Fire resistance rating assessment and calculations for the Voidcon Permanent Formwork system

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1 Introduction

This report has been compiled to provide a preliminary rational structural fire assessment and design methodology for the Voidcon Permanent Formwork system as requested by Mr Andries Botha of the Voidcon Group. The VoidPro 50, 115 and 200 formwork profiles have been considered. Figure 1 (a)-(c) provide specifications regarding the panels while (d) shows a slab cast with the permanent formwork system. Note that any depth slab can be cast with a specific VoidPro profile.

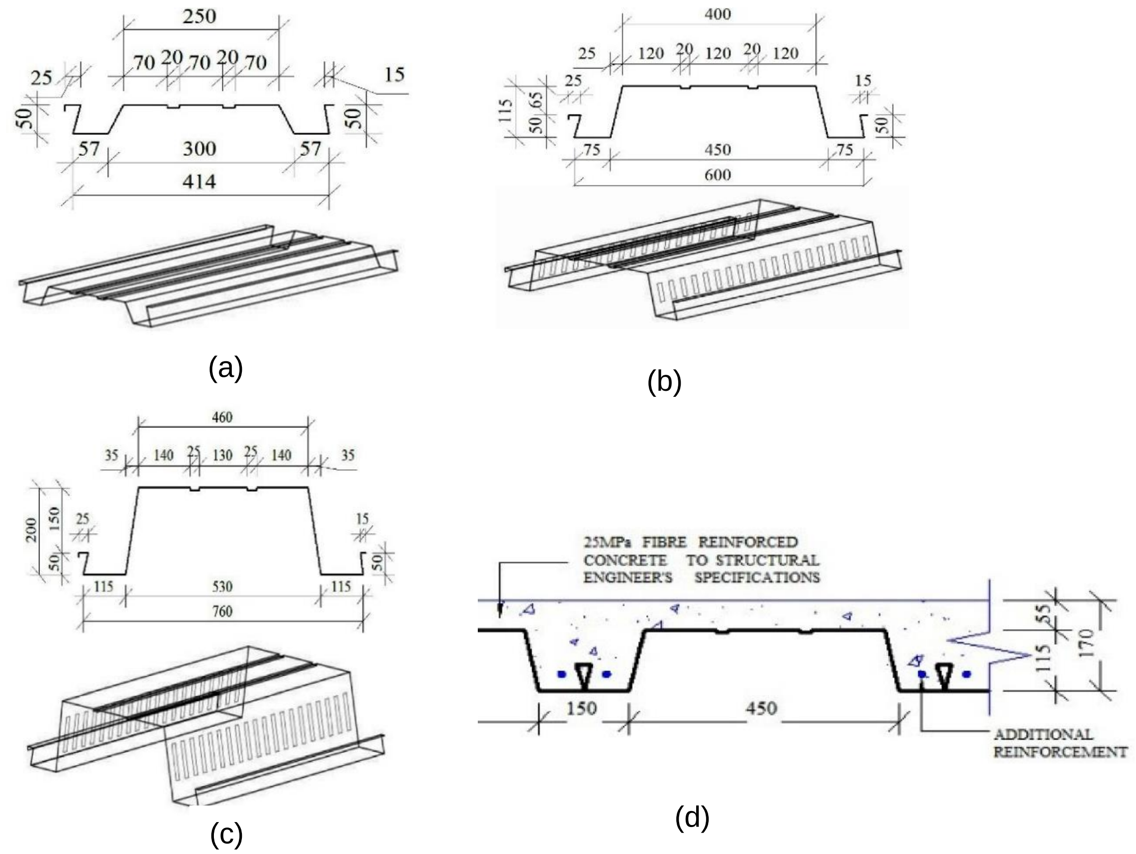

Figure 1: (a) VoidPro 50 Panel, (b) VoidPro 115 Panel, (c) VoidPro 200 Panel, and (d) VoidPro 115 formwork as part of a 170mm thick slab

In this report Section 2 provides a summary of the calculations carried out to determine fire resistance values. Additional details regarding the procedures used for calculations are contained in the Appendix A, while SANS 10400-T requirements are contained in Appendix B.

1.1 Defining Fire Resistance Ratings (FRR) of structural items

All building structures in South Africa are required to have a fire resistance rating (FRR) according to SANS 10400-T (SABS 2011b). The three fire requirements that a floor system has to satisfy are:

- "R" - Stability / strength / structural resistance – Load carrying capacity and deflection.
- "E" – Integrity – The passage of smoke and flame through the slab must be prevented.
- "I" – Insulation – This limits the amount of heat that is allowed to pass through the slab. The aim is to prevent spontaneous combustion on the upper surface of the slab. The average temperature rise of the upper surface may not exceed 140°C, and the peak temperature rise must not exceed 180°C.

