8.00 | 0.99 | 1.00 | 1.01 |
| J van Breda | 2.86 | 3.33 | 5.00 | 1.00 | 1.50 | 2.00 | 1.00 | 2.00 | 3.00 | 5.00 | 6.67 | 10.00 | 1.00 | 2.00 | 4.00 | 2.00 | 4.00 | 7.00 | 2.00 | 2.86 | 3.33 | 1.05 | 1.50 | 1.95 |
| C Eksteen | 2.50 | 2.86 | 4.00 | 0.99 | 1.00 | 1.01 | 0.75 | 1.00 | 1.25 | 2.86 | 4.00 | 4.44 | 0.99 | 1.00 | 1.01 | 0.75 | 1.00 | 1.25 | 2.00 | 2.22 | 2.50 | 0.99 | 1.00 | 1.01 |
| I Gillmore | 2.50 | 2.86 | 3.33 | 2.00 | 2.50 | 3.00 | 1.00 | 1.50 | 2.00 | 2.86 | 3.33 | 4.00 | 2.00 | 3.00 | 4.00 | 1.00 | 1.50 | 2.00 | 2.00 | 2.50 | 3.33 | 1.00 | 2.00 | 3.00 |
| D Scott | 3.96 | 4.00 | 4.04 | 0.99 | 1.00 | 1.01 | 0.99 | 1.00 | 1.01 | 4.95 | 5.00 | 5.05 | 1.98 | 2.00 | 2.02 | 1.98 | 2.00 | 2.02 | 1.98 | 2.00 | 2.02 | 1.98 | 2.00 | 2.02 |
| C Lutzeller | 2.22 | 2.67 | 3.33 | 0.99 | 1.00 | 1.01 | 1.00 | 1.50 | 2.00 | 2.50 | 3.33 | 5.00 | 0.99 | 1.00 | 1.01 | 1.00 | 1.50 | 2.00 | 1.67 | 1.82 | 2.00 | 0.99 | 1.00 | 1.01 |
| A Kilpin | 2.50 | 2.67 | 2.86 | 0.99 | 1.00 | 1.01 | 0.99 | 1.00 | 1.01 | 2.86 | 3.08 | 3.33 | 1.98 | 2.00 | 2.02 | 1.98 | 2.00 | 2.02 | 1.67 | 2.22 | 2.86 | 0.99 | 1.00 | 1.01 |
| G Lackey | 1.82 | 2.00 | 2.50 | 0.99 | 1.00 | 1.01 | 0.99 | 1.00 | 1.01 | 2.22 | 2.50 | 2.86 | 0.99 | 1.00 | 1.01 | 0.99 | 1.00 | 1.01 | 1.33 | 1.67 | 2.00 | 0.99 | 1.00 | 1.01 |
| M Papanicolau | 2.67 | 3.33 | 3.64 | 1.00 | 1.50 | 2.00 | 1.00 | 1.50 | 2.00 | 4.00 | 4.44 | 6.67 | 2.00 | 3.00 | 5.00 | 2.00 | 4.00 | 7.00 | 2.00 | 2.86 | 3.08 | 1.00 | 2.00 | 3.00 |
| W du Plessis | 3.64 | 4.00 | 4.44 | 0.99 | 1.00 | 1.01 | 0.99 | 1.00 | 1.01 | 4.00 | 4.44 | 5.00 | 1.98 | 2.00 | 2.02 | 1.98 | 2.00 | 2.02 | 2.00 | 2.22 | 2.50 | 0.99 | 1.00 | 1.01 |
| Foreman #1 | 4.95 | 5.00 | 5.05 | 0.99 | 1.00 | 1.01 | 1.00 | 1.50 | 2.00 | 4.95 | 5.00 | 5.05 | 0.99 | 1.00 | 1.01 | 1.00 | 1.50 | 2.00 | 1.98 | 2.00 | 2.02 | 0.99 | 1.00 | 1.01 |
| Foreman #2 | 4.95 | 5.00 | 5.05 | 0.99 | 1.00 | 1.01 | 0.99 | 1.00 | 1.01 | 4.95 | 5.00 | 5.05 | 0.99 | 1.00 | 1.01 | 0.99 | 1.00 | 1.01 | 1.98 | 2.00 | 2.02 | 0.99 | 1.00 | 1.01 |
| Foreman #3 | 5.66 | 5.71 | 5.77 | 0.99 | 1.00 | 1.01 | 0.99 | 1.00 | 1.01 | 5.66 | 5.71 | 5.77 | 1.98 | 2.00 | 2.02 | 1.98 | 2.00 | 2.02 | 2.48 | 2.50 | 2.53 | 1.98 | 2.00 | 2.02 |

*Seed Variable 1, Maximum Variable 1 and Maximum Variable 4 are in terms of the number of workers on a 20m-spanning frame

## APPENDIX D: Philosophy-of-Design Expert Opinions

### D1. Civil Engineers

The philosophy-of-design opinions of the civil engineers that took part in the expert survey are set forth in Tables D1.1 & D1.2 hereafter.

### D2. Steel- and Roofing Contractors

The philosophy-of-design opinions of the steel- and roofing contractors that took part in the expert survey are set forth in Table D2.1 hereafter.

**Table D1.1. Alternative Sources of the Imposed Roof Load and the Appropriate Treatment thereof**

| Expert | Provision for stacking of roof cladding on purlins | | Provision for the weight of services suspended from the roof. | | | Are there any other special cases that produces a large imposed roof load. For example, during the installation of services or equipment in the roof? Or equipment hanging from the roof. How should these cases be provided for? |
|---|---|---|---|---|---|---|
| | Does Stacking on Purlins occur? | Does SABS need to provide for overstacking through it's prescribed live roof load? Provide reason. | Weight of services | | How do you provide for the weight of services suspended from the roof (such as waterpipes, lighting and air-conditioning)? | |
| | | | average (kN/m$^2$) | max (kN/m$^2$) | | |
| H. Loubscher Pr. Ing | Occurs, as a result of negligence of building contractors | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. | 0.05 - 0.1 | 0.1 - 0.15 | When there is small uncertainty in the geometry of the services, I use 0.1 kN/m2 dead load, otherwhise where larger uncertainty exists, such as for large shopping centres I use 0.1 kN/m2, live load. | No, I have not encountered any such cases in my experience. Few people are involved in installation of services and equipment involved are supported by the ground. The code cannot provide for equipment hanging from roof as this should be determined for each specific case. |
| I.H. de Villiers PhD structures Pr. Ing | Never occurs | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. | 0.1 | 0.15 | For normal cases, I use 0.1 kN/m2 dead load. For special cases, such as large water pipes in the roof (of which I should be made aware of by the client) I determine the weight for that specific case and treat it as a dead load. | No, workmen involved in installation of services are easily covered by the 0.3 kn/m2 of the code. Equipment suspended from the roof should be brought under the engineers attention by the client and should be provided for in addition to the 0.3 kN/m2 of the code |
| P.J de Villiers Pr. Ing | Never occurs | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. | 0.05 - 0.1 | 0.15 | For normal cases, I use 0.1 kN/m2 dead load. For special cases, such as large water pipes in the roof (of which I should be made aware of by the client) I determine the weight for that specific case and treat it as a dead load. | No, workmen involved in installation of services are easily covered by the 0.3 kn/m2 of the code. Equipment suspended from the roof should be brought under the engineers attention by the client and should be provided for in addition to the 0.3 kN/m2 of the code |

| | | | | | | |
|---|---|---|---|---|---|---|
| F. Hayman Pr. Ing | Occurs, as a result of negligence from building contractors | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. If structural damage occurs during construction as a result of overstacking or weight of workmen, the building contractor should correct it on site at his own expense. | 0.1 | 0.2 | For normal cases, I use 0.1 kN/m2 dead load. For special cases, such as large water pipes in the roof (of which I should be made aware of by the client) I determine the weight for that specific case and treat it as a dead load. I know of engineers who accept that the 0.3 kN/m2 live roof load value of the SABS makes provision for services. | Yes, during occupancy changes of the building or due to incompetent contractors. An extreme situation would be where the building is situated next to a sports ground and spectators would climb onto the roof. |
| W. Hugo Pr. Ing | Never occurs | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. | 0.1 | 0.15 | When there is small uncertainty in the geometry of the services, I use 0.1 kN/m2 dead load, otherwise where larger uncertainty exists, such as for large shopping centres I use 0.1 kN/m2, live load. | No, I have not encountered any such cases in my experience. Few people are involved in installation of services and equipment involved are supported by the ground. The code cannot provide for equipment hanging from roof as this should be determined for each specific case. |
| G. Basteaanse Pr. Ing | Occurs, as a result of negligence from building contractors | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. | 0.15 | 0.2 | When there is small uncertainty in the geometry of the services, I use 0.15 kN/m2 dead load, otherwise where larger uncertainty exists, such as for large shopping centres I use 0.2 kN/m2, dead load. | It rarely occurs. I have dealt with large waterpipes hanging from the roof (of 400 - 500mm diameter). Such instances should be brought under the engineers attention so that he can provide separately for them. |
| W. Jordaan Pr. Tech | Occurs, as a result of negligence from building contractors | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. | 0.25 | 0.3 | Normally use 0.25 kN/m2 live load. This is conservative but I feel it is necessary when the geometry and the weight of the services are not certain. | Some air-conditioning systems may cause large point loads on the frame. Such instances should be brought under the engineers attention so that he can provide separately for them. |

| | | | | | | |
|---|---|---|---|---|---|---|
| G. Adema<br>Pr. Ing | Occurs, as a result of negligence from building contractors | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. | 0.1 | 0.2 | When the services are known, calculate the weight and treat as dead load. Usually 0.1 kN/m2 is sufficient for normal types of services such as lighting, water sprinkler systems and air-conditioning. | Not aware of any such cases |
| A. Davis<br>Pr. Ing | Occurs, as a result of negligence from building contractors | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. | 0.15 | 0.2 | Ask from the client the type of services, calculate the weight and treat as dead load. Usually 0.1 kN/m2 is sufficient for normal types of services such as lighting, water sprinkler systems and air-conditioning. | A special case would be where building contractors abuse the rafters or purlins in the erection of roof sheeting |
| W. Kleinhans<br>Pr. Ing | Occurs, as a result of negligence from building contractors | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. | 0.1 | - | Normally I use 0.1 kN/m2 live load. This is if I do not hear from the electrical or mechanical engineers of any heavy equipment that will be hanging from the roof. | Cannot name any such cases. Abnormal loads should be reported to the engineer. |
| A. Eckermans<br>Pr. Ing | Occurs, as a result of negligence from building contractors | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. | 0.1 | - | Normally I use 0.1 kN/m2 live load. Special cases should be reported. | Not aware of any such cases. Special cases should be reported. |

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| | | | | | | |
|---|---|---|---|---|---|---|
| F. van Zyl PhD structures Pr. Ing | Never occurs | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. | 0.1 | 0.15 | For normal cases, I use 0.1 kN/m2 dead load. For special cases, such as large water pipes in the roof (of which I should be made aware of by the client) I determine the weight for that specific case and treat it as a dead load. | No, workmen involved in installation of services are easily covered by the 0.3 kn/m2 of the code. Equipment suspended from the roof should be brought under the engineers attention by the client and should be provided for in addition to the 0.3 kN/m2 of the code |
| A. Ellmer Pr. Ing | Never occurs | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. | 0.2 - 0.3 | 0.3 | In preliminary design I use 0.2 kN/m2 dead load. This is for typical services such as electric cabels and water sprinklers. This is on the safe side. | No, the load due to workmen on the roof during installation is nominal. Equipment hanging from the roof I treat as dead load. |
| E. Houghting Pr. Ing | Occurs, as a result of negligence from building contractors | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. | 0.1-0.15 | 0.15 | For normal cases, I use 0.1 kN/m2 dead load. For special cases, such as large water pipes in the roof (of which I should be made aware of by the client) I determine the weight for that specific case and treat it as a dead load. | Not many such instances. I have come across cases where fittings of up to 120 kg where installed in the roof, therefore large point loads should be catered for. Such instances should be brought under the engineers attention so that he can calculate the weight and make provision for it. |
| P Storey Pr. Ing | Occurs, as a result of negligence from building contractors | No, building contractors' responsibility to ensure no overstacking, therefore weight of sheets is dead load. | 0.1-0.2 | 0.2 | I use 0.2 kN/m2 dead load. This is for typical services such as electric cabels and water sprinklers. This is on the safe side. | No, the load due to workmen on the roof during installation is nominal. Equipment hanging from the roof I treat as dead load. |

