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WP 3.6 The performance of ferritic stainless steel composite decking during fire

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EXECUTIVE SUMMARY

This report describes an investigation into the performance of ferritic stainless steel decking exposed to fire. Designs for slabs utilising ferritic stainless steel decking were compared to equivalent slabs designed using galvanised steel decking.

The analysis shows that the difference in structural fire performance between ferritic stainless steel decking and galvanised steel decking when used in composite slabs is small. Some increase in bending resistance at longer exposure times (greater than 90 minutes) is seen, but only if the slab contains no bar reinforcement, which is unusual.

Contents

1 INTRODUCTION

This report describes work undertaken towards Task 3.6 of the SAFSS project, concerning the performance of ferritic stainless steel composite decking when exposed to fire.

The resistance of the decking was predicted using the design models given in EN 1994-1-1[1] and EN 1994-1-2[2]. The results obtained for a ferritic stainless steel decking product are then compared to those obtained for a similar galvanised steel decking product.

All materials progressively lose their ability to support a load when they are heated. This is due to the changes in material properties at elevated temperature compared to the material properties at ambient temperature. If components of a structure are heated sufficiently, they may collapse. The consequences of such a collapse may vary, depending on how critical the component is in controlling the overall behaviour of the structure.

The fire resistance of composite slabs is dependent on the temperature distribution in the deck profile and slab, the strength retention of the materials and the shear bond strength between the decking and the concrete at elevated temperatures. The study described in this report did not consider the impact that using ferritic decking would have on the temperature distribution through the slab nor the shear bond strength, as these have only a slight effect on the overall fire resistance of a slab. The impact of the different strength retention factors for ferritic stainless steel is the focus of this work.

2 PERFORMANCE REQUIREMENTS FOR COMPOSITE SLABS AT ELEVATED TEMPERATURE

In accordance with EN 1994-1-2, an element performing a separating function (as normally required for a slab) when exposed to fire must meet three criteria; R, E and I.

R is the resistance to collapse, i.e. the ability to maintain loadbearing resistance (which applies to loadbearing elements only).

E is the resistance to fire penetration, i.e. an ability to maintain the integrity of the element against the penetration of flames and hot gases.

I is the resistance to the transfer of excessive heat, i.e. the ability to provide insulation from high temperatures.

The procedure described in Annex D of EN 1994-1-2 determines the resistance of the composite slab to collapse (R). Section D2 and D3 of EN 1994-1-2 Annex D give guidance on calculating the sagging and hogging moment resistances of a composite slab respectively.

Section D.1 of EN 1994-1-2 Annex D allows the designer to calculate a time period during which the insulation requirement (I) is satisfied for a particular slab.

The integrity criteria (E) cannot be verified by calculation. Instead, reference to test evidence is required. Integrity failure in slabs constructed with decking has never been observed, and no differences in the integrity resistance of decking products formed using ferritic stainless steel is expected. For these reasons it can be reasonably considered that the integrity criteria is inherently satisfied. No further analysis is required.

2.1.1 EN 1994-1-2 Annex D - Geometrical limits

Four decking profiles are included in this study; Cofraplus 60, Cofraplus 77, Cofrastra 40 and Cofrastra 56. Cofraplus 60 and Cofraplus 77 are trapezoidal, while Cofrastra 40 and Cofrastra 56 are re-entrant.

Annex D also gives limitations on dimensions of these profiles and these are listed in Table D.7 Section D5 of EN 1994-1-2 Annex D for the two deck profile types, and also given in Table 2.1.

Table 2.1 EN 1994-1-2 Annex D Table D7

Re-entrant steel sheet profiles	Trapezoidal steel sheet profiles
77.0 mm ≤ l_1 ≤ 135.0 mm	80.0 mm ≤ l_1 ≤ 155.0 mm
110.0 mm ≤ l_2 ≤ 150.0 mm	32.0 mm ≤ l_2 ≤ 12.0 mm
38.5 mm ≤ l_3 ≤ 97.5 mm	40.0 mm ≤ l_3 ≤ 115.0 mm
50.0 mm ≤ h_1 ≤ 130.0 mm	50.0 mm ≤ h_1 ≤ 125.0 mm
30.0 mm ≤ h_2 ≤ 60.0 mm	50.0 mm ≤ h_2 ≤ 100.0 mm

Variable definitions are given in Figure 2.1.

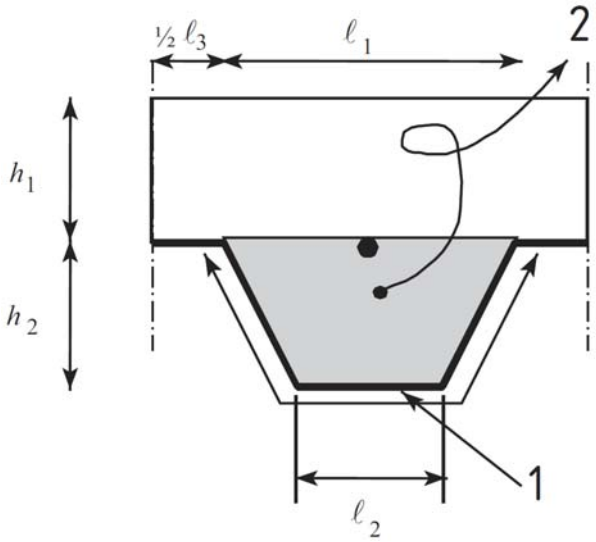

Figure 2.1 Variable definitions used in the calculation of structural performance of composite decking during fire.

1 is the exposed surface length (L_r)
2 is the area of concrete within the rib of the composite slab (A)

For the purpose of this desk study all deck profiles considered conform to the dimensional limitations described in Table D.7 Section D5 of EN 1994-1-2 Annex D.

3 MECHANICAL RESISTANCE (CRITERION R)

Given the mechanical properties of ferritic stainless steel are different to those of carbon steel it can be expected that the mechanical resistance of a slab constructed with either material will not be the same. In order to investigate the extent to which these differences affect slab design, moment resistances of various designs of composite slab were calculated, utilising both ferritic stainless steel decking and conventional galvanised steel decking.

The sagging moment resistances were computed for three different deck arrangements.