If a slab is not required to perform a separating / compartmentalisation function the "E" and "I" criteria do not need to be satisfied (SABS 2000). In this report it is assumed that all three requirements are must be satisfied – "REI". It has been found the load-bearing and insulation requirements govern the design in fire on the VoidPro slabs.

1.2 Code basis for design

South Africa is about to adopt the Eurocode EN 1992-1-1 standard for the design of concrete structures, with the old SANS 10100-1 (SABS 2000) standard soon falling away. This new EN-based concrete design code will be referred to as SANS 51992-1-1. Furthermore, the South African loading code, SANS 10160 (SABS 2011a), is based upon the Eurocode suite of documents. The Eurocodes are commonly acknowledged as the most technically advanced suite of design standards in the world, especially for structural fire design. Hence, they have been adopted for this work. The 500°C isotherm design method utilised in this work is found in EN 1992-1-2 (BSI 2005a) while the temperature considerations for trapezoidal decks have been obtained from EN 1994-1-2 (BSI 2005b). In South Africa studies have been conducted to determine the suitability of EN codes for use here, which has shown they can be applied relatively easily (Walls & Botha 2016; Walls & Viljoen 2016; Walls et al. 2014).

In addition to the codes discussed above the SCI (UK) has produced the guideline document PN005c-GB (SCI 2014) for the determination of temperatures and strength in profiled slabs. Due to the significantly more complex calculations associated with this design document it has only be used for checking insulation requirements and reinforcement temperature, and not for a simplified structural stability design method. However, for future computer software the methods proposed in this document could be adopted for the VoidPro profiles.

1.3 Scope of work

The VoidPro profiles have a similar profile to existing trapezoidal decking systems, for which there is a large amount of test data and codified design information available. However, there are two differences relative to existing trapezoidal panels, meaning that rational design and engineering judgment has been necessary in this report, namely:

- The re-entrant dovetail section of the VoidPro decks, which is used to link panels together. This provides an atypical trough section which will influence heat transfer through the slab.
- The dimensions of the VoidPro decks fall outside of the recommended dimensions according to EN 1994-1-2 for the use of standard design equations for the determination of temperatures and panel properties.

Hence, for certain calculations carried out below it has been necessary to carry out a number of calculations to obtain a final estimate of FRR. For an accurate FRR full-scale fire tests on the system would need to be conducted. For full-scale tests floors or similar structural components are typically tested according to SANS 10177 (SABS 2005).

Note that the recommendations provided in this report do not apply to:

- Post-tensioned slabs. Additional literature should be consulted for such elements.
- Light-weight concretes. However, research has shown that light-weight concrete typically perform better than normal-weight concrete (BSI 2005a)
- Time-temperature graphs significantly different to the ISO 834 (ISO 1999) standard fire curve.

1.4 Comments based on existing design recommendations

It has been noted that in certain permanent formwork design guidelines currently available in South Africa it is stated that the inclusion of a top mesh of reinforcing steel provides a specific fire rating, and no further reinforcing steel is required. This is only possible when there is sufficient anchorage given to reinforcement at supports, for instance where the shear studs of composite beams provide restraint. When restrained composite concrete slabs are heated in fire they sag, experience large deflections and become hanging catenaries (Bailey & Moore 2000). This behaviour if often referred to as ‘tensile membrane behaviour' or 'catenary action'. However, in a typical brick structure concrete slab reinforcement is not linked to adjacent brickwork, or any other part of the structure. Therefore, unconnected top rebar cannot provide anchorage and allow the formation of catenary action. In such cases the presence of only a top mesh will not be sufficient for providing load-bearing fire ratings of 60 or 120 minutes, and trough rebar is required. Wang et al (2012) note that:

> *Should catenary action be used as the load-bearing mechanism for structural robustness in fire, the surrounding structure, in addition to the connections, must be designed to possess sufficient resistance to the combined catenary force and other applied forces in the structural member.*

Based on such behaviour the design recommendations provided in this document have been conservatively based upon assuming that the slab decks are not restrained around their perimeter, and slabs have been designed as simply supported beams T-beams.

For composite structures it will be possible to obtain higher fire ratings using less reinforcement by including anchorage and catenary behaviour (although insulation requirements will be unaffected). Design methods such as the Slab Panel Method (SPM) (Clifton 2006) or MACS+ (Nadjai et al. 2012) are suitable for such designs. However, the detailing rules required to use these methods must be adhered to, where sufficient anchorage of reinforcement around shear studs is required. A simplified explanation regarding how the SPM design procedure works is presented by Geldenhuys & Walls (2015).

2 Design calculations

For the design of a VoidPro slab in fire the following process should be followed:

(a) Ensure specifications comply with the assumptions of Section 2.1 (or are superior to these assumptions – e.g. additional cover).
(b) For a 30 minute fire rating the guidelines of Section 2.2 are followed.
(c) For resistances of 60 minutes or more ensure that structural strength is satisfied according to the guidelines of Section 2.4.
(d) Check that insulation requirements are satisfied based on the total slab thickness provisions of Section 2.5.