**Table D1.2. General Aspects regarding Imposed Roof Loads**

| Expert | What is the engineers' responsibility in terms of occupancy changes during the lifetime of the structure? | What is the reason for the overseas loading codes (ASCE, BS, Euro) having a higher imposed roof load for inaccessible roofs than the SABS has? | Any comments on the current prescribed load intensities of the SABS for the imposed roof load? |
|---|---|---|---|
| H. Loubscher Pr. Ing | It is not possible to provide for occupancy changes in the original design. The client and contractors is responsible for notifying the engineer if any changes are to take place | Snow, as well as more severe wheather conditions in general in these countries. | The current values of SABS are just about right. Sufficient conservatism is provided. |
| I.P. de Villiers PhD structures Pr. Ing | It is not possible to provide for occupancy changes in the original design. The client and contractors is responsible for notifying the engineer if any changes are to take place | Snow, as well as more severe wheather conditions in general in these countries. | The current values of SABS are a bit too conservative. |
| P.J de Villiers Pr. Ing | The engineer has to re-design the structure for its new purpose / loading. Uneconomical to increase the prescribed values to compensate for such changes. | Snow, as well as more severe wheather conditions in general in these countries. | The current values of SABS are a bit too conservative. |
| F. Hayman Pr. Ing | The engineer has to design for the clients needs at that stage. In case of changes the engineer should be consulted with. | Snow, as well as more severe wheather conditions in general in these countries. | The current values of the SABS are acceptable. |
| W. Hugo Pr. Ing | It is not possible to provide for occupancy changes in the original design. The client and contractors is responsible for notifying the engineer if any changes are to take place | Snow, as well as more severe wheather conditions in general in these countries. | The current values of SABS are a bit too conservative. |
| G. Basteaanse Pr. Ing | The engineer has no responsibility to design for unknown future usage changes. | Snow. All prescribed loads are in general higher overseas than in SA because more money is available. | The current values of SABS are a bit too conservative. |
| W. Jordaan Pr. Tech | The engineer has to design for the clients needs at that stage. In case of changes the engineer should be consulted with. | Snow, as well as more severe wheather conditions in general in these countries. | The current values of the SABS are acceptable. |
| G. Adema Pr. Ing | The engineer has to design for the clients needs at that stage. In a less competitive arena it would be advantageous to design with some flexibility. | Snow, as well as more severe wheather conditions in general in these countries. | The current values of the SABS are acceptable. |
| A. Davis Pr. Ing | The engineer has to satisfy the clients needs at that stage. You should cater for minor changes, but to design for all possible loading situations would result in uneconomical structures. | More severe wheather conditions in these overseas countries. Example, in Aus where the climate isn't much different than ours, the prescribed live roof load is very similar to that of the SABS. | If the SABS is to allow for construction loads, the values are non - conservative, otherwhise it is acceptable. |

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| | | | |
|---|---|---|---|
| W. Kleinhans<br>Pr. Ing | It is not possible to provide for occupancy changes in the original design. The client and contractors is responsible for notifying the engineer if any changes are to take place | Snow, as well as more severe wheather conditions in general in these countries. | The current values of the SABS are acceptable. |
| A. Eckermans<br>Pr. Ing | It is not possible to provide for occupancy changes in the original design. The client and contractors is responsible for notifying the engineer if any changes are to take place | Snow, as well as more severe wheather conditions in general in these countries. | The current values of the SABS are acceptable. |
| F. van Zyl<br>PhD structures<br>Pr. Ing | The engineer has to design for the clients needs at that stage. In case of changes the engineer should be consulted with. | Snow, as well as more severe wheather conditions in general in these countries. | The current values of the SABS are acceptable. |
| A. Ellmer<br>Pr. Ing | The engineer has to design for the clients needs at that stage, and he should inform the client for what purpose the building is designed for, and that any changes are to be reported to the engineer beforehand. | Snow, as well as more severe wheather conditions in general in these countries. Specifically Germany is more conservative. This is also due to the fact that more money is available than in developing countries. | Sufficient reliability is not obtained with the current SABS roof live loads. |
| E. Houghting<br>Pr. Ing | Not responsible. In SA it is necessary to design as economically as possible to be competitive. The engineer should establish from the client, the degree of flexibility he wants in the structure. | In these overseas countries it is possible to design buildings for multi purposes because of more availible money. Also, there are more extreme wheather conditions, for example snow and ice. | Sufficient reliability is not obtained with the current SABS roof live loads. |
| P. Storey<br>Pr. Ing | The engineer has to design for the clients needs at that stage. In case of changes the engineer should be consulted with. | Snow, as well as more severe wheather conditions in general in these countries. | The current values of the SABS are acceptable. |

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**Table D2.1 Questions to Steel- and Roofing Contractors regarding the limiting of Uncertainty**

| Expert | Does it happen that workers congregate on a small area, and if so what are the reasons for this? | Do you provide any additional support for the weight of stacked materials and workmen? | Are there any heavy equipment involved (> 20kg) in erecting the sheeting | Are you aware of any roof sheeting companies that use erection methods that are extreme? (damaging to the structure) | Are there any other operations on the roof causing significant roof loads |
|---|---|---|---|---|---|
| D Payne | It may happen if there is no supervision. There are no specific reasons for them to congregate. | No | No | No | No |
| A Loins | It may happen if there is no supervision. There are no specific reasons for them to congregate. | No | No | No | No |
| J Jacobs | It may happen if there is no supervision. There are no specific reasons for them to congregate. | No | No | No | No |
| G McNeil | It may happen if there is no supervision. There are no specific reasons for them to congregate. | No | No | No | No |
| J van Breda | It may happen if there is no supervision. There are no specific reasons for them to congregate. | No, we accept that the 0.5 - 0.3 kN/m2 live roof load prescribed by the SABS is sufficient to allow for the effects of workmen and stacked materials. As long as there is some supervision and the site foremans are not negligent. | No | No | No |
| C Eksteen | It may happen if there is no supervision. There are no specific reasons for them to congregate. | No | No | No | No |

| | | | | | |
|---|---|---|---|---|---|
| I Gillmore | It may happen if there is no supervision. There are no specific reasons for them to congregate. | No | No | No | No |
| D Scott | It may happen if there is no supervision. There are no specific reasons for them to congregate. | No | No | No | No |
| C Lutzeller | I have seen on occasion where 5-6 workers are standing on one spot on the roof. We do not promote congregation and any structural damage that occurs as a result of this is our responsibility. | No | No | There are no companies with such polacies. But I have come across instances where for example 15 bays' sheeting was stacked on the purlins, between the rafters. Such negligence cannot be accounted for in the design. | No |
| A Kilpin | It may happen if there is no supervision. There are no specific reasons for them to congregate. | No | No | No | No |
| G Lackey | It may happen if there is no supervision. There are no specific reasons for them to congregate. | No | No | No | No |
| M Papanicolau | It may happen if there is no supervision. There are no specific reasons for them to congregate. | No | No | No | No |
| W du Plessis | It may happen if there is no supervision. There are no specific reasons for them to congregate. | No | No | No | No |

## APPENDIX E: EXCAL

The spreadsheet programme EXCAL calculates the relative weights for the individual experts according to the Classical Method and uses the normalised weights to combine their opinions for the Maximum Variables, as explained in Section 4.1.

Refer to the attached diskette for EXCAL. A printout of EXCAL is presented on the following pages.

**EXCAL** (seed variables = lognormally distributed)

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**APPENDIX F: PROKON Analyses to Evaluate Conversion Methodology**

PROKON analyses were carried out to determine the amount of error made by the conversion methodology for Maximum Variable 1: The Maximum Amount of Construction Workers on the Frame.

Equations (27) & (39) in Section 5.1.1.1 & 5.1.3.1 translate the load due to workers on the roof into EUDL's for the moment at column eaves and the maximum moment in the roof element respectively. Specifically, two types of errors can be made (see Section 5.1.1.1):

- The error due to linear interpolation
- The error for alternative configurations of workers on the frame.

## F1. The EUDL for the Moment at Column Eaves

The results of the PROKON analyses for the error due to linear interpolation and the error for alternative configurations are shown in the PROKON output pages F1.1 & F1.2 respectively (turn page). These results are to be verified with those for the $x_1$, $x_2$ and $x_3$ - values shown in Table 27 in Section 5.1.1.1.

## F2. The EUDL for the Maximum Moment in the Roof Element

The results of the PROKON analyses for the error due to linear interpolation and the error for alternative configurations are shown in the PROKON output pages F2.1 & F2.2 respectively (turn pages). These results are to be verified with those for the $x_1$, $x_2$ and $x_3$ - values shown in Table 30 in Section 5.1.3.1. Note that the building geometries and stiffnesses are so chosen that the moment in the roof element is maximised as discussed in Section 5.1.3.

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## APPENDIX F1.1

**X-Moments for Load Case X1** ($x_1$-value of the workers on the roof)

## X-Moments for Load Case X2   (x$_2$-value of the workers on the roof)



## X-Moments for Load Case X3   (x$_3$-value of the workers on the roof)



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| | Job Number | | Sheet |
|---|---|---|---|
| **PROKON** | Job Title | | |
| Software Consultants (Pty) Ltd | Client | | |
| Internet: http://www.prokon.com | | | |
| E-Mail : mail@prokon.com | Calcs by | Checked by | Date |

```
========= S p a c e - F r a m e   A n a l y s i s  -  P R O K O N ===========
                 Ver W1.9.08 - 19 Nov 2002

TITLE : APPENDIX F1.1

Data file : C:\JdeV\Appendix F1.1.A03
Created on: 2/19/03

========================= NODAL POINT COORDINATES ============================

Node no. X-coord     Y-coord     Z-coord  Node no. X-coord     Y-coord     Z-coord
          m           m           m                 m           m           m

1        0.000       0.000       0.000     2        0.000       6.000       0.000
3       10.000       7.760       0.000     4       20.000       6.000       0.000
5       20.000       0.000       0.000     6        9.250       7.631       0.000
7       10.750       7.631       0.000     8        8.500       7.500       0.000
9       11.500       7.500       0.000    10        7.375       7.300       0.000
11      12.625       7.300       0.000

============================ ELEMENT DATA ===================================

Beam         Secn. type       Fixity    Length        ß
                                          m           (°)

1-2          COL                00       6.000       0.00
2-10         RAFTER             00       7.489       0.00
8-10         RAFTER             00       1.143       0.00
6-8          RAFTER             00       0.761       0.00
3-6          RAFTER             00       0.761       0.00
3-7          RAFTER             00       0.761       0.00
7-9          RAFTER             00       0.761       0.00
9-11         RAFTER             00       1.143       0.00
4-11         RAFTER             00       7.489       0.00
4-5          COL                00       6.000       0.00


============================ SECTION PROPERTIES ==============================

Section : COL    Section designation: 356x171x45      I1
----------------
   A         Ixx        Iyy        J       Material
  m^2        m^4        m^4        m^4

 5.700E-3   121E-6    8.10E-6    160E-9    Steel:300W

Section : RAFTER Section designation: 356x171x45      I1
----------------
   A         Ixx        Iyy        J       Material
  m^2        m^4        m^4        m^4

 5.700E-3   121E-6    8.10E-6    160E-9    Steel:300W


================================= MATERIALS ==================================

Designation            E        poisson    Density    Exp. coeff.
                      kPa                  kN/m^3
Steel:300W            206.0E6   0.30       77.00       11.70E-6


============================== SUPPORT DATA ==================================

                          Prescribed displacements
Node    Fixity      X           Y           Z        X-Rot       Y-Rot       Z-Rot
                    m           m           m        rad.        rad.        rad.

1       XY          0.00        0.00        0.00      0.00        0.00        0.00
5       XY          0.00        0.00        0.00      0.00        0.00        0.00

                          Spring constants
Node    Fixity      X           Y           Z        X-Rot       Y-Rot       Z-Rot
                   kN/m        kN/m        kN/m      kNm/rad     kNm/rad     kNm/rad


================================= LOADS ======================================
```