1. Deck Only.
2. Deck and top mesh reinforcement in slab.
3. Deck, top mesh reinforcement in slab and rib reinforcement.

The partial factors of the various materials used when designing for fire are obtained from Section 2.3 (1) of EN 1994-1-2. The values are as follows:

$\gamma_{M,fi,c}$ is the partial factor for concrete during fire =1.0
$\gamma_{M,fi,s}$ is the partial factor for steel reinforcement during fire =1.0
$\gamma_{M,fi,a}$ is the partial factor for steel during fire =1.0

It is often the case that reliance on the deck strength only for sagging moment resistance is not sufficient to provide the required fire resistance. This is especially true for longer exposure periods, since the high temperatures reached by the exposed steel results in a significant reduction in strength. If this is the case it is necessary to add additional reinforcement in the rib of the slab to improve the moment resistance.

It is highly unusual for a slab to be designed without top mesh reinforcement. This design is presented for comparison purposes only.

The hogging moment resistance of a composite slab is primarily determined by the mesh reinforcement and the concrete strength at the support. Given the position of the neutral axis, the decking generally provides little moment resistance. Any minimal contribution is often ignored in the design of the section at the support location. No difference is therefore expected between ferritic decking and galvanised decking in this situation.

3.1 Temperature distribution using EN 1994-1-2 Annex D

The method in EN 1994-1-2 Annex D can be used to determine the sagging resistance. The following procedure is followed:

1. The deck is divided into three sections: lower flange, web and upper flange.
2. For each section, a temperature is calculated for the material at a particular fire exposure time; 60, 90 or 120 minutes.
3. For each temperature calculated and for each material used (galvanised steel or stainless steel decking) a reduction factor for the yield strength of the steel decking is calculated.

3.1.1 Calculation of the steel decking temperature

Annex D, Section D.2 of EN 1994-1-2 gives a method of calculating the temperatures of the lower flange, web and upper flange of the steel decking. The steel decking temperature is given by Eq. (1).

$$\boldsymbol{\theta_a = b_0 + b_1 \frac{1}{l_3} + b_2 \frac{A}{L_r} + b_3 \phi + b_4 \phi^2} \qquad \text{Eq. (1)}$$

Where:

θ_a is the temperature of the lower flange, web or upper flange

A is the concrete volume of the rib per metre of rib length (see Figure 2.1) (mm^3/m)

L_r is the exposed area of the rib per metre of rib length (mm^2/m)

A/L_r is the rib geometry factor (mm)

Φ is the view factor of the upper flange

l_3 is the width of the upper flange (see Figure 2.1) (mm)

For normal weight concrete values for the factors $b_0 - b_4$ are given in Table D.2 of EN 1994-1-2. These values are also given in Table 3.1.

Table 3.1 Coefficients for the determination of the temperatures of the parts of the steel decking

Concrete	Fire Exposure Time	Decking part	b_0	b_1	b_2	b_3	b_4
	(min)		(°C)	(°C).mm	(°C).mm	(°C)	(°C)
Normal weight concrete	60	Lower flange	951	-1197	-2.32	86.4	-150.7
		Web	661	-833	-2.96	537.7	-351.9
		Upper flange	340	-3269	-2.62	1148.4	-679.8
	90	Lower flange	1080	-839	-1.55	65.1	-108.1
		Web	816	-959	-2.21	464.9	-340.2
		Upper flange	618	-2786	-1.79	767.9	-472.0
	120	Lower flange	1063	-679	-1.13	46.7	-82.8
		Web	925	-949	-1.82	344.2	-267.4
		Upper flange	770	-2786	-1.67	592.6	-379.0

3.1.2 Calculation of the rib reinforcement temperature

In relation to the additional reinforcement provided in the rib (if needed), the temperature of the reinforcement can be found using the method outlined in Annex D, Section D.2 (3) of EN 1994-1-2. The steel reinforcement temperature is given by Eq. (2).

$$\theta_s = c_0 + \left(c_1 \frac{u_3}{h_2}\right) + (c_2 z) + \left(c_3 \frac{A}{L_r}\right) + (c_4 z) + \left(c_5 \frac{1}{l_3}\right) \qquad \text{Eq. (2)}$$

Where:

θ_s is the temperature of additional reinforcement in the rib (°C)
u_3 is the distance to the lower flange (see Figure 3.1) (mm)
z refer to Eq. (3)
α is the angle of the web (degrees)

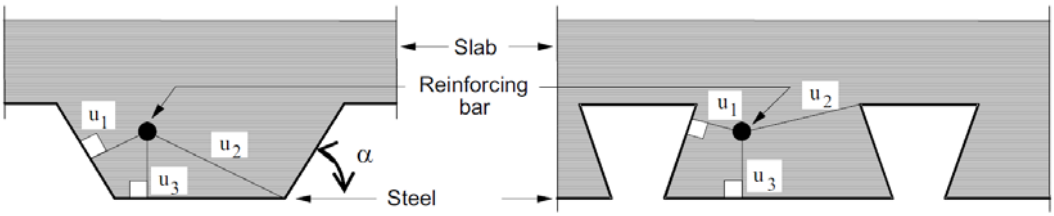

Figure 3.1 Parameter for the position of the reinforcement bars.

From Figure 3.1 the z-factor which indicates the position of the reinforcement bar is given by Eq. (3).

$$\frac{1}{z} = \frac{1}{\sqrt{u_1}} + \frac{1}{\sqrt{u_2}} + \frac{1}{\sqrt{u_3}} \qquad \text{Eq. (3)}$$

u_1, u_2 are the shortest distance of the centre of the reinforcement bar to any point of the webs of the steel sheet
u_3 is the distance of the centre of the reinforcement bar to the lower flange of the steel sheet.

For normal weight concrete, values for the factors $c_0 - c_4$ are given in Table D.2 of EN 1994-1-2. These values are also given in Table 3.2.

Table 3.2 Coefficients for the determination of the temperature of the reinforcement bars in the rib.

Concrete	Fire Exposure Time	c_0	c_1	c_2	c_3	c_4	c_5
		(°C)	(°C)	(°C.mm$^{0.5}$)	(°C.mm)	(°C/°)	(°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

3.2 Material reduction factors

The material reduction factors for galvanised steel, concrete and reinforcing steel at specific temperatures are given in Table 3.3. The material reduction factors for stainless steel are given in Table 3.4.