(e) If deflections could cause loss of compartmentation ensure that construction details are provided to prevent this.

2.1 Design specifications

The following assumptions have been made to calculate FRRs of the VoidPro panels:

Concrete strength:	C25/30 (i.e. 30MPa cube strength)
Rebar properties:	Yield strength – 450 MPa. Young's modulus – 200 GPa. Cold-worked.
Cover:	25mm
Aggregate type:	Siliceous (calcareous aggregates such as limestone have increased fire resistance)
Top steel / mesh:	If required minimum top reinforcement as per EN 1994-1-1 should be added to reduce cracking for continuous beams.
Bar diameter:	Y10 for temperature calculations as it provides hotter estimates.
Design layout:	All sections are designed as simply-supported T-beams, as per EN 1994-1-2.

The entire steel formwork profile is neglected for calculations as it rapidly loses strength in fire. It has been calculated that at 60 minutes the temperature of the horizontal trough section reaches 740°C resulting in it having only 12% of its original strength. Furthermore, in certain full-scale tests it has been observed that decks melt through or debond (Mensinger et al. 2012). Hence, trough rebar is necessary for ensuring structural integrity in fire.

It appears that the re-entrant or dovetail portion of the decking system will be beneficial in terms of ensuring that the VoidPro profiles do not debond during a fire. Furthermore, this dovetail portion will remain cooler than the exposed perimeter section, thereby increasing the load-bearing capacity in fire. However, until additional testing and finite element model results are available its beneficial influence has been neglected.

2.2 Low fire resistance rating specifications – 30 minutes

EN 1994-1-2 highlights that a 30 minute load-bearing "R" fire rating can be obtained relatively easily, even without bottom reinforcement, based on the following:

> *For a design complying with EN 1994-1-1 [ambient temperature code], the fire resistance of composite concrete slabs with profiled steel sheets, with or without additional reinforcement, is at least 30 minutes, when assessed under the load bearing criterion "R". (Clause 2.1.2)*

2.3 Minimum reinforcement requirements for cracking (optional)

EN 4-1-2 does not specifically deal with a minimum reinforcement area that must be present for fire resistance. If a slab has been assessed such that it will not lose its integrity and insulation requirements due to cracking then mesh does not need to be provided. However, to prevent cracking of slabs according to EN 1994-1-1 (BSI 2011) the following recommendations are provided to control serviceability limit state requirements:

> *Where continuous slabs are designed as simply-supported in accordance with 9.4.2(5), the cross-sectional area of the anti-crack reinforcement above the ribs should be not less than 0.2% of the cross-sectional area of the concrete above the ribs for un-propped construction and 0.4% of this cross-sectional area for propped construction. (Clause 9.8.1 (2))*

Clause 9.4.2 (5) states that:

> *A continuous slab may be designed as a series of simply supported spans. Nominal reinforcement in accordance with 9.8.1 should be provided over intermediate supports. (Clause 9.4.2 (5))*

Based on minimum reinforcement areas of 0.2% for un-propped construction and 0.4% for propped construction reinforcing specifications are proposed in Table 1. Either mesh or normal reinforcement may be used.

Profile	**Total slab thickness (mm)**	**Un-propped construction**		**Propped construction**	
		Min. mesh area (mm^2/m)	**Mesh Ref. No.**	**Min. mesh area (mm^2/m)**	**Mesh Ref. No.**
VoidPro 50	100	100	100	200	311
	125	150	193	300	395
	150	200	311	400	617
VoidPro 115	170	110	100	220	311
	180	130	193	260	395
	200	170	245	340	500
	225	220	311	440	617
VoidPro 200	255	110	100	220	311
	275	150	193	300	395
	300	200	311	400	617
	350	300	395	600	888
	400	400	617	800	Y12-150

Table 1: Minimum rebar area and possible Mesh Ref. No. to satisfy minimum reinforcement requirements according to EN 1994-1-1

2.4 FRR calculations – "R" - Structural strength

The resistance of floor slabs in sagging is to be carried out according to the 500°C Isotherm Method of EN 2-1-2 (BSI 2005a), or Buchanan (2001). The basis for this simplified method is that all concrete with a temperature greater than 500°C is to be ignored, while all concrete with a temperature less than 500°C is to be assumed to have its full strength. Reinforcement is designed with a reduced strength based on its temperature.

The process to be followed is:

1. Calculate the fire limit bending moments, based on the load combinations provided in Equation 1 and Table 2. The load combination to be checked at the fire limit state (FLS) is based upon the accidental load combination (ACC) of SANS 10160-1 (SABS 2011a):

	$FLS\ Load = 1.0 \times Permanent\ Load + \psi \times Imposed\ load$	*Eq. 1*

   The combination factor, ψ, is taken according to SANS 10160 as specified in Table 2.