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**PROKON**

*Software Consultants (Pty) Ltd*
*Internet: http://www.prokon.com*
*E-Mail : mail@prokon.com*

| Job Number | | Sheet |
|---|---|---|
| Job Title | | |
| Client | | |
| Calcs by | Checked by | Date |

```
Load Case   Description

X1          x₁-value for the workers on the roof
X2          x₂-value for the workers on the roof
X3          x₃-value for the workers on the roof
WE,X1       The EUDL that results in the same eaves moment as X1
WE,X2       The EUDL that results in the same eaves moment as X2
WE,X3       The EUDL that results in the same eaves moment as X3

Own weight not added to any load case/combination

============================= LOAD CASE X1      ================================


*** BEAM ELEMENT LOADS ***

Element    Direction        P           a           Wl          Wr          dT
                            kN           m          kN/m        kN/m         °C


3-6          Y            0.00        0.00        -2.04       -2.04        0.00
3-7          Y            0.00        0.00        -2.04       -2.04        0.00


============================= LOAD CASE X2      ================================


*** BEAM ELEMENT LOADS ***

Element    Direction        P           a           Wl          Wr          dT
                            kN           m          kN/m        kN/m         °C


6-8          Y            0.00        0.00        -1.56       -1.56        0.00
3-6          Y            0.00        0.00        -1.56       -1.56        0.00
3-7          Y            0.00        0.00        -1.56       -1.56        0.00
7-9          Y            0.00        0.00        -1.56       -1.56        0.00


============================= LOAD CASE X3      ================================


*** BEAM ELEMENT LOADS ***

Element    Direction        P           a           Wl          Wr          dT
                            kN           m          kN/m        kN/m         °C


8-10         Y            0.00        0.00        -1.37       -1.37        0.00
6-8          Y            0.00        0.00        -1.37       -1.37        0.00
3-6          Y            0.00        0.00        -1.37       -1.37        0.00
3-7          Y            0.00        0.00        -1.37       -1.37        0.00
7-9          Y            0.00        0.00        -1.37       -1.37        0.00
9-11         Y            0.00        0.00        -1.37       -1.37        0.00


============================= LOAD CASE WE,X1   ================================


*** BEAM ELEMENT LOADS ***

Element    Direction        P           a           Wl          Wr          dT
                            kN           m          kN/m        kN/m         °C


2-10         Y            0.00        0.00        -0.22       -0.22        0.00
8-10         Y            0.00        0.00        -0.22       -0.22        0.00
6-8          Y            0.00        0.00        -0.22       -0.22        0.00
3-6          Y            0.00        0.00        -0.22       -0.22        0.00
3-7          Y            0.00        0.00        -0.22       -0.22        0.00
7-9          Y            0.00        0.00        -0.22       -0.22        0.00
9-11         Y            0.00        0.00        -0.22       -0.22        0.00
4-11         Y            0.00        0.00        -0.22       -0.22        0.00


============================= LOAD CASE WE,X2   ================================


*** BEAM ELEMENT LOADS ***

Element    Direction        P           a           Wl          Wr          dT
                            kN           m          kN/m        kN/m         °C

2-10         Y            0.00        0.00        -0.33       -0.33        0.00
8-10         Y            0.00        0.00        -0.33       -0.33        0.00
```

```
6-8        Y        0.00      0.00     -0.33     -0.33      0.00
3-6        Y        0.00      0.00     -0.33     -0.33      0.00
3-7        Y        0.00      0.00     -0.33     -0.33      0.00
7-9        Y        0.00      0.00     -0.33     -0.33      0.00
9-11       Y        0.00      0.00     -0.33     -0.33      0.00
4-11       Y        0.00      0.00     -0.33     -0.33      0.00


============================= LOAD CASE WE,X3 =================================


*** BEAM ELEMENT LOADS ***

Element     Direction        P          a         Wl        Wr        dT
                            kN          m        kN/m      kN/m       °C

2-10       Y        0.00      0.00     -0.49     -0.49      0.00
8-10       Y        0.00      0.00     -0.49     -0.49      0.00
6-8        Y        0.00      0.00     -0.49     -0.49      0.00
3-6        Y        0.00      0.00     -0.49     -0.49      0.00
3-7        Y        0.00      0.00     -0.49     -0.49      0.00
7-9        Y        0.00      0.00     -0.49     -0.49      0.00
9-11       Y        0.00      0.00     -0.49     -0.49      0.00
4-11       Y        0.00      0.00     -0.49     -0.49      0.00


========================== OUTPUT: LINEAR ANALYSIS ===========================
============ BEAM ELEMENT END FORCES IN LOCAL ELEMENT AXES at ULS ============


Elem      Lcase       Axial    Y-Shear     M-xx      Axial    Y-Shear     M-xx
                       kN         kN        kNm        kN        kN        kNm

1-2       X1         1.53      -1.00       0.00      -1.53      1.00      -6.01
          X2         2.34      -1.52       0.00      -2.34      1.52      -9.13
          X3         3.60      -2.30       0.00      -3.60      2.30     -13.81
          WE,X1      2.22      -0.96       0.00      -2.22      0.96      -5.76
          WE,X2      3.32      -1.44       0.00      -3.32      1.44      -8.62
          WE,X3      4.91      -2.12       0.00      -4.91      2.12     -12.74
2-10      X1         1.25       1.33       6.01      -1.25     -1.33       3.98
          X2         1.90       2.04       9.13      -1.90     -2.04       6.15
          X3         2.89       3.14      13.81      -2.89     -3.14       9.74
          WE,X1      1.33       2.02       5.76      -1.05     -0.41       3.33
          WE,X2      1.99       3.02       8.62      -1.57     -0.61       4.97
          WE,X3      2.94       4.47      12.74      -2.32     -0.90       7.35
8-10      X1         1.25      -1.33      -5.50      -1.25      1.33       3.98
          X2         1.91      -2.04      -8.48      -1.91      2.04       6.15
          X3         2.63      -1.62     -12.46      -2.90      3.14       9.74
          WE,X1      1.00      -0.16      -3.65      -1.05      0.41       3.33
          WE,X2      1.50      -0.24      -5.46      -1.57      0.61       4.97
          WE,X3      2.22      -0.35      -8.07      -2.32      0.90       7.35
6-8       X1         1.25      -1.33      -6.51      -1.25      1.33       5.50
          X2         1.70      -0.89      -9.60      -1.90      2.04       8.48
          X3         2.44      -0.62     -13.32      -2.62      1.63      12.46
          WE,X1      0.97       0.00      -3.71      -1.00      0.16       3.65
          WE,X2      1.46       0.00      -5.55      -1.50      0.24       5.46
          WE,X3      2.16       0.00      -8.21      -2.22      0.36       8.07
3-6       X1         0.99       0.17      -6.96      -1.25      1.34       6.51
          X2         1.50       0.26      -9.84      -1.70      0.90       9.60
          X3         2.27       0.39     -13.41      -2.44      0.62      13.32
          WE,X1      0.95       0.16      -3.65      -0.97      0.00       3.71
          WE,X2      1.42       0.24      -5.46      -1.46      0.00       5.55
          WE,X3      2.09       0.36      -8.07      -2.16      0.00       8.21
3-7       X1         0.99       0.17      -6.96      -1.25      1.34       6.51
          X2         1.50       0.26      -9.84      -1.70      0.90       9.60
          X3         2.27       0.39     -13.41      -2.44      0.62      13.32
          WE,X1      0.95       0.16      -3.65      -0.97      0.00       3.71
          WE,X2      1.42       0.24      -5.46      -1.46      0.00       5.55
          WE,X3      2.09       0.36      -8.07      -2.16      0.00       8.21
7-9       X1         1.25      -1.33      -6.51      -1.25      1.33       5.50
          X2         1.70      -0.89      -9.60      -1.90      2.04       8.48
          X3         2.44      -0.62     -13.32      -2.62      1.63      12.46
          WE,X1      0.97       0.00      -3.71      -1.00      0.16       3.65
          WE,X2      1.46       0.00      -5.55      -1.50      0.24       5.46
          WE,X3      2.16       0.00      -8.21      -2.22      0.36       8.07
9-11      X1         1.25      -1.33      -5.50      -1.25      1.33       3.98
          X2         1.91      -2.04      -8.48      -1.91      2.04       6.15
          X3         2.63      -1.62     -12.46      -2.90      3.14       9.74
          WE,X1      1.00      -0.16      -3.65      -1.05      0.41       3.33
          WE,X2      1.50      -0.24      -5.46      -1.57      0.61       4.97
          WE,X3      2.22      -0.35      -8.07      -2.32      0.90       7.35
```

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 4-11 | X1 | 1.25 | 1.33 | 6.01 | -1.25 | -1.33 | 3.98 |
| | X2 | 1.90 | 2.04 | 9.13 | -1.90 | -2.04 | 6.15 |
| | X3 | 2.89 | 3.14 | 13.81 | -2.89 | -3.14 | 9.74 |
| | WE,X1 | 1.33 | 2.02 | 5.76 | -1.05 | -0.41 | 3.33 |
| | WE,X2 | 1.99 | 3.02 | 8.62 | -1.57 | -0.61 | 4.97 |
| | WE,X3 | 2.94 | 4.47 | 12.74 | -2.32 | -0.90 | 7.35 |
| 4-5 | X1 | 1.53 | 1.00 | 6.01 | -1.53 | -1.00 | 0.00 |
| | X2 | 2.34 | 1.52 | 9.13 | -2.34 | -1.52 | 0.00 |
| | X3 | 3.60 | 2.30 | 13.81 | -3.60 | -2.30 | 0.00 |
| | WE,X1 | 2.22 | 0.96 | 5.76 | -2.22 | -0.96 | 0.00 |
| | WE,X2 | 3.32 | 1.44 | 8.62 | -3.32 | -1.44 | 0.00 |
| | WE,X3 | 4.91 | 2.12 | 12.74 | -4.91 | -2.12 | 0.00 |

============================== STATISTICAL DATA ==============================

Own weight of structure = 0.00 kN

No. of real numbers in Stiffness matrix = 591   (4728 bytes)
Time used to analyse = 0: 0:0.010 seconds
Total number of : Nodes             = 11
                  Beam Elements      = 10
                  Shell Elements     = 0
                  Supports           = 2
                  Section properties = 2
                  Load cases         = 6
                  Load combinations  = 0

============================== END OF OUTPUT ==============================

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**APPENDIX F1.2**

**X-Moments for Load Case X3,ALT**     (x3-value for alternative configuration of workers on the roof)



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## X-Moments for Load Case X1,ALT   (x1-value for alternative configuration of workers on the roof)
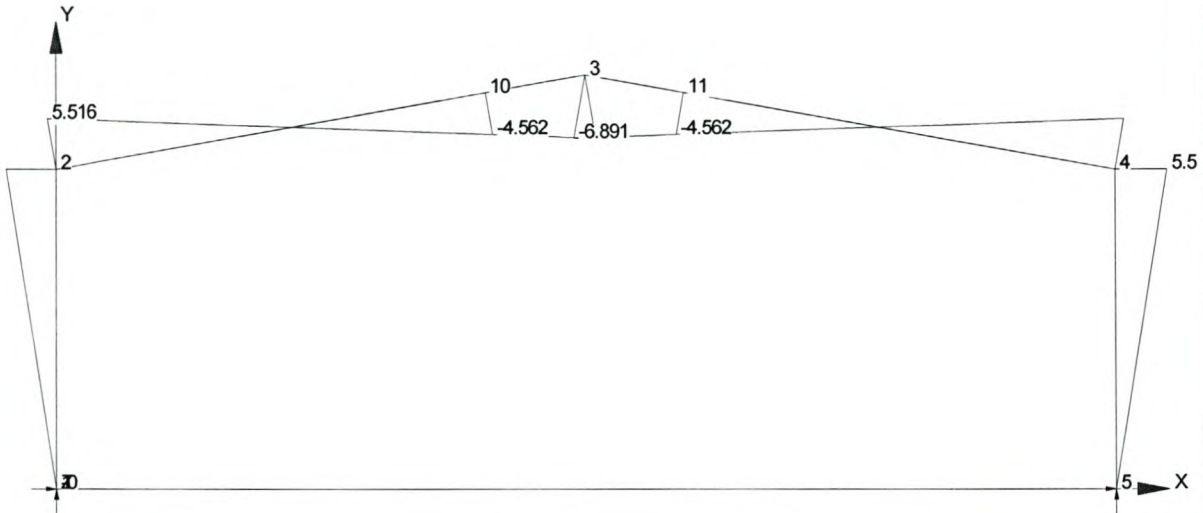


```
========== S p a c e - F r a m e   A n a l y s i s  -  P R O K O N  ==========
                  Ver W1.9.08 - 19 Nov 2002

TITLE : Appendix F1.2

Data file : C:\JdeV\Appendix F1.2.A03
Created on: 2/19/03

========================== NODAL POINT COORDINATES =============================

Node no. X-coord    Y-coord    Z-coord  Node no. X-coord    Y-coord    Z-coord
          m          m          m                 m          m          m

1        0.000      0.000      0.000    2         0.000      6.000      0.000
3       10.000      7.760      0.000    4        20.000      6.000      0.000
5       20.000      0.000      0.000    10        8.125      7.433      0.000
11      11.875      7.433      0.000

=============================== ELEMENT DATA ==================================

Beam       Secn. type         Fixity   Length      ß
                                          m        (°)

1-2        COL                  00      6.000      0.00
2-10       RAFTER               00      8.250      0.00
3-10       RAFTER               00      1.903      0.00
3-11       RAFTER               00      1.903      0.00
4-11       RAFTER               00      8.250      0.00
4-5        COL                  00      6.000      0.00

=========================== SECTION PROPERTIES ================================

Section : COL    Section designation: 356x171x45    I1
----------------
    A      Ixx       Iyy       J       Material
   m^2     m^4       m^4      m^4
```

```
  5.700E-3   121E-6  8.10E-6   160E-9   Steel:300W

Section : RAFTER Section designation: 356x171x45      I1
----------------
    A         Ixx      Iyy       J      Material
   m^2        m^4      m^4      m^4

  5.700E-3   121E-6  8.10E-6   160E-9   Steel:300W


=============================== MATERIALS ===============================

Designation              E      poisson   Density    Exp. coeff.
                        kPa               kN/m^3
Steel:300W           206.0E6    0.30      77.00      11.70E-6


============================= SUPPORT DATA =============================

                          Prescribed displacements
Node    Fixity      X         Y         Z       X-Rot     Y-Rot     Z-Rot
                    m         m         m       rad.      rad.      rad.

1       XY        0.00      0.00      0.00      0.00      0.00      0.00
5       XY        0.00      0.00      0.00      0.00      0.00      0.00

                            Spring constants
Node    Fixity      X         Y         Z       X-Rot     Y-Rot     Z-Rot
                   kN/m      kN/m      kN/m    kNm/rad   kNm/rad   kNm/rad




================================ LOADS ================================

Load Case   Description

X3,ALT
X1,ALT

Own weight not added to any load case/combination

============================ LOAD CASE X3,ALT ============================


*** BEAM ELEMENT LOADS ***

Element     Direction       P         a        Wl        Wr        dT
                           kN         m       kN/m      kN/m       °C

3-10           Y         0.00      0.00     -1.44     -1.44      0.00
3-11           Y        -1.58      0.00     -1.44     -1.44      0.00

============================ LOAD CASE X1,ALT ============================


*** BEAM ELEMENT LOADS ***

Element     Direction       P         a        Wl        Wr        dT
                           kN         m       kN/m      kN/m       °C

3-11           Y        -2.81      0.00      0.00      0.00      0.00

========================== OUTPUT: LINEAR ANALYSIS ==========================
============ BEAM ELEMENT END FORCES IN LOCAL ELEMENT AXES at ULS ==============

Elem      Lcase      Axial    Y-Shear    M-xx      Axial    Y-Shear    M-xx
                      kN        kN       kNm        kN        kN       kNm

1-2       X3,ALT     3.49     -2.26      0.00     -3.49      2.26     -13.58
          X1,ALT     1.40     -0.92      0.00     -1.40      0.92      -5.52
2-10      X3,ALT     2.84      3.04     13.58     -2.84     -3.04      11.51
          X1,ALT     1.15      1.22      5.52     -1.15     -1.22       4.56
3-10      X3,ALT     2.37     -0.39    -14.78     -2.83      3.05      11.51
          X1,ALT     1.15     -1.22     -6.89     -1.15      1.22       4.56
3-11      X3,ALT     2.09      1.16    -14.78     -2.83      3.05      11.51
          X1,ALT     0.66      1.54     -6.89     -1.15      1.22       4.56
4-11      X3,ALT     2.84      3.04     13.58     -2.84     -3.04      11.51
          X1,ALT     1.15      1.22      5.52     -1.15     -1.22       4.56
```