Table 3.3 EN 1994-1-2 Strength reduction factors

Temperature	Galvanised structural steel	Normal weight concrete	Reinforcing steel
°C	$k_{y,\theta,i}$	$k_{c,\theta}$	$k_{y,\theta,i}$
20	1.00	1.00	1.00
100	1.00	1.00	1.00
200	1.00	0.95	1.00
300	1.00	0.85	1.00
400	1.00	0.75	0.94
500	0.78	0.60	0.67
600	0.47	0.45	0.40
700	0.23	0.30	0.12
800	0.11	0.15	0.11
900	0.06	0.08	0.08
1000	0.04	0.04	0.05
1100	0.02	0.01	0.03
1200	0.00	0.00	0.00

It should be noted that the values obtained from Annex D, are for a specific time requirement. It is therefore only possible to obtain the sagging moment resistance of the section at 4 specific times; 0, 60, 90 and 120 minutes exposure. It does not allow for the calculation of the sagging moment resistance at a user specified time.

Two items will undergo property change due to an increase in temperature. These are:

- The decking (three separate values are determined for the lower flange, the web and the top flange)
- The rib reinforcement steel

Once the insulation requirement is satisfied (see Section 4), it is reasonable to assume that the temperature at the top of the slab will be cool enough that no strength reduction in these locations will occur (see Section 3.2 for reduction factors). The allowable compressive strength of the concrete and the allowable tensile stress in the top mesh reinforcement will be unchanged from ambient values.

3.2.1 Stainless steel reduction factors

The stress-strain relationship for stainless steel differs from that of carbon steel, meaning reduction factors will differ to the galvanised steel values given previously.

Reduction factors for this analysis are taken from those obtained as part of Work Package 4 by Afshan and Gardner [3].

Table 3.4 Reduction factors for grade 1.4003 ferritic stainless steel

Steel Temp	$k_{E,\Phi}$	$k_{0.2p,\theta}$	$k_{u,\theta}$	$k_{2\%,\theta}$
°C				
20	1.00	1.00	1.00	0.31
100	0.98	0.93	0.93	0.33
200	0.95	0.91	0.89	0.35
300	0.92	0.89	0.87	0.30
400	0.86	0.87	0.84	0.43
500	0.81	0.75	0.82	0.46
600	0.75	0.43	0.33	0.50
700	0.54	0.16	0.13	0.50
800	0.33	0.10	0.09	0.50
900	0.21	0.06	0.07	0.50
1000	0.09	0.04	0.05	0.50
1100	1.00	1.00	1.00	0.31
1200	0.98	0.93	0.93	0.33

The yield strength reduction of the stainless steel deck for a certain temperature may be found using table C.1 in conjunction with the equation C.1 of EN 1993-1-2 Annex C:

Equation C.1 of EN 1993-1-2 Annex C:

$$\boldsymbol{f_{y,\theta} = f_{0.2p,\theta} + k_{2\%,\theta}\left(f_{u,\theta} - f_{0.2p,\theta}\right)} \qquad \text{Eq. (4)}$$

$f_{y,\theta}$ is the yield strength of the stainless steel for a specific temperature θ.
$f_{0.2p,\theta}$ is the 2% proof strength of the stainless steel for a specific temperature θ.
$k_{2\%,\theta}$ is a reduction factor for the stainless steel for a specific temperature θ.
$f_{u,\theta}$ is the ultimate strength of the stainless steel for a specific temperature θ.

Where:

$$\boldsymbol{f_{0.2p,\theta} = k_{0.2p,\theta} f_y} \qquad \text{Eq. (5)}$$

$$f_{u,\theta} = k_{u,\theta} f_y \qquad \text{Eq. (6)}$$

The values of $k_{0.2p,\theta}$, $k_{u,\theta}$ and $k_{2\%,\theta}$ may be obtained from table C.1 of Annex C EN 1993-1-2. The values f_y and f_u may be obtained from EN 1993-1-4[4] Table 2.1. For 1.4003 cold rolled strip, with a thickness less than 6mm as is the case with composite decking:

f_y = 280 N/mm^2
f_u = 450 N/mm^2

3.3 Sagging resistance – Example calculation for Cofraplus 60

Using Eq. (1) and Table 3.1, for a 60 fire rating the temperature of the three portions of the steel decking can be calculated (upper flange, lower flange and web).

Table 3.5 Temperature and design yield strength of Cofraplus 60 decking for a 60 minute fire rating

Decking part	Temperature	f_y (G.S.)*	f_y (S.S.)*	Area	Tensile strength (G.S.)*	Tensile strength (S.S.)*
	°C	[N/mm²]	[N/mm²]	[mm²]	[kN]	[kN]
Lower flange	864	27.3	40.5	62.0	1.69	1.72
Web	783	45.8	49.5	61.0	2.79	4.55
Upper flange	719	72.7	67.1	108.0	7.85	5.23

*G.S. = Galvanised steel. S.S. = Stainless steel.

Table 3.6 Temperature and design yield strength of ferritic stainless steel decking for a 60 minute fire rating (Cofraplus 60 profile)

Material	Tensile strength	Lever arm (Measured from top of slab)	Bending moment (M_{pl})(from top of slab)
	[kN]	[m]	kNm
Lower flange	1.72	0.130	0.224
Web x 2	4.55	0.101	0.459
Upper flange	5.23	0.072	0.377
Concrete	-11.50*	0.001	-0.015
	Plastic neutral axis distance (top of slab)=		2.6mm
		Σ M_{pl}=	1.045 kNm
		Σ M_{pl}=	5.02 kNm/m

*Negative value indicate material in compression

By repeating the calculation outlined above the sagging moment resistance of the slab can be calculated for different fire ratings and different deck shapes. The addition of rib reinforcement can be accounted for by adding an additional tensile strength term to Table 3.6. Eq. (2) can be used to find the bar temperature, with strength reduction factors from Table 3.2, for reinforcing steel.

The results of the numerical analysis are summarized in Section 3.4.