2. Determine the depth of the 500°C temperature isotherm for the standard fire. See Table 3 and Figure 2 for more details.
3. Determine the temperature of reinforcing steel at the centre of bars and calculate the reduced strength of reinforcing steelwork ($f_{yT} = k_{yT} f_y$). This is provided in Table 4.
4. Calculate the resistance of the section according to normal concrete design methods (SANS 10100-1, SANS 51992-1-1, or to Buchanan (2001)) but using the reduced rebar strength, f_{yT}. Ignore the contribution of the permanent formwork. The following equations are suggested as a simple approach:

   (a) Depth of concrete compression block:

	$a_f = \dfrac{A_s f_{y,T}}{0.67 f_{cu} b}$	*Eq. 2*

   (b) Sagging moment capacity in fire:

	$M_f = A_s f_{y,T}\left(d - a_f/2\right)$	*Eq. 3*

Where:

a_f	Depth of compression block
A_s	Area of reinforcing steel in tension (per rib)
b	Width of the rib
d	Effective depth to centre of reinforcement
f_{cu}	Characteristic strength of the concrete

$f_{y,T}$ Yield strength of reinforcing steel at temperature T, where $f_{y,T} = k_{y,T} f_y$

5. Ensure that the neutral axis (referred to as x in SANS 10100-1) of the section falls within the upper section of the beam which is cooler than 500°C. If this does not occur the design must be adjusted or specialist literature consulted. This depth of the neutral axis is calculated as $\frac{a_f}{0.9}$, and is measured from the top of the slab.

Note that EN documents permit using a lower partial material factor for the ACC limit state (i.e. higher resistances are then predicted). Hence, partial material factors for concrete and steelwork may be been taken as 1.0.

Category	**Specific use**	**ψ – Combination factor**
A	Domestic and residential areas	0.3
B	Public areas not susceptible to crowding	0.3
C	Public areas where people may congregate	0.3
D	Shopping areas	0.3
E1	Light industrial use	0.5
E2	Industrial use	0.6
E3	Storage areas	0.8
H	Inaccessible roofs	0.0
J	Accessible flat roofs, excluding occupancy categories A to D	0.3
K	Accessible flat roofs with occupancies A to D	In accordance with Categories A to D

Table 2: Combination factor for the fire limit state (FLS).

FRR	**a_{500} - 500°C Isotherm Depth (mm)**
60 min	23
90 min	32
120 min	40

Table 3: Depth of 500°C isotherm for the design of slabs in sagging according to EN 1994-1-2 Table D.5

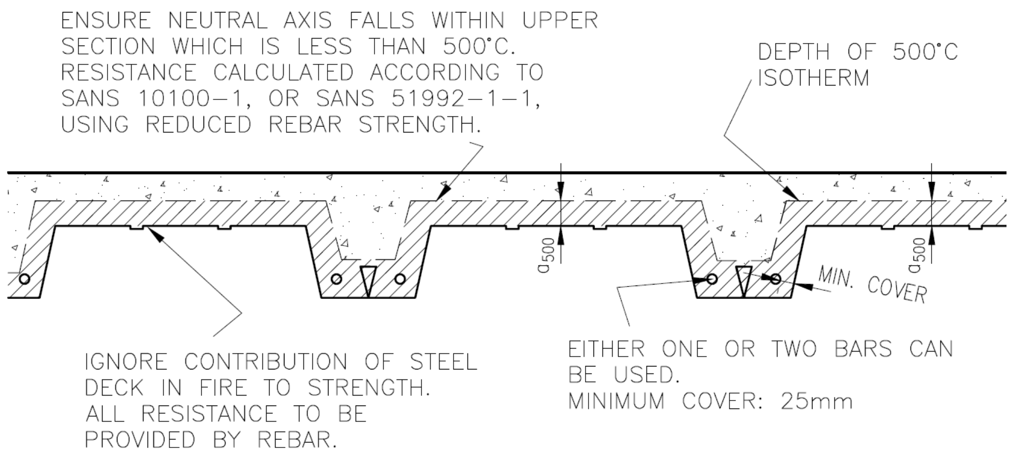

Figure 2: Fire limit state (FLS) design considerations and layout

Fire resistance time (min)	T - Steel temp (°C):	k_{yT} - Reduction factor
60 min	460	0.78
90 min	610	0.37
120 min	720	0.19

Table 4: Temperature and reduction factors for reinforcing steel at different standard fire times assuming 20mm

The temperatures obtained for rebar have been compared with the SCI report guidelines (SCI 2014) and found to be conservative. With additional research reductions in temperature may be justified.

For continuous slabs a savings in sagging reinforcement can be made using continuity, but the literature listed above should be consulted for the calculation of hogging moment capacity. For composite steel-concrete floors in fire specialist literature should be consulted such as that of the MACS+ design software or Slab Panel Method (SPM) for designing slabs in fire. This can lead to reductions in required reinforcement and reduced passive protection requirements for steel beams. Detailing requirements associated with the aforementioned methods must be adhered to, to ensure that cracking in slabs does not occur.