| Job Number | | | | | Sheet | |
|---|---|---|---|---|---|---|
| Job Title | | | | | | |
| Client | | | | | | |
| Calcs by | | | Checked by | | Date | |

| 4-5 | X3,ALT | 3.49 | 2.26 | 13.58 | -3.49 | -2.26 | 0.00 |
|---|---|---|---|---|---|---|---|
| | X1,ALT | 1.40 | 0.92 | 5.52 | -1.40 | -0.92 | 0.00 |

============================== STATISTICAL DATA ===============================

Own weight of structure = 0.00 kN

No. of real numbers in Stiffness matrix = 363   (2904 bytes)
Time used to analyse = 0: 0:0.000 seconds
Total number of : Nodes             = 7
                  Beam Elements      = 6
                  Shell Elements     = 0
                  Supports           = 2
                  Section properties = 2
                  Load cases         = 2
                  Load combinations  = 0

=============================== END OF OUTPUT ==================================

## APPENDIX F2.1

## X-Moments for Load Case X1 ($x_1$-value of the workers on the roof)
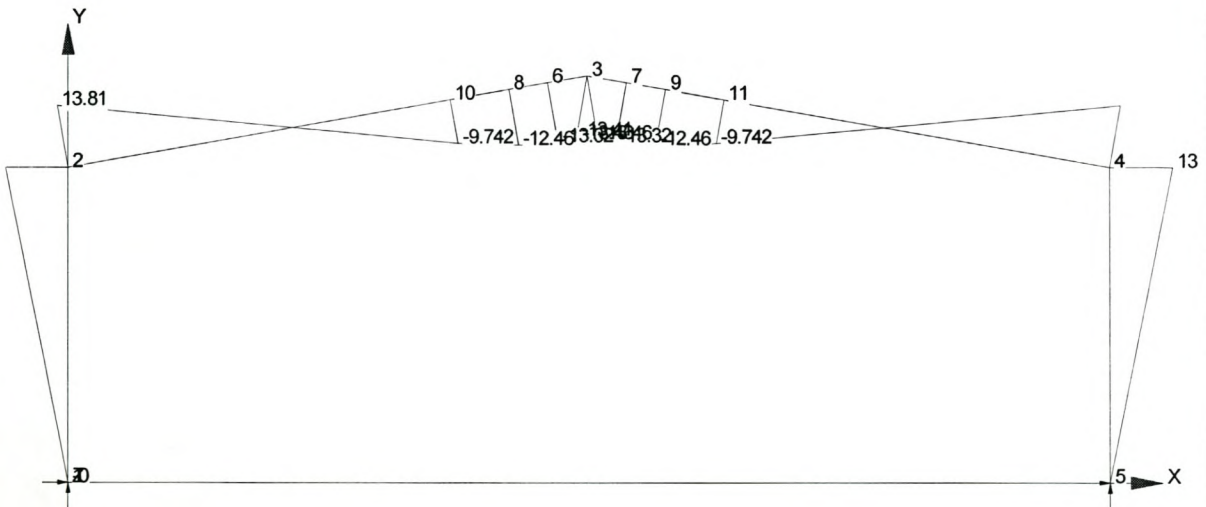
## X-Moments for Load Case X2 ($x_2$-value of the workers on the roof)



## X-Moments for Load Case X3 ($x_3$-value of the workers on the roof)

```
========== S p a c e - F r a m e   A n a l y s i s  -  P R O K O N  ===========
                    Ver W1.9.08 - 19 Nov 2002

TITLE : APPENDIX F2.1

Data file : C:\JdeV\Appendix F2.1.A03
Created on: 2/19/03

========================== NODAL POINT COORDINATES ============================

Node no. X-coord    Y-coord    Z-coord   Node no. X-coord    Y-coord    Z-coord
         m          m          m                  m          m          m

1         0.000      0.000      0.000     2         0.000      5.500      0.000
3        10.000      7.260      0.000     4        20.000      5.500      0.000
5        20.000      0.000      0.000     6         9.250      7.131      0.000
7        10.750      7.131      0.000     8         8.500      7.000      0.000
9        11.500      7.000      0.000     10        7.375      6.800      0.000
11       12.625      6.800      0.000

============================== ELEMENT DATA ===================================

Beam          Secn. type         Fixity    Length       ß
                                            m           (°)


1-2           COL                  00       5.500       0.00
2-10          RAFTER               00       7.489       0.00
8-10          RAFTER               00       1.143       0.00
6-8           RAFTER               00       0.761       0.00
3-6           RAFTER               00       0.761       0.00
3-7           RAFTER               00       0.761       0.00
7-9           RAFTER               00       0.761       0.00
9-11          RAFTER               00       1.143       0.00
4-11          RAFTER               00       7.489       0.00
4-5           COL                  00       5.500       0.00


============================ SECTION PROPERTIES ===============================

Section : COL    Section designation: 356x171x45      I1
----------------
    A        Ixx       Iyy       J       Material
   m^2       m^4       m^4       m^4

 5.700E-3   121E-6   8.10E-6   160E-9   Steel:300W

Section : RAFTER Section designation: 356x171x45      I1
----------------
    A        Ixx       Iyy       J       Material
   m^2       m^4       m^4       m^4

 5.700E-3   121E-6   8.10E-6   160E-9   Steel:300W


================================= MATERIALS ===================================

Designation            E        poisson    Density    Exp. coeff.
                      kPa                   kN/m^3
Steel:300W          206.0E6     0.30        77.00      11.70E-6


============================== SUPPORT DATA ===================================

                          Prescribed displacements
Node    Fixity      X          Y          Z         X-Rot      Y-Rot      Z-Rot
                    m          m          m         rad.       rad.       rad.

1       XY          0.00       0.00       0.00      0.00       0.00       0.00
5       XY          0.00       0.00       0.00      0.00       0.00       0.00

                            Spring constants
Node    Fixity      X          Y          Z         X-Rot      Y-Rot      Z-Rot
                    kN/m       kN/m       kN/m      kNm/rad    kNm/rad    kNm/rad


================================= LOADS =======================================
```

```
Load Case  Description

X1          x₁-value of the workers on the roof
X2          x₂-value of the workers on the roof
X3          x₃-value of the workers on the roof
WR,X1       EUDL that results in the same ridge moment as for X1
WR,X2       EUDL that results in the same ridge moment as for X2
WR,X3       EUDL that results in the same ridge moment as for X3

Own weight not added to any load case/combination

============================= LOAD CASE X1    =================================


*** BEAM ELEMENT LOADS ***

Element     Direction      P          a          Wl          Wr          dT
                           kN         m          kN/m        kN/m        °C

3-6         Y              0.00       0.00       -2.04       -2.04       0.00
3-7         Y              0.00       0.00       -2.04       -2.04       0.00

============================= LOAD CASE X2    =================================


*** BEAM ELEMENT LOADS ***

Element     Direction      P          a          Wl          Wr          dT
                           kN         m          kN/m        kN/m        °C

6-8         Y              0.00       0.00       -1.56       -1.56       0.00
3-6         Y              0.00       0.00       -1.56       -1.56       0.00
3-7         Y              0.00       0.00       -1.56       -1.56       0.00
7-9         Y              0.00       0.00       -1.56       -1.56       0.00

============================= LOAD CASE X3    =================================


*** BEAM ELEMENT LOADS ***

Element     Direction      P          a          Wl          Wr          dT
                           kN         m          kN/m        kN/m        °C

8-10        Y              0.00       0.00       -1.37       -1.37       0.00
6-8         Y              0.00       0.00       -1.37       -1.37       0.00
3-6         Y              0.00       0.00       -1.37       -1.37       0.00
3-7         Y              0.00       0.00       -1.37       -1.37       0.00
7-9         Y              0.00       0.00       -1.37       -1.37       0.00
9-11        Y              0.00       0.00       -1.37       -1.37       0.00

============================= LOAD CASE WR,X1 =================================


*** BEAM ELEMENT LOADS ***

Element     Direction      P          a          Wl          Wr          dT
                           kN         m          kN/m        kN/m        °C

2-10        Y              0.00       0.00       -0.45       -0.45       0.00
8-10        Y              0.00       0.00       -0.45       -0.45       0.00
6-8         Y              0.00       0.00       -0.45       -0.45       0.00
3-6         Y              0.00       0.00       -0.45       -0.45       0.00
3-7         Y              0.00       0.00       -0.45       -0.45       0.00
7-9         Y              0.00       0.00       -0.45       -0.45       0.00
9-11        Y              0.00       0.00       -0.45       -0.45       0.00
4-11        Y              0.00       0.00       -0.45       -0.45       0.00

============================= LOAD CASE WR,X2 =================================


*** BEAM ELEMENT LOADS ***

Element     Direction      P          a          Wl          Wr          dT
                           kN         m          kN/m        kN/m        °C

2-10        Y              0.00       0.00       -0.66       -0.66       0.00
8-10        Y              0.00       0.00       -0.66       -0.66       0.00
```

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```
6-8        Y        0.00      0.00     -0.66     -0.66      0.00
3-6        Y        0.00      0.00     -0.66     -0.66      0.00
3-7        Y        0.00      0.00     -0.66     -0.66      0.00
7-9        Y        0.00      0.00     -0.66     -0.66      0.00
9-11       Y        0.00      0.00     -0.66     -0.66      0.00
4-11       Y        0.00      0.00     -0.66     -0.66      0.00


============================ LOAD CASE WR,X3 =================================


*** BEAM ELEMENT LOADS ***

Element    Direction        P          a         Wl        Wr        dT
                            kN          m        kN/m      kN/m       °C

2-10       Y        0.00      0.00     -0.91     -0.91      0.00
8-10       Y        0.00      0.00     -0.91     -0.91      0.00
6-8        Y        0.00      0.00     -0.91     -0.91      0.00
3-6        Y        0.00      0.00     -0.91     -0.91      0.00
3-7        Y        0.00      0.00     -0.91     -0.91      0.00
7-9        Y        0.00      0.00     -0.91     -0.91      0.00
9-11       Y        0.00      0.00     -0.91     -0.91      0.00
4-11       Y        0.00      0.00     -0.91     -0.91      0.00


=========================== OUTPUT: LINEAR ANALYSIS =============================
=========== BEAM ELEMENT END FORCES IN LOCAL ELEMENT AXES at ULS ==============

Elem      Lcase       Axial    Y-Shear    M-xx      Axial    Y-Shear    M-xx
                        kN        kN        kNm        kN        kN        kNm