3.4 Summary of results and discussion

Values were computed for the sagging moment resistance of the various deck profiles and reinforcement layouts as described previously. The results are presented in Table 3.7:

Table 3.7 Sagging moment resistance for trapezoidal decking profiles. Total concrete depth =130mm. Deck thickness =1mm

Decking	Fire Exposure Time	Decking Type	Deck Only	Deck + Mesh	Deck, Mesh + Rib Rebar
			kNm/m	kNm/m	kNm/m
Cofraplus 60	0	Galvanised Steel	41.71	42.01	47.65
		Ferritic Steel	34.28	35.23	41.75
	60	Galvanised Steel	6.37	8.57	17.93
		Ferritic Steel	5.02	7.30	16.82
	90	Galvanised Steel	2.91	5.30	14.16
		Ferritic Steel	3.20	5.57	14.42
	120	Galvanised Steel	2.09	4.52	10.44
		Ferritic Steel	2.42	4.83	10.73
Cofraplus 77	0	Galvanised Steel	43.48	43.44	44.86
		Ferritic Steel	36.00	36.95	39.47
	60	Galvanised Steel	9.64	11.56	17.13
		Ferritic Steel	7.61	9.65	15.51
	90	Galvanised Steel	3.68	6.00	12.24
		Ferritic Steel	3.72	6.05	12.78
	120	Galvanised Steel	2.40	4.80	9.75
		Ferritic Steel	2.68	5.06	9.99

Table 3.8 Sagging moment resistance for re-entrant decking profiles. Total concrete depth =130mm. Deck thickness =1mm

Decking	Fire Exposure Time	Decking Type	Deck Only	Deck + Mesh	Deck, Mesh + Rib Rebar
			kNm/m	kNm/m	kNm/m
Cofrastra 40	0	Galvanised Steel	63.12	62.78	73.14
		Ferritic Steel	52.00	53.33	65.24
	60	Galvanised Steel	21.84	23.32	38.60
		Ferritic Steel	23.50	24.87	39.85
	90	Galvanised Steel	8.93	11.05	28.07
		Ferritic Steel	7.98	10.15	27.32
	120	Galvanised Steel	4.43	6.77	23.99
		Ferritic Steel	4.17	6.52	23.78
Cofrastra 56	0	Galvanised Steel	64.94	64.38	69.93
		Ferritic Steel	53.71	55.91	63.14
	60	Galvanised Steel	23.96	25.27	35.89
		Ferritic Steel	24.37	25.61	36.08
	90	Galvanised Steel	11.42	13.36	25.72
		Ferritic Steel	10.44	12.43	24.92
	120	Galvanised Steel	5.57	7.83	21.06
		Ferritic Steel	5.21	7.50	21.25

For the Cofraplus 60 deck profile, the percentage difference in sagging moment resistance of ferritic stainless steel decking to that of the galvanised steel deck are presented in Table 3.9.

Table 3.9 Percentage difference in sagging moment resistance for Ferritic Stainless Steel Decking to that of Galvanised Steel Decking (Cofraplus 60 profile).

Fire Exposure Time	Deck Only	Deck + Mesh	Deck + Mesh + Rib Reinforcement
	kNm/m	kNm/m	kNm/m
0	-18%*	-16%*	-12%*
60	-21%*	-15%*	-6%*
90	10%	5%	2%
120	15%	7%	3%

* Negative values indicate that designs utilising galvanised steel decking have a greater sagging resistance compared to stainless steel decking.

Table 3.9 shows that the resistance of the slabs constructed using ferritic stainless steel decking is greater at greater exposure times, but is less at ambient temperature and at 60 minutes exposure. The primary reason galvanised steel decking is stronger at ambient temperature is due to the larger f_y value assumed in the numerical analysis (350 N/mm^2) compared to f_y stainless steel (280N/mm^2). This superior strength is

eroded at higher fire resistance periods, since the stainless steel retains more of its original strength at higher temperatures.

In all cases the rib reinforcement used for the analysis was 10 mm diameter with f_y = 500 N/mm^2. The reinforcement position was set 10 mm above the top flange of the deck, at the centre line of the rib. It was observed that the Cofrastra 40 deck for a fire rating of 60 minutes resulted in a higher moment resistance compared with the Cofrastra 56 profile. This appears unusual at first as the Cofrastra 56 has a deeper deck profile. The higher moment resistance is due to the distance from the top surface of the concrete to the top flange of the deck (and in turn the rib reinforcement 10 mm above the top flange) reducing as the deck depth increases (Total concrete depth is constant at 130 mm).

The possibility of reducing the bar diameter for slabs with large exposure times was investigated, since the slabs constructed with stainless steel decking generally had greater resistance than those constructed with galvanised steel decking. The investigation showed that for rib reinforcement sizes between 8 mm and 12 mm inclusive, it was not possible to reduce the bar size in all cases, as the increase in force provided by the decking was not enough to compensate for the reduction in force in the bars.

4 INSULATION (CRITERION I)

A composite slab performing a separating function must ensure that the temperature rise above the slab is small enough to prevent further spread of fire at other floors. This is known as the insulation criterion.

The insulation criterion (I) may be determined using the approach outlined in EN 1994-1-2 Annex D. Section D.1 states that the fire resistance with respect to insulation (I) for both the average temperature rise (=140ºC) and the maximum temperature rise (=180ºC), may be determined according to Eq. (7).

$$\boldsymbol{t_{\mathrm{i}} = a_0 + a_1 h_1 + a_2 \phi + a_3 \frac{A}{L_{\mathrm{r}}} + a_4 \frac{1}{l_3} + a_5 \frac{A}{L_{\mathrm{r}}} \frac{1}{l_3}} \qquad \text{Eq. (7)}$$

Where:

t_{i} is the fire resistance with respect to thermal insulation (min)

The definition of all the other variables is given in Section 3.1.

$$\boldsymbol{\frac{A}{L_{\mathrm{r}}} = \frac{h_2 \left(\frac{l_1 + l_2}{2}\right)}{l_2 + 2\sqrt{{h_2}^2 + \left(\frac{l_1 - l_2}{2}\right)^2}}} \qquad \text{Eq. (8)}$$

$$\phi = \left(\frac{\sqrt{{h_2}^2 + \left(l_3 + \frac{l_1 - l_2}{2}\right)^2} - \sqrt{{h_2}^2 + \left(\frac{l_1 - l_2}{2}\right)^2}}{l_3} \right) \qquad \text{Eq. (9)}$$

The factors a_1 - a_5 are given in Table D.1 of EN 1994-1-2. For normal weight concrete these values are summarised in Table 4.1.