2.5 FRR calculations – "E" - Insulation

The standard design calculations provided in EN 4-1-2 (BSI 2005b) for the determination of insulation FRR for trapezoidal decks do not fully apply to Voidcon Permanent Formwork system due to its geometry being outside the dimensions that design tables have been based upon. Hence, it has been necessary to perform a variety of calculations to obtain an estimate of insulation FRR. The insulation FRR value determined is based upon considering:

(a) a flat slab the thickness of the flange (EN 1992-1-2) (very conservative / lower bound),
(b) an equivalent flat slab with an averaged thickness (EN 1994-1-2),
(c) using the guidelines of BS 5950-8 (BSI 2003), and
(d) a trapezoidal deck (EN 1994-1-2) (potentially non-conservative / upper bound).

From the four procedures a suggested estimate of the design insulation FRR value to be adopted is given based on interpretation of the data. Further testing would be required to refine these estimates. The VoidPro 115 and 200 panels are wide so start becoming closer to a flat slab in the middle sections, although the ribs do assist with reducing the average temperature rise.

Profile	Total slab thickness (mm)	Insulation FRR (min)				
		EN 1992-1-2 flat slab (lower limit)	BS 5950-8	EN 1994-1-2 Effective slab	EN 1994-1-2 Trapezoidal deck (upper limit)	Suggested design value
VoidPro 50	**100**	<30	<30	30	49	**30**
	125	50	75	75	87	**60**
	150	90	120	90	126	**90**
VoidPro 115	**170**	<30	<30	55	67	**30**
	180	35	45	75	82	**45**
	200	65	105	90	113	**75**
	225	105	120	120	152	**105**
VoidPro 200	**255**	<30	<30	60	81	**45**
	275	50	75	90	112	**60**
	300	90	120	180	151	**120**
	350	105	120	240	228	**120**
	400	105	120	240	306	**120**

Table 5: Insulation fire resistance rating for the VoidPro Permanent Formwork system for different thickness slabs

Based on the calculations above the minimum thickness required to obtain a 60 minute resistance for the different profiles is given below. This gives a required average thickness of 70-75mm above the flute to provide the required 1 hour rating. It is believed that the thicknesses recommended could be reduced if further testing is conducted.

Profile	Min. thickness for 60min insulation FRR	Min. thickness above flute (mm)
VoidPro 50	120	70
VoidPro 115	190	75
VoidPro 200	275	75

Table 6: Minimum thickness of VoidPro systems to satisfy a 60 minute insulation fire resistance rating

2.5.1 The use of screeds

Note that if a structural screed/grout is placed on the concrete its thickness may be included when satisfying minimum thickness requirements. However, its thickness must not be included when carrying out the structural design calculations outlined in this document. For example, a 170mm thick VoidPro 115 slab with a 20mm thick screed on top (minimum thickness) will be equivalent to a 190mm slab for insulation resistance, but for structural resistance the slab must be designed as 170mm thick. However, it must be ensured that the screed is securely bonded to the slab, will not delaminate during a fire, and does not have a waterproofing layer, insulation layer or other such layer in between the concrete and screed.

2.6 FRR calculations – "I" - Integrity

The integrity of concrete slabs with permanent steel formwork is typically high. Buchanan (2001) notes that:

> *Composite steel-concrete slabs generally have excellent integrity because even if cracks occur in the concrete slab, the continuous steel deck will prevent any passage of flames or hot gases through the floor system.*

This is supported by Clifton (2006) who states that:

> *With a concrete slab cast onto steel deck, integrity is retained even when a full depth crack greater than around 1mm in width develops in the slab, because the decking will seal the base of the crack against the passage of flame or hot gases.*

Hence, integrity will not be dealt with in detail in this report. However, engineers should ensure that at joints between VoidPro slabs and walls, columns, staircase or other such items that there cannot be a passage of smoke or flame, even if slabs expand or sag. If cracks can open up in areas which may allow smoke or flames through then anti-crack reinforcement should be added as per Table 1. For the design of composite slabs it is possible that restraint around the perimeter of slabs can lead to gaping cracks occurring in the middle of slabs as they hang (Bailey & Toh 2007; Stadler 2012). Additional literature should be consulted to deal with 60 or 120 minute fire ratings for composite slabs where restraint is present.

2.7 FRR Calculations - Deflections

Note that if the deflection of slabs during a fire could result in loss of compartmentation it will be necessary to include systems that allow for deformations to safely occur, such as the inclusion of deformable ceramic blankets (Clifton 2006).

3 Conclusions

Based on the findings of this report as estimate of fire resistance rating (FRR) has been provided for the VoidPro permanent formwork decking systems. The FRR is based upon satisfying structural, insulation and integrity requirements. All panels are designed as simply-supported decks with sufficient top mesh to address ambient temperature criteria. With further testing and modelling results can be refined.