1-2       X1          1.53     -1.10     -0.00     -1.53      1.10     -6.03
          X2          2.34     -1.67      0.00     -2.34      1.67     -9.17
          X3          3.60     -2.52     -0.00     -3.60      2.52    -13.87
          WR,X1       4.54     -2.15     -0.00     -4.54      2.15    -11.82
          WR,X2       6.59     -3.12      0.00     -6.59      3.12    -17.16
          WR,X3       9.14     -4.33     -0.00     -9.14      4.33    -23.81
2-10      X1          1.35      1.32      6.03     -1.35     -1.32      3.83
          X2          2.05      2.02      9.17     -2.05     -2.02      5.92
          X3          3.11      3.11     13.87     -3.11     -3.11      9.40
          WR,X1       2.91      4.10     11.82     -2.32     -0.80      6.52
          WR,X2       4.22      5.95     17.16     -3.37     -1.16      9.46
          WR,X3       5.85      8.25     23.81     -4.68     -1.61     13.12
8-10      X1          1.35     -1.31     -5.33     -1.35      1.31      3.83
          X2          2.05     -2.01     -8.22     -2.05      2.01      5.92
          X3          2.84     -1.58    -12.07     -3.11      3.10      9.40
          WR,X1       2.24     -0.29     -7.14     -2.33      0.80      6.52
          WR,X2       3.25     -0.43    -10.36     -3.38      1.16      9.46
          WR,X3       4.50     -0.59    -14.37     -4.68      1.60     13.12
6-8       X1          1.34     -1.32     -6.33     -1.34      1.32      5.33
          X2          1.84     -0.87     -9.32     -2.04      2.02      8.22
          X3          2.66     -0.58    -12.90     -2.84      1.59     12.07
          WR,X1       2.18      0.03     -7.24     -2.24      0.30      7.14
          WR,X2       3.16      0.05    -10.51     -3.24      0.44     10.36
          WR,X3       4.38      0.07    -14.58     -4.50      0.61     14.37
3-6       X1          1.08      0.19     -6.76     -1.34      1.32      6.33
          X2          1.64      0.28     -9.54     -1.84      0.87      9.32
          X3          2.48      0.43    -12.96     -2.66      0.59     12.90
          WR,X1       2.12      0.36     -7.09     -2.18     -0.03      7.24
          WR,X2       3.08      0.53    -10.29     -3.16     -0.04     10.51
          WR,X3       4.27      0.73    -14.28     -4.38     -0.06     14.58
3-7       X1          1.08      0.19     -6.76     -1.34      1.32      6.33
          X2          1.64      0.28     -9.54     -1.84      0.87      9.32
          X3          2.48      0.43    -12.96     -2.66      0.59     12.90
          WR,X1       2.12      0.36     -7.09     -2.18     -0.03      7.24
          WR,X2       3.08      0.53    -10.29     -3.16     -0.04     10.51
          WR,X3       4.27      0.73    -14.28     -4.38     -0.06     14.58
7-9       X1          1.34     -1.32     -6.33     -1.34      1.32      5.33
          X2          1.84     -0.87     -9.32     -2.04      2.02      8.22
          X3          2.66     -0.58    -12.90     -2.84      1.59     12.07
          WR,X1       2.18      0.03     -7.24     -2.24      0.30      7.14
          WR,X2       3.16      0.05    -10.51     -3.24      0.44     10.36
          WR,X3       4.38      0.07    -14.58     -4.50      0.61     14.37
9-11      X1          1.35     -1.31     -5.33     -1.35      1.31      3.83
          X2          2.05     -2.01     -8.22     -2.05      2.01      5.92
          X3          2.84     -1.58    -12.07     -3.11      3.10      9.40
          WR,X1       2.24     -0.29     -7.14     -2.33      0.80      6.52
          WR,X2       3.25     -0.43    -10.36     -3.38      1.16      9.46
          WR,X3       4.50     -0.59    -14.37     -4.68      1.60     13.12
```

```
4-11      X1       1.35      1.32      6.03     -1.35     -1.32      3.83
          X2       2.05      2.02      9.17     -2.05     -2.02      5.92
          X3       3.11      3.11     13.87     -3.11     -3.11      9.40
          WR,X1    2.91      4.10     11.82     -2.32     -0.80      6.52
          WR,X2    4.22      5.95     17.16     -3.37     -1.16      9.46
          WR,X3    5.85      8.25     23.81     -4.68     -1.61     13.12
4-5       X1       1.53      1.10      6.03     -1.53     -1.10     -0.00
          X2       2.34      1.67      9.17     -2.34     -1.67      0.00
          X3       3.60      2.52     13.87     -3.60     -2.52     -0.00
          WR,X1    4.54      2.15     11.82     -4.54     -2.15     -0.00
          WR,X2    6.59      3.12     17.16     -6.59     -3.12      0.00
          WR,X3    9.14      4.33     23.81     -9.14     -4.33     -0.00


=============================== STATISTICAL DATA ===============================

Own weight of structure = 0.00 kN

No. of real numbers in Stiffness matrix = 591   (4728 bytes)
Time used to analyse = 0: 0:0.010 seconds
Total number of : Nodes            = 11
                  Beam Elements    = 10
                  Shell Elements   = 0
                  Supports         = 2
                  Section properties = 2
                  Load cases       = 6
                  Load combinations = 0


=============================== END OF OUTPUT ===============================
```
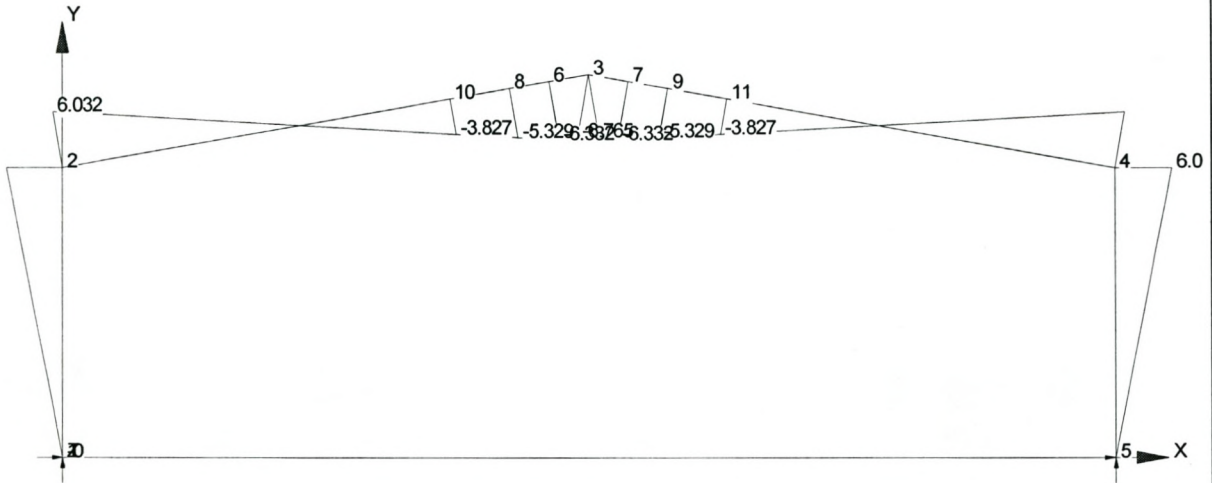
| Job Number | | Sheet |
| --- | --- | --- |
| Job Title | | |
| Client | | |
| Calcs by | Checked by | Date |

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## APPENDIX F2.2

**X-Moments for Load Case X3,ALT**   ($x_3$-value for alternative configuration of workers on the roof)



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| Job Number | | Sheet |
|---|---|---|
| Job Title | | |
| Client | | |
| Calcs by | Checked by | Date |

# X-Moments for Load Case X1,ALT   (x₂-value for alternative configuration of workers on the roof)
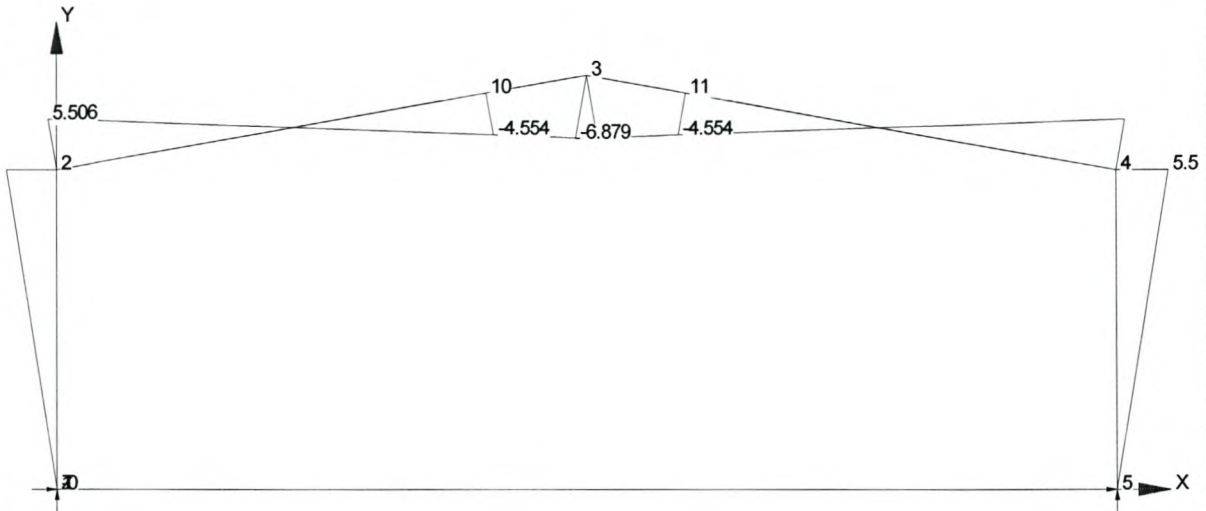


```
========== S p a c e - F r a m e   A n a l y s i s  -  P R O K O N ==========
                 Ver W1.9.08 - 19 Nov 2002

TITLE : APPENDIX F2.2

Data file : C:\JdeV\Appendix F2.2.A03
Created on: 2/19/03


========================== NODAL POINT COORDINATES ==========================

Node no. X-coord     Y-coord     Z-coord  Node no. X-coord     Y-coord     Z-coord
         m           m           m                 m           m           m

1        0.000       0.000       0.000   2         0.000       6.000       0.000
3        10.000      7.760       0.000   4         20.000      6.000       0.000
5        20.000      0.000       0.000   10        8.125       7.433       0.000
11       11.875      7.433       0.000


============================== ELEMENT DATA ==============================

Beam       Secn. type        Fixity    Length      ß
                                         m         (°)

1-2        COL               00        6.000       0.00
2-10       RAFTER            00        8.250       0.00
3-10       RAFTER            00        1.903       0.00
3-11       RAFTER            00        1.903       0.00
4-11       RAFTER            00        8.250       0.00
4-5        COL               00        6.000       0.00


========================== SECTION PROPERTIES ==========================

Section : COL   Section designation: 356x171x45      I1
----------------
   A        Ixx       Iyy       J      Material
  m^2      m^4       m^4       m^4
```

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```
 5.700E-3  121E-6  8.10E-6  160E-9   Steel:300W

Section : RAFTER Section designation: 356x171x45    I1
----------------
    A        Ixx       Iyy       J      Material
   m^2      m^4       m^4      m^4

 5.700E-3  121E-6  8.10E-6  160E-9   Steel:300W


================================== MATERIALS ==================================

Designation              E      poisson   Density   Exp. coeff.
                        kPa              kN/m^3
Steel:300W           206.0E6    0.30      77.00     11.70E-6


============================= SUPPORT DATA ====================================

                          Prescribed displacements
Node    Fixity      X         Y         Z       X-Rot     Y-Rot     Z-Rot
                    m         m         m       rad.      rad.      rad.

1       XY        0.00      0.00      0.00      0.00      0.00      0.00
5       XY        0.00      0.00      0.00      0.00      0.00      0.00

                           Spring constants
Node    Fixity      X         Y         Z       X-Rot     Y-Rot     Z-Rot
                   kN/m      kN/m      kN/m    kNm/rad   kNm/rad   kNm/rad



================================== LOADS ======================================

Load Case   Description

X3,ALT
X1,ALT

Own weight not added to any load case/combination

============================= LOAD CASE X3,ALT ================================


*** BEAM ELEMENT LOADS ***

Element      Direction      P         a         Wl        Wr        dT
                            kN        m        kN/m      kN/m       °C

3-10           Y         0.00      0.00      -1.40     -1.40      0.00
3-11           Y        -1.57      0.00      -1.40     -1.40      0.00

============================= LOAD CASE X1,ALT ================================


*** BEAM ELEMENT LOADS ***

Element      Direction      P         a         Wl        Wr        dT
                            kN        m        kN/m      kN/m       °C

3-11           Y        -2.80      0.00      0.00      0.00       0.00

=========================== OUTPUT: LINEAR ANALYSIS ===========================
=========== BEAM ELEMENT END FORCES IN LOCAL ELEMENT AXES at ULS ==============

Elem      Lcase      Axial    Y-Shear     M-xx      Axial    Y-Shear     M-xx
                      kN        kN        kNm        kN        kN        kNm

1-2       X3,ALT     3.41      -2.21      0.00      -3.41      2.21     -13.28
          X1,ALT     1.40      -0.92      0.00      -1.40      0.92      -5.51
2-10      X3,ALT     2.77       2.97     13.28      -2.77     -2.97      11.25
          X1,ALT     1.15       1.22      5.51      -1.15     -1.22       4.55
3-10      X3,ALT     2.32      -0.39    -14.46      -2.77      2.98      11.25
          X1,ALT     1.14      -1.22     -6.88      -1.14      1.22       4.55
3-11      X3,ALT     2.05       1.15    -14.46      -2.77      2.98      11.25
          X1,ALT     0.66       1.54     -6.88      -1.14      1.22       4.55
4-11      X3,ALT     2.77       2.97     13.28      -2.77     -2.97      11.25
          X1,ALT     1.15       1.22      5.51      -1.15     -1.22       4.55
```

```
4-5        X3,ALT        3.41     2.21    13.28     -3.41     -2.21     0.00
           X1,ALT        1.40     0.92     5.51     -1.40     -0.92     0.00

============================= STATISTICAL DATA ================================

Own weight of structure = 0.00 kN

No. of real numbers in Stiffness matrix = 363   (2904 bytes)
Time used to analyse = 0: 0:0.000 seconds
Total number of : Nodes               = 7
                  Beam Elements       = 6
                  Shell Elements      = 0
                  Supports            = 2
                  Section properties = 2
                  Load cases          = 2
                  Load combinations  = 0

============================= END OF OUTPUT ==================================
```

## APPENDIX G: PROBMOD

The spreadsheet programme PROBMOD calculates the first two probability moments for the Maximum and Average Variables, thereby defining the probabilistic models for them (see Section 5).