Table 4.1 Coefficients for determination of the fire resistance with respect to thermal insulation

	a_0	a_1	a_2	a_3	a_4	a_5
	(min)	(min/mm)	(min)	(min/mm)	(min)	(min/mm)
Normal weight concrete	-28.8	1.55	-12.6	0.33	-735	48.0

Using the values given in Table 4.1, Eq. (7) may be revised as follows:

$$t_i = -28.8 + 1.55h_1 - 12.6\phi + 0.33\frac{A}{L_r} - 735\frac{1}{l_3} + 48.0\frac{A}{L_r}\frac{1}{l_3} \qquad \text{Eq. (10)}$$

Using Eq. (10) the period of time where the insulation requirement remains satisfactory is calculated. The results are summarized in Table 4.2 for the four profiles included in the study.

Table 4.2 Insulation time periods of trapezoidal decking

	h_1	h_2	l_1	l_2	l_3	Φ	A/L_r	t_i
	(mm)	(mm)	(mm)	(mm)	(mm)	(-)	(mm)	(min)
Trapezoidal profiles								
Cofraplus 60	72	58	100	62	108	0.73	25.5	**87**
Cofraplus 77	53	77	110	70	82	0.45	30.3	**66**
Re-entrant profiles								
Cofraplus 40	90	40	90	124	60	0.14	23.4	**123**
Cofraplus 56	74	56	110	137	40	0.04	30.4	**113**

Table 4.2 shows that the insulation requirement for the Cofraplus 60 and Cofraplus 77 profiles with a total slab depth of 130 mm will not be satisfied for a 90 and 120 minute fire rating. For the Cofrastra 56 profile, with a total slab depth of 130 mm, the insulation requirement for a 120 minute fire rating will not be satisfied. For these slab arrangements, the limiting criterion will be the insulation requirement.

5 CONCLUSION

For ferritic stainless steel decking, the strength of the decking at ambient temperature was taken as $f_{0.2p}$=280 N/mm^2. For galvanised steel decking the strength of the decking at ambient temperature was taken as f_y=350 N/mm^2 The lower initial strength of the ferritic decking results in a lower sagging bending moment resistance at ambient temperature.

Stainless steel has superior retention of strength at higher temperatures than galvanised steel. This is reflected in the increased sagging resistance of a slab constructed with stainless steel decking at elevated temperature, compared to slabs constructed with galvanised steel decking. However the differences observed are small.

In many cases, but particularly for trapezoidal profiles at long exposure times, the insulation criterion rather than the strength criterion governs the slab depth required. In these cases the superior strength retention of stainless steel at elevated temperature has no effect on the design, since the strength of the materials is not fully utilised.

6 REFERENCES

1 EN 1994-1-1:2004 Eurocode 4. Design of composite steel and concrete structures. General rules and rules for buildings, European committee for standardization (CEN).

2 EN 1994-1-2:2005 Eurocode 4. Design of composite steel and concrete structures. General rules, structural fire design, European committee for standardization (CEN).

3 S. Afshan and L. Gardner, Structural Applications of Ferritic Stainless Steels. WP 4 Structural Fire Resistance (SCI), June 2013

4 EN 1993-1-4:2006, Eurocode 3. Design of steel structures. Part 1-4 General rules, supplementary rules for stainless steels, European committee for standardization (CEN).

APPENDIX A BENDING RESISTANCE OF DECKING WITH VARIOUS ARRANGEMENTS OF REINFORCEMENT

6.1.2 Cofraplus 60

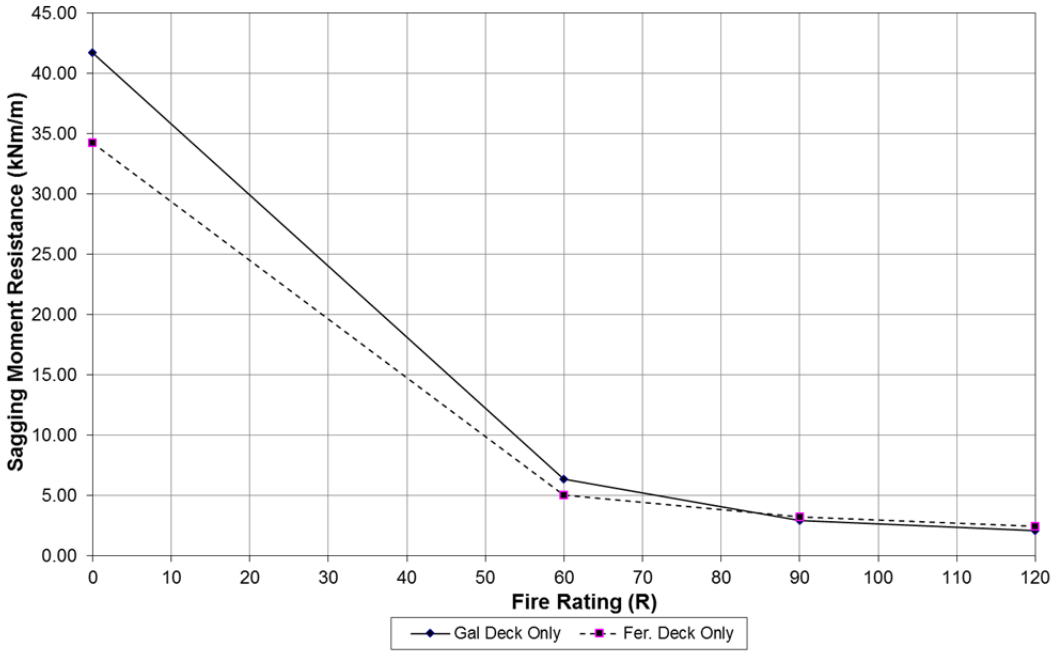

Figure 6.1 Sagging moment resistance of Cofraplus 60 (deck only) for a range of fire resistance values.