4 Appendix A – Design calculations

Calculations used for the determination of Fire Resistance Ratings (FRRs) used in this report are presented below. The layout of the typical trapezoidal deck section considered by EN 1994-1-2 is shown in Figure 3. The inclusion of a structural screed (dimension h_3) can be used to increase the insulation rating of slabs, although it has been excluded in the calculations below.

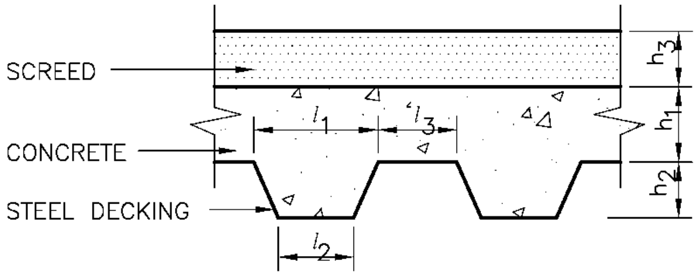

Figure 3: Trapezoidal deck layout and details

4.1 FRR calculations – "R" - Structural strength

The temperature of reinforcing steel has been determined based on Section D.2 of EN 4-1-2 and using the guidelines of the NCCI document (SCI 2014). Results are summarised in Table 4, with calculations presented below.

EN 4-1-2 calculates the temperature of reinforcing steel as:

$$\theta_s = c_0 + \left(c_1.\frac{u_3}{h_2}\right) + (c_2.z) + \left(c_3.\frac{A}{L_r}\right) + (c_4.\alpha) + \left(c_5.\frac{1}{l_3}\right) \qquad \textit{Eq. 4}$$

where u_3 is defined below, α is angle of the web of the ribs and $\frac{A}{L_r}$ defines the geometry of the section as defined below. The parameters c_0 to c_5 are given by:

Concrete	Fire resistance [min]	c_0 [°C]	c_1 [°C]	c_2 [°C]. mm$^{0.5}$	c_3 [°C].mm	c_4 [°C/°]	c_5 [°C].mm
Normal weight concrete	60	1191	-250	-240	-5,01	1,04	-925
	90	1342	-256	-235	-5,30	1,39	-1267
	120	1387	-238	-227	-4,79	1,68	-1326
Light weight concrete	30	809	-135	-243	-0,70	0,48	-315
	60	1336	-242	-292	-6,11	1,63	-900
	90	1381	-240	-269	-5,46	2,24	-918
	120	1397	-230	-253	-4,44	2,47	-906

Table 7: Parameters for the determination of rebar temperature in ribs

The parameter z defines the position of the bar in the rib:

$$\frac{1}{z} = \frac{1}{\sqrt{u_1}} + \frac{1}{\sqrt{u_2}} + \frac{1}{\sqrt{u_3}} \qquad \text{Eq. 5}$$

With the dovetail section of the VoidPro section the dimension u_2, as shown in Figure 4, has been conservatively reduced to be 75% of the measured value. With further finite element modelling and full-scale testing it may be possible to adjust this assumption. It is assumed that bars are placed at the hottest position which will be on the far left or right of troughs, with 20mm cover to the side. The values of u_1 and u_3 have a much larger influence on rebar temperature than u_2.

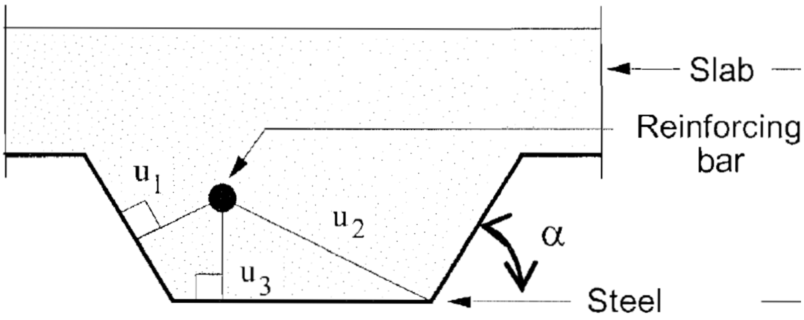

Figure 4: Parameters for the determination of temperature of rebar (BSI 2005a)

The geometry of the slab is defined by the parameter A/L_r which is calculated as follows:

$$\frac{A}{l_r} = \frac{h_2.\left(\frac{l_1+l_2}{2}\right)}{l_2 + 2\sqrt{h_2^2 + \left(\frac{l_1-l_2}{2}\right)^2}} \qquad \text{Eq. 6}$$

The area of the cross-section has been reduced by the area of the dovetail section. However, insulation resistances are primarily governed by the thickness of the slab on the top of the formwork, i.e. h_1, so the dovetail has limited influence on insulation results.