Refer to the attached diskette for PROBMOD. A printout of PROBMOD is presented on the following two pages.

## PROBMOD

| | Max1 | | | Max2 | | | Max3 | | | Max4 | | | Max5 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| x1 (#/m) | x2 (#/m) | x3 (#/m) | x1 (#) | x2 (#) | x3 (#) | x1 (#) | x2 (#) | x3 (#) | x1 (#/m) | x2 (#/m) | x3 (#/m) | x1 (#) | x2 (#) | x3 (#) |
| 0.173141 | 0.2648 | 0.399963 | 1.633997 | 2.2101866 | 3.17033181 | 1.257406021 | 2.214837562 | 3.36644377 | 0.081413 | 0.125016 | 0.163566 | 1.226582 | 1.468708 | 1.710834 |

### MAXIMUM VARIABLE 1

| | | | | Moment at column eaves | | | | | Max moment on roof beam / truss | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | #/m | kN/m | Prokon | kN/m$^2$ (5m) | kN/m$^2$ (4m) | | | | kN/m | Prokon | kN/m$^2$ (5m) | kN/m$^2$ (4m) |
| x1 | 0.173 | 0.226 | 0.236 | 0.047 | 0.059 | | | | 0.454 | 0.454 | 0.091 | 0.114 |
| x2 | 0.265 | 0.338 | 0.358 | 0.072 | 0.089 | | | | 0.659 | 0.659 | 0.132 | 0.165 |
| x3 | 0.400 | 0.491 | 0.532 | 0.106 | 0.133 | | | | 0.914 | 0.914 | 0.183 | 0.229 |

| | 1st 2 mom (5m spacing) | | | 4m spacing | | | 1st 2 mom (5m spacing) | | | 4m spacing | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | alpha | std dev | avg | std dev | avg | | alpha | std dev | avg | std dev | avg |
| x2-x3 | 68.522 | 0.019 | 0.072 | 0.023 | 0.089 | x2-x3 | 46.789 | 0.027 | 0.132 | 0.034 | 0.165 |
| x1-x3 | 68.686 | 0.019 | 0.072 | 0.023 | 0.090 | x1-x3 | 44.191 | 0.029 | 0.129 | 0.036 | 0.161 |

| | 1st 2 mom using equation (5m) | | | 4m spacing | | | 1st 2 mom using equation (5m) | | | 4m spacing | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | std dev | avg | std dev | avg | | | std dev | avg | std dev | avg |
| | | 0.020 | 0.0732 | 0.0246 | 0.0915 | | | 0.0303 | 0.132 | 0.038 | 0.164649 |

| M Factor | 3.2 |
|---|---|

| 1st 2 mom l.t.o. #/m | | |
|---|---|---|
| | alpha | std dev |
| x2-x3 | 17.712 | 0.0724 |
| x1-x3 | 17.932 | 0.0715 |

### MAXIMUM VARIABLE 2

| | | Pos Mom (kN/m$^2$) | | Neg Mom (kN/m$^2$) | |
|---|---|---|---|---|---|
| | # | L=5, s=1.7m | L=4, s=1.4m | L=5, s=1.7m | L=4, s=1.4m |
| x1 | 1.634 | 0.294119 | 0.426736 | 0.2596301 | 0.38832818 |
| x2 | 2.2102 | 0.397834 | 0.577214 | 0.3511823 | 0.52526268 |
| x3 | 3.1703 | 0.537092 | 0.764276 | 0.4815495 | 0.7011567 |

| Pos Mom (kN/m$^2$) | | | | Neg Mom (kN/m$^2$) | | | |
|---|---|---|---|---|---|---|---|
| 1st 2 mom, l = 5m, spas = 1.7m | | | | 1st 2 mom, l = 5m, spas = 1.7m | | | |
| | alpha | std dev | avg | | alpha | std dev | avg |
| x2-x3 | 17.185 | 0.075 | 0.398 | x2-x3 | 18.357 | 0.070 | 0.351 |
| x1-x3 | 16.740 | 0.077 | 0.394 | x1-x3 | 18.328 | 0.070 | 0.351 |
| 1st 2 mom, l = 4m, spas = 1.4m | | | | 1st 2 mom, l = 4m, spas = 1.4m | | | |
| | alpha | std dev | avg | | alpha | std dev | avg |
| x2-x3 | 12.794 | 0.100 | 0.577 | x2-x3 | 13.606 | 0.094 | 0.525 |
| x1-x3 | 12.050 | 0.106 | 0.566 | x1-x3 | 13.002 | 0.099 | 0.517 |

### MAXIMUM VARIABLE 3

| | max. stacking + steel sheets | | | avg. stacking + fibre-cement | | | | |
|---|---|---|---|---|---|---|---|---|
| | # - 1 | N/m$^2$ 10kg | | | # - 1 | kN/m2 15kg | distribution | Average (N) Std. Dev (N) |
| x1 | 0.257 | 0.026 | | Avg | 0.357142857 | 0.053571429 | Lognormal | -3.6352183 1.1903604 |
| x2 | 1.215 | 0.121 | | st dev | 0.623329793 | 0.094695041 | 95%-value | 0.186887 |
| x3 | 2.366 | 0.237 | | | | | | |

| | 1st 2 mom | | | | | |
|---|---|---|---|---|---|---|
| | alpha | std dev | avg | 0.9508464 | 95%-value | 0.239 |
| x2-x3 | 20.781 | 0.063 | 0.121 | alpha = | 20.5138357 | |
| x1-x3 | 19.285 | 0.067 | 0.113 | u = | 0.0933564 | |

### MAXIMUM VARIABLE 4

| | | | | Moment at column eaves | | | | | Max moment on roof beam / truss | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | #/m | kN/m | Prokon | kN/m$^2$ (5m) | kN/m$^2$ (4m) | | | | kN/m | Prokon | kN/m$^2$ (5m) | kN/m$^2$ (4m) |
| x1 | 0.081 | 0.109 | 0.109 | 0.022 | 0.027 | | | | 0.225 | 0.225 | 0.045 | 0.056 |
| x2 | 0.125 | 0.166 | 0.169 | 0.034 | 0.042 | | | | 0.337 | 0.337 | 0.067 | 0.084 |
| x3 | 0.164 | 0.214 | 0.224 | 0.045 | 0.056 | | | | 0.431 | 0.431 | 0.086 | 0.108 |

| | 1st 2 mom (5m spacing) | | | 4m spacing | | | 1st 2 mom (5m spacing) | | | 4m spacing | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | alpha | std dev | avg | std dev | avg | | alpha | std dev | avg | std dev | avg |
| x2-x3 | 218.605 | 0.006 | 0.034 | 0.007 | 0.042 | x2-x3 | 126.621 | 0.010 | 0.067 | 0.013 | 0.084 |
| x1-x3 | 177.554 | 0.007 | 0.031 | 0.009 | 0.039 | x1-x3 | 98.382 | 0.013 | 0.062 | 0.016 | 0.077 |

| | 1st 2 mom using equation (5m) | | | 4m spacing | | | 1st 2 mom using equation (5m) | | | 4m spacing | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | std dev | avg | std dev | avg | | | std dev | avg | std dev | avg |
| | | 0.006 | 0.036 | 0.007 | 0.045 | | | 0.010 | 0.067 | 0.013 | 0.084 |

| M Factor | 3.2 |
|---|---|

| mom l.t.o. #/m | | |
|---|---|---|
| | alpha | std dev |
| x2-x3 | 62.081 | 0.0207 |
| x1-x3 | 49.510 | 0.0259 |

| MAXIMUM VARIABLE 5 | | | | |
|---|---|---|---|---|
| | Pos Mom (kN/m$^2$) | | Neg Mom (kN/m$^2$) | |
| # | L=5, s=1.7m | =4, s=1.4 | L=5, s=1.7m | L=4, s=1.4m |
| x1  1.2266 | 0.260 | 0.394 | 0.199 | 0.303 |
| x2  1.4687 | 0.264 | 0.384 | 0.233 | 0.349 |
| x3  1.7108 | 0.308 | 0.447 | 0.272 | 0.407 |

| Pos Mom (kN/m$^2$) | | | | Neg Mom (kN/m$^2$) | | | |
|---|---|---|---|---|---|---|---|
| 1st 2 mom, I = 5m, spas = 1.7m | | | | 1st 2 mom, I = 5m, spas = 1.7m | | | |
| | alpha | std dev | avg | | alpha | std dev | avg |
| x2-x3 | 54.912 | 0.023 | 0.264 | x2-x3 | 62.206 | 0.021 | 0.233 |
| x1-x3 | 84.380 | 0.015 | 0.280 | x1-x3 | 56.216 | 0.023 | 0.229 |
| 1st 2 mom, I = 4m, spas = 1.4m | | | | 1st 2 mom, I = 4m, spas = 1.4m | | | |
| | alpha | std dev | avg | | alpha | std dev | avg |
| x2-x3 | 37.847 | 0.034 | 0.384 | x2-x3 | 41.590 | 0.031 | 0.349 |
| x1-x3 | 77.409 | 0.017 | 0.416 | x1-x3 | 39.185 | 0.033 | 0.346 |

| Average Construction Workers on Frame | | | | | | |
|---|---|---|---|---|---|---|
| 1st Mom obtained from load survey (#/m) = 0.2085897 | | | | distribution | Average (N) | Std. Dev (N) |
| 2nd Mom obtained from load survey (#/m)  0.0629018 | | | | Lognormal | -1.610904628 | 0.295020258 |

| Maximum moment in Roof Element | | | | |
|---|---|---|---|---|
| | #/m | kN/m | Pkon | kN/m2 (5m) | kN/m2 (4m) |
| implied x2 | 0.2086 | 0.536 | 0.536 | 0.1072253 | 0.13403162 |
| implied x3 | 0.3244 | 0.778 | 0.778 | 0.1556167 | 0.19452087 |

| | 5m spacing (kN/m$^2$) | | | 4m spacing (kN/m$^2$) | |
|---|---|---|---|---|---|
| | avg | alpha | std dev | avg | std dev |
| x2-x3 | 0.1072 | 49.455 | 0.0259 | 0.1340316 | 0.032418 |

| Average Construction Workers on Purlin | | | | | | |
|---|---|---|---|---|---|---|
| 1st Mom obtained from load survey (#/m) = 1.6428571 | | | | distribution | Average (N) | Std. Dev (N) |
| 2nd Mom obtained from load survey (#/m)  0.6555523 | | | | Lognormal | 0.42256044 | 0.384386384 |

| Maximum Positive Moment | | | |
|---|---|---|---|
| | # | =5, s=1.7m | =4, s=1.4m |
| implied x2 | 1.6429 | 0.295714 | 0.42905 |
| implied x3 | 2.8715 | 0.486459 | 0.692226 |

| 1st 2 mom, I = 5m, spas = 1.7m | | | |
|---|---|---|---|
| | alpha | std dev | avg |
| x2-x3 | 12.547 | 0.102 | 0.296 |
| 1st 2 mom, I = 4m, spas = 1.4m | | | |
| | alpha | std dev | avg |
| x2-x3 | 9.093 | 0.141 | 0.429 |

| Average Load due to Stacking of Roof Cladding on Frames | | |
|---|---|---|
| | # - 1 | kN/m$^2$ 10kg |
| Avg | 0.3571 | 0.035714 |
| st dev | 0.6233 | 0.06313 |

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**APPENDIX H: PARSTUDY**

The spreadsheet programme PARSTUDY calculates the ratio of the maximum moment in the roof element caused by a concentrated load at the roof ridge to that caused by a uniformly distributed load as explained in Section 5.1.3.

Refer to the attached diskette for PARSTUDY. A printout of PARSTUDY is presented on the following page.