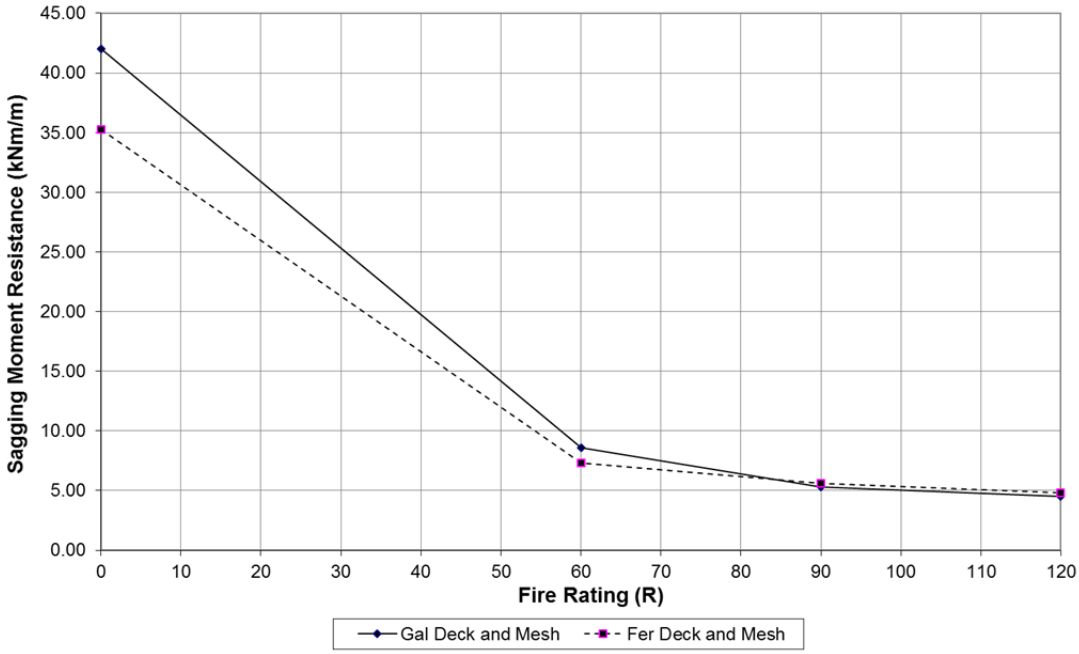

Figure 6.2 Sagging moment resistance of Cofraplus 60 (deck + top mesh in slab) for a range of fire resistance values.

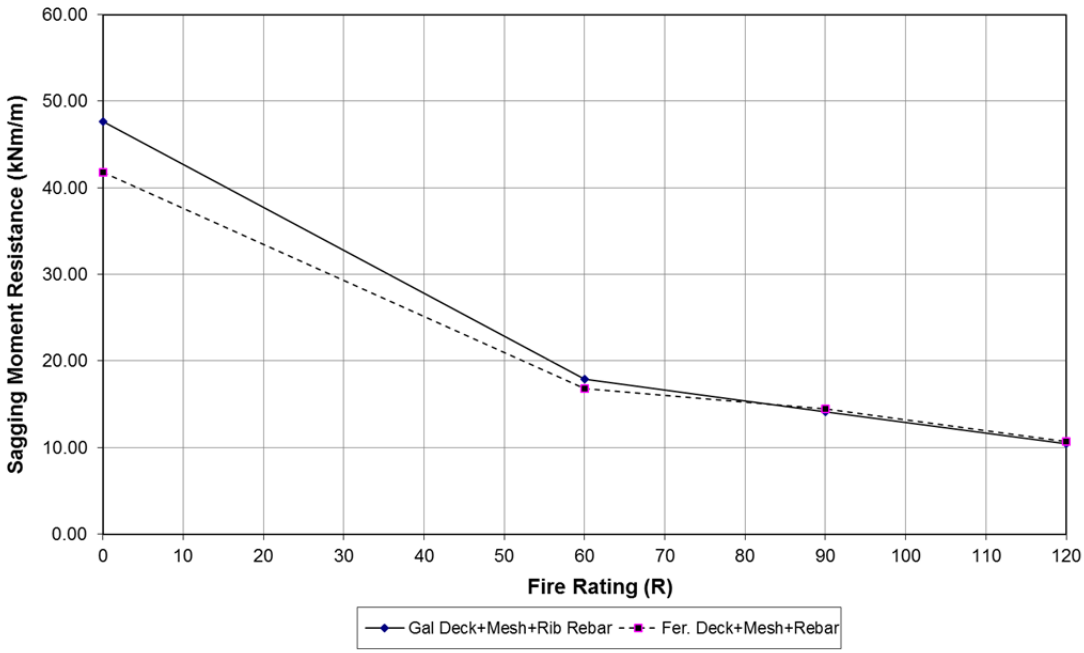

Figure 6.3 Sagging moment resistance of Cofraplus 60 (deck, top mesh and rib reinforcement in slab) for a range of fire resistance values.

6.1.3 Cofraplus 77

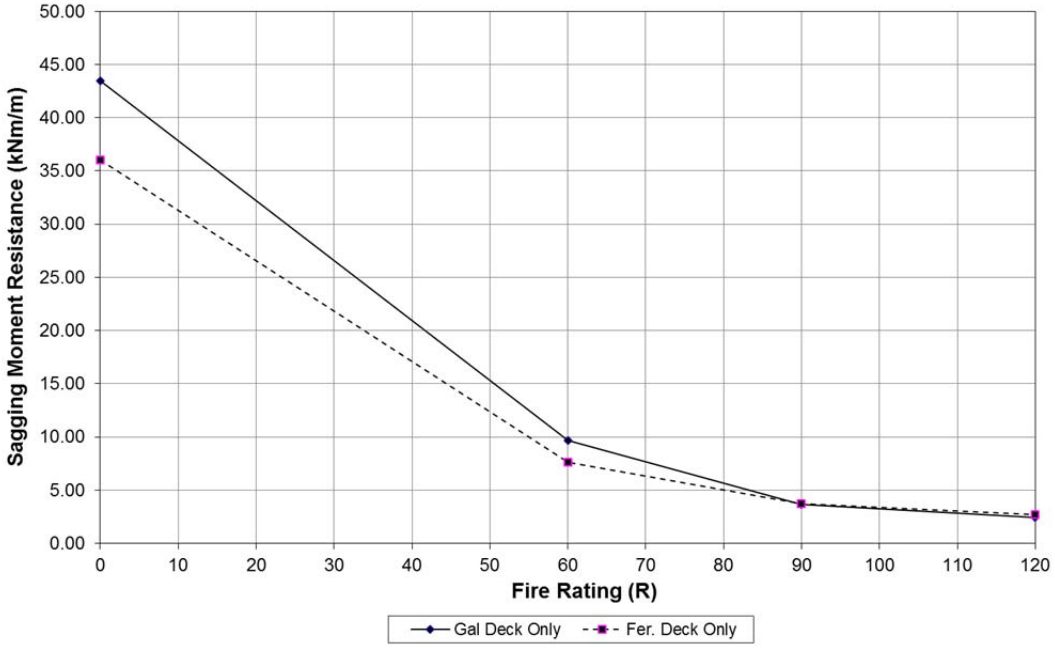

Figure 6.4 Sagging moment resistance of Cofraplus 77 (deck only) for a range of fire resistance values.