For the Voidcon 200 profile the results obtained for the methodology above become non-conservative due to the presence of the large cross-section, with parameters outside of what the design calculations are based upon. Hence, the guidelines from the SCI (2014) has been used as a lower bound limit for solutions. The equations for determining the temperature of rebar are determined based upon:

$$\theta_s = ax_{max}^2 + bx_{max} + c \qquad \text{Eq. 7}$$

$$x_{max} = \max\left(\bar{x}, cover + \frac{\emptyset}{2}\right) \qquad \text{Eq. 8}$$

$$\bar{x} = -\frac{b}{2a} \quad \text{Eq. 9}$$

$$\sigma = \left(\frac{250}{h_1 + h_2}\right)^{0.5} \quad \text{Eq. 10}$$

FRR (min)	a	b	c
30	0.055	$-8.9\sigma^{-0.045}$	450
60	0.048	$-11.2\sigma^{-0.05}$	760
90	0.036	$-10.6\sigma^{-0.03}$	890
120	0.03	$-10.3\sigma^{-0.15}$	985

Table 8: Parameters for rebar temperature calculation (SCI 2014)

Based on the above it has been found that the different VoidPro systems have approximately the same temperatures at each FRR rime as shown in Table 4. For simplicity these values have been rounded to the nearest 10°C.

4.2 FRR calculations – "E" – Insulation

The VoidPro system has been considered as (a) a flat slab, (b) effective flat slab, and (c) trapezoidal deck to provide an estimate of the insulation FRR. For trapezoidal decks the dimensions are defined in Figure 3. Structural screeds can be used to provide additional fire resistance rating.

4.2.1 Flat slab

The most conservative solution is to consider the flange section of slabs (dimension h_1) as a flat slab which is given no shielding by the trough sections. The thickness of flange required to limit the unexposed face to 160°C (i.e. 140°C increase above 20°C ambient temperature) is given by Table 9, based on: (a) EN 1992-1-2 flat slab guidelines, (b) temperature profiles in Table D.5 of EN 1994-1-2, (c) BS5950-8, and (d) the NCCI-PC005c document. The NCCI guidelines only apply to smaller trapezoidal geometries so would not be fully applicable to deeper VoidPro slabs.

Time (min)	Min. flange (h_1) thickness			
	EN 1992-1-2 – Flat slab	EN 1994-1-2	BS5950-8	NCCI
30	60	45	60	60
60	80	69	70	60
90	100	100	80	70
120	120	-	90	80

Table 9: Flat slab thickness required to provide insulation FRR assuming no trough sections

The older CIRIA 107 (Lawson 1985) recommendations specify that for a 1 hour fire rating the thickness of concrete flange should be a minimum of 75mm.

4.2.2 Effective flat slab

An alternative method provided by EN 1994-1-2 is to convert trapezoidal decks into equivalent flat slabs of thickness h_{eff} for which calculations can then be done based upon standard flat slab design methodologies. The effective thickness is calculated by:

$$h_{eff} = h_1 + 0.5h_2\left(\frac{l_1+l_2}{l_1+l_3}\right) \quad \text{for } h_2/h_1 \leq 1.5 \text{ and } h_1 > 40\text{mm} \qquad \textit{Eq. 11}$$

$$h_{eff} = h_1\left[1 + 0.75\left(\frac{l_1+l_2}{l_1+l_3}\right)\right] \quad \text{for } h_2/h_1 > 1.5 \text{ and } h_1 > 40\text{mm} \qquad \textit{Eq. 12}$$

$$h_{eff} = h_1 \quad \text{for } l_3 > 2l_1 \qquad \textit{Eq. 13}$$

Minimum values for h_{eff} for specific insulation FRR times are provided in the following table:

Insulation FRR (min)	Minimum effective thickness (mm)
30	$60 - h_3$
60	$80 - h_3$
90	$100 - h_3$
120	$120 - h_3$

Table 10: Minimum effective thickness of slabs according to EN 1994-1-2. Note that if there is no screed h_3 is taken as zero.

Table 11 summaries the insulation FRR values calculated for the VoidPro panels based on effective thickness calculations:

Profile	Total slab thickness (mm)	h_{eff} - Effective slab thickness (mm)	Insulation FRR based on h_{eff} (min)
VoidPro 50	100	67	30
	125	92	75
	150	117	90
VoidPro 115	170	79	55
	180	93	75
	200	119	90
	225	144	120
VoidPro 200	255	84	60
	275	114	90
	300	152	180
	350	220	240
	400	270	240

Table 11: Insulation FRR based on effective slab thickness calculations

4.2.3 Trapezoidal deck calculations

For standard trapezoidal panels the insulation FRR can be calculated using the equations of EN 4-1-2 and applied to the Voidcon Permanent Formwork system for different thicknesses. as provided in Table 12. The insulation fire resistance ratings have been rounded down to the nearest 15 minute standard fire rating. These solutions are *non-conservative* since the geometry of the Voidcon system is outside that of the standard trapezoidal layouts the equations have been design for. Of particular concern is the large distance between trough sections, distance l_3, which will be more susceptible to radiant heat flux due to the reduced shielding effect of troughs. However, these calculations provide a good upper bound solution which can be used in conjunction with the above, as the system is somewhere between a flat slab and trapezoidal deck. The trough sections do provide radiative heat shielding for the flanges, and reduce the average heat rise.