# PARSTUDY

| | | | |
|---|---|---|---|
| roof slope | 0.27 | roof angle (rad) | 0.261799 |
| $k = l_2 / l_1$ | 1 | $D_n$ (N/m$^2$) | 200 |
| | | $L_n$ (N/m$^2$) | 300 |

**L1 =** 0.69

| | | | | | L1 = 1.2 Dn + 1.6 Ln | | | | Point | ratio |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Uniformly Distributed | | | | | |
| v = h/L | 1+d/(2v) | B | N | He / WL | Mneg/L$^2$ | x/L | Mpos/L$^2$ | Mpos/L$^2$ | |
| 0.09 | 2.5 | 4.8 | 19.7 | 0.5 | 0.03 | 0.43 | 0.008 | 0.04 | 5.52 |
| 0.095 | 2.4 | 4.8 | 18.8 | 0.5 | 0.03 | 0.43 | 0.008 | 0.04 | 5.34 |
| 0.1 | 2.3 | 4.7 | 18.0 | 0.5 | 0.04 | 0.43 | 0.009 | 0.05 | 5.17 |
| 0.125 | 2.1 | 4.6 | 15.2 | 0.4 | 0.04 | 0.44 | 0.011 | 0.05 | 4.54 |
| 0.15 | 1.9 | 4.5 | 13.5 | 0.4 | 0.04 | 0.45 | 0.014 | 0.06 | 4.13 |
| 0.175 | 1.8 | 4.4 | 12.4 | 0.3 | 0.04 | 0.45 | 0.016 | 0.06 | 3.85 |
| 0.2 | 1.7 | 4.4 | 11.7 | 0.3 | 0.04 | 0.46 | 0.018 | 0.07 | 3.63 |
| 0.225 | 1.6 | 4.5 | 11.2 | 0.3 | 0.04 | 0.46 | 0.020 | 0.07 | 3.47 |
| 0.25 | 1.5 | 4.5 | 10.8 | 0.2 | 0.04 | 0.47 | 0.022 | 0.07 | 3.34 |
| 0.275 | 1.5 | 4.5 | 10.5 | 0.2 | 0.0430 | 0.47 | 0.0232 | 0.08 | 3.23 |
| 0.3 | 1.4 | 4.6 | 10.2 | 0.2 | 0.04 | 0.47 | 0.025 | 0.08 | 3.14 |
| 0.325 | 1.4 | 4.7 | 10.1 | 0.2 | 0.04 | 0.47 | 0.026 | 0.08 | 3.07 |
| 0.35 | 1.4 | 4.7 | 9.9 | 0.2 | 0.04 | 0.48 | 0.027 | 0.08 | 3.00 |
| 0.375 | 1.4 | 4.8 | 9.8 | 0.2 | 0.04 | 0.48 | 0.029 | 0.08 | 2.94 |
| 0.4 | 1.3 | 4.9 | 9.8 | 0.2 | 0.04 | 0.48 | 0.030 | 0.09 | 2.89 |
| 0.425 | 1.3 | 5.0 | 9.7 | 0.1 | 0.04 | 0.49 | 0.031 | 0.09 | 2.86 |
| 0.45 | 1.3 | 5.0 | 9.7 | 0.1 | 0.04 | 0.48 | 0.032 | 0.09 | 2.81 |
| 0.475 | 1.3 | 5.1 | 9.7 | 0.1 | 0.04 | 0.48 | 0.033 | 0.09 | 2.77 |
| 0.5 | 1.3 | 5.2 | 9.7 | 0.1 | 0.04 | 0.48 | 0.034 | 0.09 | 2.74 |

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**APPENDIX I: COMBAN**

The spreadsheet programme COMBAN determines the probabilistic models for the combination of MW+AS (maximum workers and average over-staking on the frames) and AW+MS (average workers and maximum over-stacking on the frames), as discussed in Section 6.6.

Refer to the attached diskette for COMBAN. A printout of COMBAN is presented on the following page.

## COMBAN

### MW+AS

| MW (extreme type 1) | | AS (lognormal) | |
|---|---|---|---|
| avg | 0.1646488 | avg | 0.03571429 |
| std dev | 0.03426481 | std dev | 0.06313003 |
| **Parameters** | | **Parameters** | |
| alpha | 98.660 | Z | -4.0406834 |
| u | 0.15880041 | gamma | 1.19036039 |
| MW* = | 0.164 | AS* = | 0.136 |
| F(MW*) | 0.550 | | |
| f(MW*) | 32.4591872 | | |
| std dev,N | 0.01219576 | std dev,N | 0.16188901 |
| avg,N | 0.16248213 | avg,N | -0.1421993 |
| alpha*(MW | -0.08 | alpha*(AS)= | -1.00 |
| MW* = | 0.1653 | AS* = | 0.3547 |

| BETA at c | 0.52 | 3.0781 | F(MW+AS) = | 0.99895819 |
|---|---|---|---|---|

### AW+MS

| MS (extreme type 1) | | AW (lognormal) | |
|---|---|---|---|
| avg | 0.12148376 | avg | 0.13403162 |
| std dev | 0.06252246 | std dev | 0.03241776 |
| **Parameters** | | **Parameters** | |
| alpha | 98.660 | Z | -2.0381057 |
| u | 0.11563537 | gamma | 0.23843712 |
| MS* = | 0.126 | AW* = | 0.19 |
| F(MS*) | 0.698 | | |
| f(MS*) | 24.7651155 | | |
| std dev,N | 0.01408367 | std dev,N | 0.04530305 |
| avg,N | 0.11869915 | avg,N | 0.11829885 |
| alpha*(MS | -0.30 | alpha*(AW)= | -0.95 |
| MS* = | 0.1317 | AW* = | 0.2533 |

| BETA at c | 0.385 | 3.1197 | F(AW+MS) = | 0.99909463 |
|---|---|---|---|---|

### Extreme type 1 paper

| c* | F((MW+AS)<c*) | z = -ln(-ln(Fy(y))) |
|---|---|---|
| 0.18 | 0.50 | 0.36 |
| 0.2 | 0.73 | 1.16 |
| 0.25 | 0.91 | 2.36 |
| 0.3 | 0.96 | 3.14 |
| 0.35 | 0.98 | 3.74 |
| 0.4 | 0.99 | 4.23 |
| 0.45 | 0.99 | 4.65 |
| 0.5 | 0.99 | 5.02 |

### Lognormal paper

| c* | F((MW+AS)<c*) | ln(c*) | z (normal) |
|---|---|---|---|
| 0.18 | 0.50 | 2.89 | 0.00 |
| 0.2 | 0.73 | 3.00 | 0.61 |
| 0.25 | 0.91 | 3.22 | 1.34 |
| 0.3 | 0.96 | 3.40 | 1.72 |
| 0.35 | 0.98 | 3.56 | 1.99 |
| 0.4 | 0.99 | 3.69 | 2.18 |
| 0.45 | 0.99 | 3.81 | 2.35 |
| 0.5 | 0.99 | 3.91 | 2.48 |

### Comparing MW+AS & AW+MS

| 1-F | -LOG(1-F) | MW+AS | AW+MS |
|---|---|---|---|
| 0.100 | 1.000 | 0.226 | 0.300 |
| 0.050 | 1.301 | 0.287 | 0.315 |
| 0.010 | 2.000 | 0.400 | 0.348 |
| 0.001 | 3.000 | 0.525 | 0.385 |

| c* | F((MW+AS)<c*) |
|---|---|
| 0.180 | 0.003 |
| 0.200 | 0.037 |
| 0.250 | 0.505 |
| 0.300 | 0.906 |
| 0.350 | 0.990 |
| 0.400 | 0.999 |
| 0.450 | 1.000 |
| 0.500 | 1.000 |

| F (%) | Lc original | Lc generalised |
|---|---|---|
| 50 | 0.18 | 0.155 |
| 60 | 0.187 | 0.173 |
| 70 | 0.196 | 0.195 |
| 80 | 0.21 | 0.224 |
| 90 | 0.246 | 0.272 |
| 95 | 0.288 | 0.318 |
| 97 | 0.335 | 0.353 |
| 99 | 0.435 | 0.428 |



$y = 13.925x - 1.4957$
$R^2 = 0.9415$



$y = 2.2929x - 6.2897$
$R^2 = 0.947$
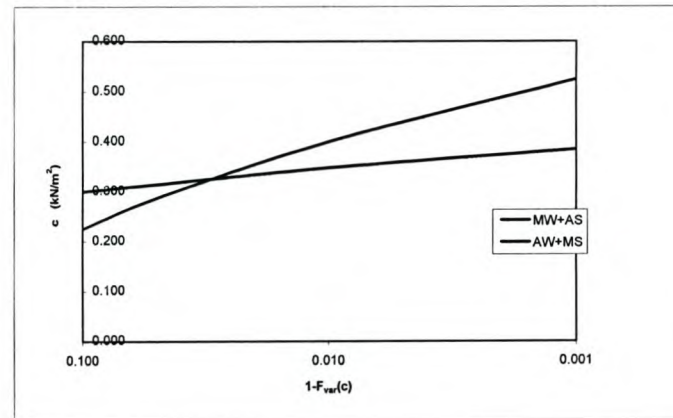
## APPENDIX J: RELAN

The spreadsheet programme RELAN determines the levels of reliability provided for by the SABS 0160-1989 ultimate limit-states design criteria for the various cases considered (see Section 7).

Refer to the attached diskette for RELAN. A printout of RELAN is presented on the following pages.

## RELAN

### MW+AS

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Dcov = | 0.1 | Davg = | 0.2 | | | | |
| Rcov = | 0.15 | | | | | | |

| | D | | MW | | AS | | R |
|---|---|---|---|---|---|---|---|
| avg | 0.2 | avg | 0.164649 | avg | 0.035714 | avg | 1 |
| std dev | 0.02 | std dev | 0.034265 | std dev | 0.06313 | std dev | 0.15 |

| LN parameters | | Extreme Type 1 para. | | LN parameters | | LN parameters | |
|---|---|---|---|---|---|---|---|
| Z | -1.614413 | alpha | 37.43128 | Z | -4.040683 | Z | -0.011125 |
| gamma | 0.099751 | u | 0.149234 | gamma | 1.19036 | gamma | 0.149166 |

| D* = | 0.21 | MW* = | 0.164 | AS* = | 0.54 | R* = | 0.8 |
|---|---|---|---|---|---|---|---|
| | | F(MW*) | 0.562488 | | | | |
| | | f(MW*) | 12.11454 | | | | |
| std dev ,N | 0.020948 | std dev ,N | 0.032526 | std dev ,N | 0.642795 | std dev ,N | 0.119333 |
| avg ,N | 0.198709 | avg,N | 0.158884 | avg ,N | -1.309229 | avg ,N | 0.969615 |
| alpha*(Avg | -0.03 | alpha* | -0.05 | alpha* | -0.98 | alpha* | 0.18 |
| D* | 0.2007 | MW* | 0.1636 | AS* | 0.5415 | R* | 0.9058 |

Beta = 2.933557
p(failure)% 0.167558

| D | R |
|---|---|
| 0.2 | 1.01 |
| 0.35 | 1.17 |
| 0.5 | 1.35 |

### AW+MS

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Dcov = | 0.1 | Davg = | 0.5 | | | | |
| Rcov = | 0.15 | | | | | | |

| | D | | MS | | AW | | R |
|---|---|---|---|---|---|---|---|
| avg | 0.5 | avg | 0.121484 | avg | 0.134032 | avg | 1.33 |
| std dev | 0.05 | std dev | 0.062522 | std dev | 0.032418 | std dev | 0.1995 |

| LN parameters | | Extreme Type 1 para. | | LN parameters | | LN parameters | |
|---|---|---|---|---|---|---|---|
| Z | -0.698122 | alpha | 20.51384 | Z | -2.038106 | Z | 0.274054 |
| gamma | 0.099751 | u | 0.093356 | gamma | 0.238437 | gamma | 0.149166 |

| D* = | 0.54 | MS* = | 0.274 | AW* = | 0.15 | R* = | 0.95 |
|---|---|---|---|---|---|---|---|
| | | F(MS*) | 0.875717 | | | | |
| | | f(MS*) | 0.492049 | | | | |
| std dev ,N | 0.053866 | std dev ,N | 0.115915 | std dev ,N | 0.035766 | std dev ,N | 0.141708 |
| avg ,N | 0.495754 | avg,N | 0.045371 | avg ,N | 0.128852 | avg ,N | 1.25908 |
| alpha*(Avg | -0.28 | alpha* | -0.60 | alpha* | -0.18 | alpha* | 0.73 |
| D* | 0.5411 | MS* | 0.2553 | AW* | 0.1488 | R* | 0.9453 |

Beta = 3.034099
p(failure)% 0.120635

| D | R |
|---|---|
| 0.2 | 0.88 |
| 0.35 | 1.1 |
| 0.5 | 1.33 |

### MW+MS

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Dcov = | 0.1 | Davg = | 0.5 | | | | |
| Rcov = | 0.15 | | | | | | |

| | D | | MW | | MS | | R |
|---|---|---|---|---|---|---|---|
| avg | 0.5 | avg | 0.164649 | avg | 0.121484 | avg | 1.39 |
| std dev | 0.05 | std dev | 0.034265 | std dev | 0.062522 | std dev | 0.2085 |

| LN para... | | Extreme Type 1 para. | | Extreme Type 1 para. | | LN parameters | |
|---|---|---|---|---|---|---|---|
| Z | -0.698122 | alpha | 37.43128 | alpha | 20.51384 | Z | 0.318178 |
| gamma | 0.099751 | u | 0.149234 | u | 0.093356 | gamma | 0.149166 |

| D* = | 0.54 | MW* = | 0.18 | MS* = | 0.24 | R* = | 0.965 |
|---|---|---|---|---|---|---|---|
| | | F(MW*) | 0.728968 | F(MS*) | 0.95182 | | |
| | | f(MW*) | 8.62587 | f(MS*) | 0.964156 | | |
| std dev ,N | 0.053866 | std dev ,N | 0.038405 | std dev ,N | 0.103848 | std dev ,N | 0.143846 |
| avg ,N | 0.495754 | avg,N | 0.156585 | avg,N | 0.067325 | avg ,N | 1.306422 |
| alpha*(Avg | -0.28 | alpha* | -0.20 | alpha* | -0.55 | alpha* | 0.76 |
| D* | 0.5432 | MW* | 0.1807 | MS* | 0.2437 | R* | 0.9676 |