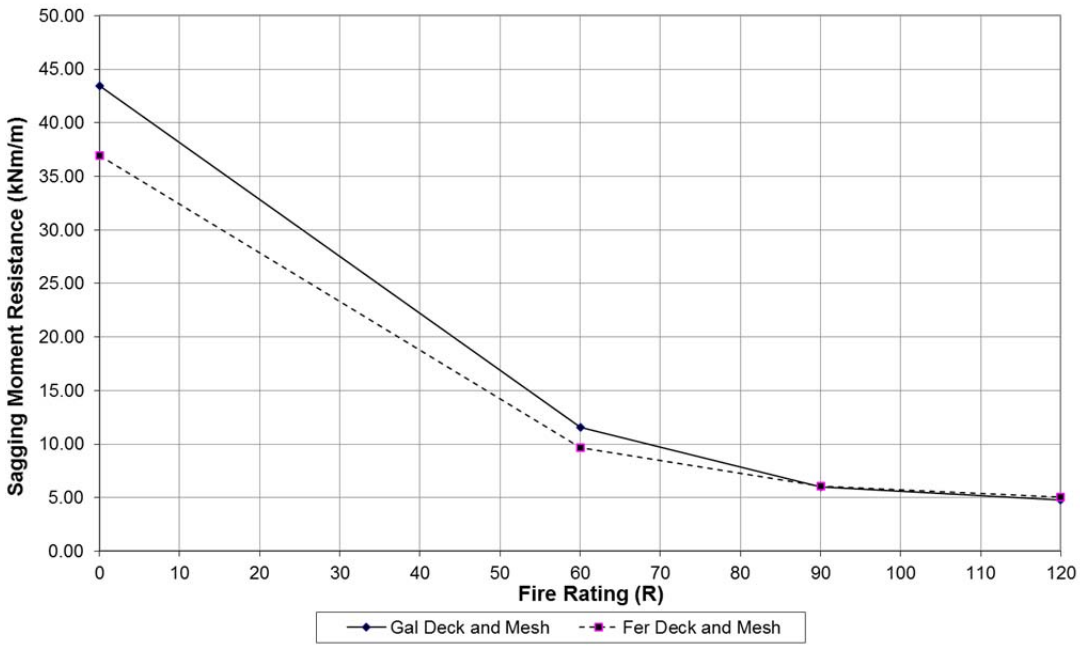

Figure 6.5 Sagging moment resistance of Cofraplus 77 (deck + top mesh in slab) for a range of fire resistance values.

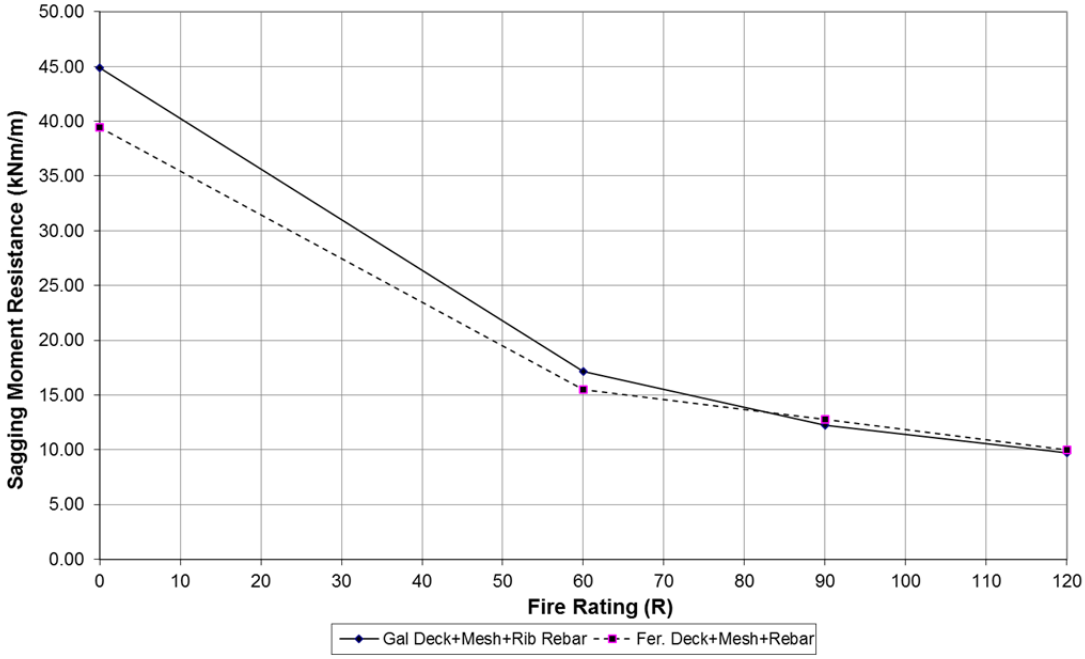

Figure 6.6 Sagging moment resistance of Cofraplus 77 (deck, top mesh and rib reinforcement in slab) for a range of fire resistance values.