Profile	Profile	Insulation FRR as trapezoidal deck (min)
VoidPro 50	100	45
	125	75
	150	120
VoidPro 115	170	60
	180	75
	200	105
	225	150
VoidPro 200	255	75
	275	105
	300	150
	350	225
	400	300

Table 12: Insulation fire resistance rating for the Voidcon Permanent Formwork system with different thicknesses when considered as trapezoidal panels

The fire resistance with respect to thermal insulation presented has been calculated as:

$$t_i = a_0 + a_1.h_1 + a_2.\varphi + a_3.\frac{A}{L_r} + \frac{a_4}{l_3} + \frac{a_5}{l_3}.\frac{A}{l_r} \qquad \text{Eq. 14}$$

The geometry of the slab is as defined in Figure 3. The coefficients a_1 to a_5 are selected from Table 13. As discussed above, normal weight concrete has been used for all calculations.

	a_0 [min]	a_1 [min/mm]	a_2 [min]	a_3 [min/mm]	a_4 [mm min]	a_5 [min]
Normal weight concrete	-28,8	1,55	-12,6	0,33	-735	48,0
Lightweight concrete	-79,2	2,18	-2,44	0,56	-542	52,3

Table 13: Design coefficients for determining insulation fire resistance ratings

The configuration factor, φ, addresses the amount of shielding that is provided to the upper flange section and is calculated by:

$$\varphi = \left(\sqrt{h_2^2 + \left(l_3 + \frac{l_1 - l_2}{2}\right)^2} - \sqrt{h_2^2 + \left(\frac{l_1 - l_2}{2}\right)^2}\right) \Big/ l_3 \qquad \text{Eq. 15}$$

5 Appendix B – SANS 10400-T Requirements

Stability requirements of structural elements or components are provided below according to SANS 10400-T Table 6.

1	2	3	4	5	6	7
Type of occupancy	**Class of occupancy**	**Stability** min				
		Single-storey building	**Double-storey building**	**3 to 10 storey building**	**11 storeys and more**	**Basement in any building**
Entertainment and public assembly	A1	30	60	120	120	120
Theatrical and indoor sport	A2	30	60	120	120	120
Place of instruction	A3	30	30	90	120	120
Worship	A4	30	60	90	120	120
Outdoor sport	A5	30	30	60	90	120
High risk commercial service	B1	60	60	120	180	120
Moderate risk commercial service	B2	30	60	120	120	120
Low risk commercial service	B3	30	30	90	120	120
Exhibition hall	C1	90	90	120	120	120
Museum	C2	60	60	90	120	120
High risk industrial	D1	60	90	120	180	240
Moderate risk industrial	D2	30	60	90	120	180
Low risk industrial	D3	30	30	60	120	120
Plant room	D4	30	30	60	90	120
Place of detention	E1	60	60	90	120	120
Hospital	E2	60	90	120	180	120
Other institutional (residential)	E3	60	60	120	180	120
Medical facilities	E4	30	30	Not applicable	Not applicable	120
Large shop	F1	60	90	120	180	120
Small shop	F2	30	60	120	180	120
Wholesalers' store	F3	60	90	120	120	120
Office	G1	30	30	60	120	120
Hotel	H1	30	60	90	120	120
Dormitory	H2	30	30	60	120	120
Domestic residence	H3	30	30	60	120	120
Detached dwelling house	H4	30	30	60	Not applicable	120
Hospitality	H5	30	30	Not applicable	Not applicable	120
High risk storage	J1	60	90	120	180	240
Moderate risk storage	J2	30	60	90	120	180
Low risk storage	J3	30	30	90	90	120
Parking garage	J4	30	30	30	90	120

NOTE 1 Unprotected steel may be used in the structural system of all single-storey and certain double-storey buildings in spite of the fact that in many cases such structural members would not comply with the requirements of this table. The practice is regarded as safe for all practical cases that are likely to occur in single-storey construction, but the possible consequences of early distortion or collapse should be considered in the design of double-storey buildings in order to be certain that escape routes will be able to serve their purpose for the required period. Particular care should be exercised where thin sections are used or in "space-frame" type structures.

NOTE 2 A further problem arises in the application of the requirement of 4.2. Distortion or collapse of any structural member should not cause loss of integrity or stability in any external wall facing a site boundary or another building as this might lead to non-compliance with the safety distance requirement. Where such a situation occurs, it would be necessary either to protect the steel to the extent required to attain the stability given in this table or to regard such wall as being of type N for the purposes of 4.2.

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