Beta = 3.0976
p(failure)% 0.097554

| D | MW+MS | MW+AS |
|---|---|---|
| 0.2 | 0.94 | 1.01 |
| 0.35 | 1.16 | 1.17 |
| 0.5 | 1.39 | 1.35 |
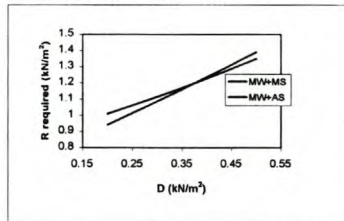


### SABS RELIABILITY WITH L = MW+AS

| | | | | | |
|---|---|---|---|---|---|
| Dcov = | 0.1 | Davg = | 0.5 | D/Dn = | 1.05 |
| Rcov = | 0.15 | | | R/Rn = | 1.05 |

| | D | | MW | | AS | | R |
|---|---|---|---|---|---|---|---|
| avg | 0.5 | avg | 0.164649 | avg | 0.035714 | avg | 1.226667 |
| std dev | 0.05 | std dev | 0.034265 | std dev | 0.06313 | std dev | 0.184 |

| LN parameters | | Extreme Type 1 para. | | LN parameters | | LN parameters | |
|---|---|---|---|---|---|---|---|
| Z | -0.698122 | alpha | 37.43128 | Z | -4.040683 | Z | 0.193175 |
| gamma | 0.099751 | u | 0.149234 | gamma | 1.19036 | gamma | 0.149166 |

| D* = | 0.51 | MW* = | 0.165 | AS* = | 0.39 | R* = | 1.082 |
|---|---|---|---|---|---|---|---|
| | | F(MW*) | 0.574505 | | | | |
| | | f(MW*) | 11.91877 | | | | |
| std dev ,N | 0.050873 | std dev ,N | 0.032886 | std dev ,N | 0.464241 | std dev ,N | 0.158415 |
| avg ,N | 0.497363 | avg,N | 0.158822 | avg ,N | -0.818639 | avg ,N | 1.203269 |
| alpha*(Avg | -0.10 | alpha* | -0.07 | alpha* | -0.94 | alpha* | 0.32 |
| D* | 0.5118 | MW* | 0.1649 | AS* | 0.3863 | R* | 1.0630 |

Beta = 2.763216
p(failure)% 0.28618

| D | original | Pf (%) | eneralised | Pf (%) |
|---|---|---|---|---|
| 0.2 | 2.66 | 0.388 | 2.86 | 0.209 |
| 0.35 | 2.72 | 0.328 | 2.89 | 0.19 |
| 0.5 | 2.76 | 0.286 | 2.87 | 0.2 |
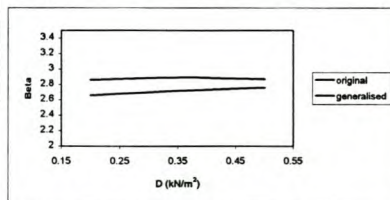


### SABS RELIABILITY WITH GENERALISED MODEL

| | | | | |
|---|---|---|---|---|
| Dlcov = | 0.1 | Davg = | 0.5 | |
| Rcov = | 0.15 | Dn/Davg = | 0.952381 | |
| Rn/Ravg = | 0.952381 | | | |

| Dl (i.t.o kN/m2) | | LL (i.t.o kN/m2) | | R (i.t.o kN/m2) | |
|---|---|---|---|---|---|
| avg | 0.5 | | 0.171 | | 1.226667 |
| std dev | 0.05 | | 0.078 | | 0.184 |

| LN parameters | | LN parameters | | LN parameters | |
|---|---|---|---|---|---|
| Z | -0.698122 | Z | -1.860601 | Z | 0.193175 |
| gamma | 0.099751 | gamma | 0.434763 | gamma | 0.149166 |

| Dl* = | 0.53 | LL* | 0.41 | R* = | 0.94 |
|---|---|---|---|---|---|
| std dev ,N | 0.052868 | std dev ,N | 0.178253 | std dev ,N | 0.140216 |

| | | | | | | |
|---|---|---|---|---|---|---|
| avg ,N | 0.496481 | avg ,N | 0.012709 | avg ,N | 1.179748 | |

| | | | | | |
|---|---|---|---|---|---|
| alpha*(Avg | -0.23 | alpha* | -0.77 | alpha* | 0.60 |
| DL* | 0.5310 | LL* | 0.4056 | R* | 0.9366 |

| | |
|---|---|
| Beta = | 2.879503 |
| p(failure)% | 0.199158 |

## SABS RELIABILITY WITH L = MW (8m frame spacing)+AS

| Dcov = | 0.1 | Davg = | 0.5 | | | D/Dn = | 1.05 |
|---|---|---|---|---|---|---|---|
| Rcov = | 0.15 | | | | | R/Rn = | 1.05 |

| | D | | MW | | AS | | R |
|---|---|---|---|---|---|---|---|
| avg | 0.5 | avg | 0.131719 | avg | 0.035714 | avg | 1.226667 |
| std dev | 0.05 | std dev | 0.027412 | std dev | 0.06313 | std dev | 0.184 |

| LN parameters | | Extreme Type 1 para. | | LN parameters | | LN parameters | |
|---|---|---|---|---|---|---|---|
| Z | -0.698122 | alpha | 46.7891 | Z | -4.040683 | Z | 0.193175 |
| gamma | 0.099751 | u | 0.119387 | gamma | 1.19036 | gamma | 0.149166 |

| D* = | 0.51 | MW* = | 0.131 | AS* = | 0.43 | R* = | 1.07 |
|---|---|---|---|---|---|---|---|

| | | | |
|---|---|---|---|
| | | F(MW*) | 0.559453 |
| | | f(MW*) | 15.20308 |

| std dev ,N | 0.050873 | std dev ,N | 0.025949 | std dev ,N | 0.511855 | std dev ,N | 0.159608 |
|---|---|---|---|---|---|---|---|
| avg ,N | 0.497363 | avg,N | 0.127118 | avg ,N | -0.944587 | avg ,N | 1.204303 |

| alpha*(Avg | -0.09 | alpha* | -0.05 | alpha* | -0.95 | alpha* | 0.30 |
|---|---|---|---|---|---|---|---|
| D* | 0.5109 | MW* | 0.1306 | AS* | 0.4291 | R* | 1.0707 |

| | |
|---|---|
| Beta = | 2.82719 |
| p(failure)% | 0.234799 |

| D | Beta |
|---|---|
| 0.2 | 2.73 |
| 0.35 | 2.78 |
| 0.5 | 2.83 |

## SABS RELIABILITY FOR MAINTENANCE WORKERS FOR LARGE AREAS

| Dlcov = | 0.1 | Davg = | 0.35 | | |
|---|---|---|---|---|---|
| Rcov = | 0.15 | Dn/Davg = | 0.952381 | | |
| Rn/Ravg = | 0.952381 | | | | |

| Dl (i.t.o kN/m2) | | LL (i.t.o kN/m2) | | R (i.t.o kN/m2) | |
|---|---|---|---|---|---|
| avg | 0.35 | avg | 0.084209 | | 1.026667 |
| std dev | 0.035 | std dev | 0.012662 | | 0.154 |

| LN parameters | | Extreme Type 1 para. | | LN parameters | |
|---|---|---|---|---|---|
| Z | -1.054797 | alpha | 101.2968 | Z | 0.015192 |
| gamma | 0.099751 | u | 0.078513 | gamma | 0.149166 |

| Dl* = | 0.44 | LL* = | 0.084 | R* = | 0.54 |
|---|---|---|---|---|---|

| | | | |
|---|---|---|---|
| | | F(max1*) | 0.811961 |
| | | f(max1*) | 17.13272 |

| std dev ,N | 0.043891 | std dev ,N | 0.015738 | std dev ,N | 0.08055 |
|---|---|---|---|---|---|
| avg ,N | 0.337121 | avg,N | 0.080069 | avg ,N | 0.880944 |

| alpha*(Avg | -0.47 | alpha* | -0.17 | alpha* | 0.87 |
|---|---|---|---|---|---|
| DL* | 0.4403 | LL* | 0.0933 | R* | 0.5336 |

| | |
|---|---|
| Beta = | 4.982759 |
| p(failure)% | 3.14E-05 |

| D | Beta |
|---|---|
| 0.2 | 5 |
| 0.35 | 5 |
| 0.5 | 5 |

## SABS RELIABILITY FOR CONSTRUCTION WORKERS FOR SMALL AREAS

| Dlcov = | 0.15 | Davg = | 0.15 | SABS load | 0.65 |
|---|---|---|---|---|---|
| Rcov = | 0.15 | Dn/Davg = | 0.952381 | | |
| Rn/Ravg = | 0.952381 | | | | |

| Dl (i.t.o kN/m2) | | LL (i.t.o kN/m2) | | R (i.t.o kN/m2) | |
|---|---|---|---|---|---|
| avg | 0.15 | avg | 0.577214 | | 1.413333 |
| std dev | 0.0225 | std dev | 0.100252 | | 0.212 |

| LN parameters | | Extreme Type 1 para. | | LN parameters | |
|---|---|---|---|---|---|
| Z | -1.908245 | alpha | 12.79356 | Z | 0.334826 |
| gamma | 0.149166 | u | 0.532113 | gamma | 0.149166 |

| Dl* = | 0.155 | LL* = | 0.9 | R* = | 1.06 |
|---|---|---|---|---|---|

| | | | |
|---|---|---|---|
| | | F(max1*) | 0.991005 |
| | | f(max1*) | 0.114558 |

| std dev ,N | 0.023121 | std dev ,N | 0.212069 | std dev ,N | 0.158116 |
|---|---|---|---|---|---|
| avg ,N | 0.148193 | avg,N | 0.39828 | avg ,N | 1.35315 |

| alpha*(Avg | -0.09 | alpha* | -0.80 | alpha* | 0.60 |
|---|---|---|---|---|---|
| DL* | 0.1543 | LL* | 0.9128 | R* | 1.0671 |

| | |
|---|---|
| Beta = | 3.037934 |
| p(failure)% | 0.11911 |

**4m, 1.4m**

| D | Beta | Pf (%) |
|---|---|---|
| 0.1 | 1.82 | 3.5 |
| 0.12 | 1.83 | 3.4 |
| 0.15 | 1.84 | 3.3 |
| 0.18 | 1.85 | 3.2 |

**5m, 1.7m**

| D | Beta | Pf (%) |
|---|---|---|
| 0.15 | 2.65 | 0.4 |
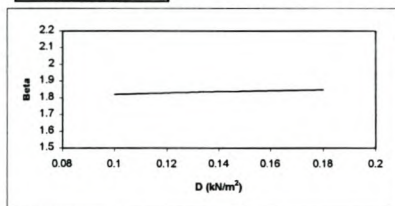


## SABS RELIABILITY FOR MAINTENANCE WORKERS FOR SMALL AREAS

| Dlcov = | 0.15 | Davg = | 0.15 | SABS load | 0.41 |
|---|---|---|---|---|---|
| Rcov = | 0.15 | Dn/Davg = | 0.952381 | | |
| Rn/Ravg = | 0.952381 | | | | |

| Dl (i.t.o kN/m2) | | LL (i.t.o kN/m2) | | R (i.t.o kN/m2) | |
|---|---|---|---|---|---|
| avg | 0.15 | avg | 0.383589 | | 0.965333 |
| std dev | 0.0225 | std dev | 0.033889 | | 0.1448 |

| LN parameters | | Extreme Type 1 para. | | LN parameters | |
|---|---|---|---|---|---|
| Z | -1.908245 | alpha | 37.84675 | Z | -0.046407 |
| gamma | 0.149166 | u | 0.368323 | gamma | 0.149166 |

| Dl* = | 0.165 | LL* = | 0.46 | R* = | 0.62 |
|---|---|---|---|---|---|

| | | | |
|---|---|---|---|
| | | F(max1*) | 0.969352 |
| | | f(max1*) | 1.141958 |

| std dev ,N | 0.024612 | std dev ,N | 0.060648 | std dev ,N | 0.092483 |
|---|---|---|---|---|---|
| avg ,N | 0.147438 | avg,N | 0.346506 | avg ,N | 0.88761 |

| alpha*(Avg | -0.22 | alpha* | -0.54 | alpha* | 0.82 |
|---|---|---|---|---|---|
| DL* | 0.1660 | LL* | 0.4593 | R* | 0.6253 |

| | |
|---|---|
| Beta = | 3.474517 |
| p(failure)% | 0.025593 |

| D | Beta |
|---|---|
| 0.15 | 3.5 |

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KTES 624·172 DEV

**VAK:** ...............................................

**TITELNR.:** 586085 ................

**DATUM:** ........................................

**SKENKER/HERKOMS:** ...................

..............................................

**BESIT:** ...........................................

**DUPLIKAATOPNAME:** ...................

**HANDTEKENING:** ..........................

**BESTEMMING:** ...............................