6.1.4 Cofrastra 40

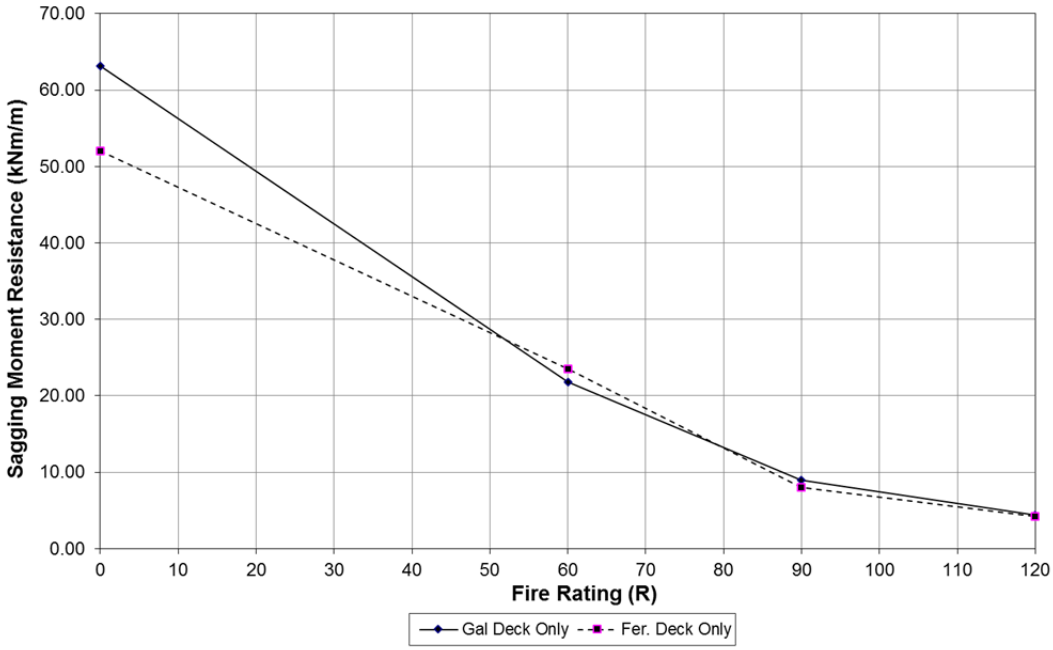

Figure 6.7 Sagging moment resistance of Cofrastra 40 (deck only) for a range of fire resistance values.

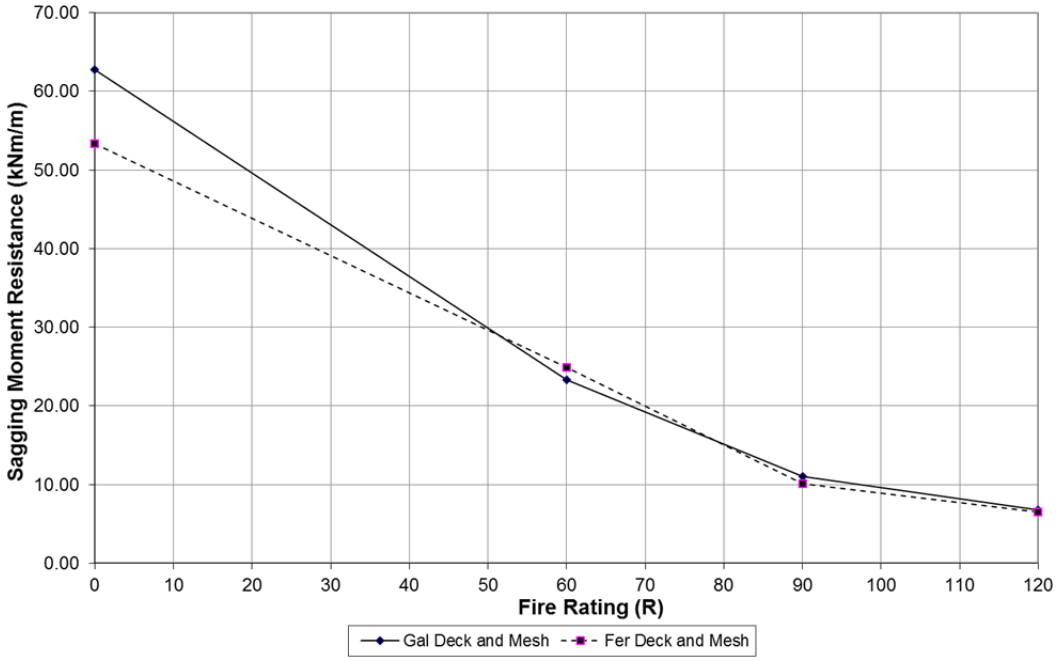

Figure 6.8 Sagging moment resistance of Cofrastra 40 (deck + top mesh in slab) for a range of fire resistance values.

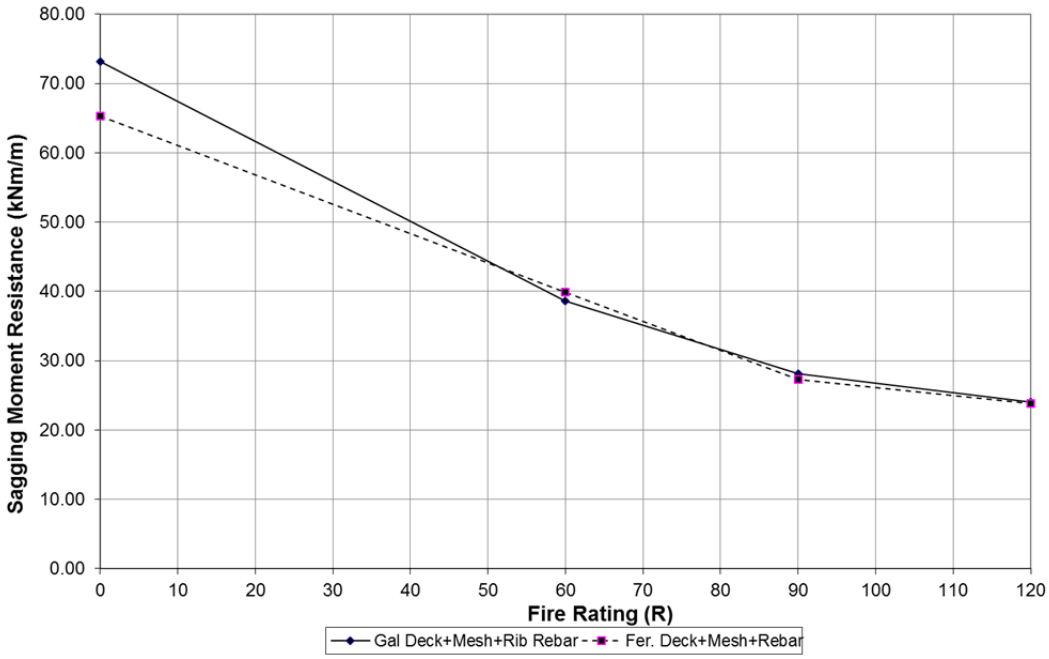

Figure 6.9 Sagging moment resistance of Cofrastra 40 (deck, top mesh and rib reinforcement in slab) for a range of fire resistance values.

6.1.5 Cofrastra 56

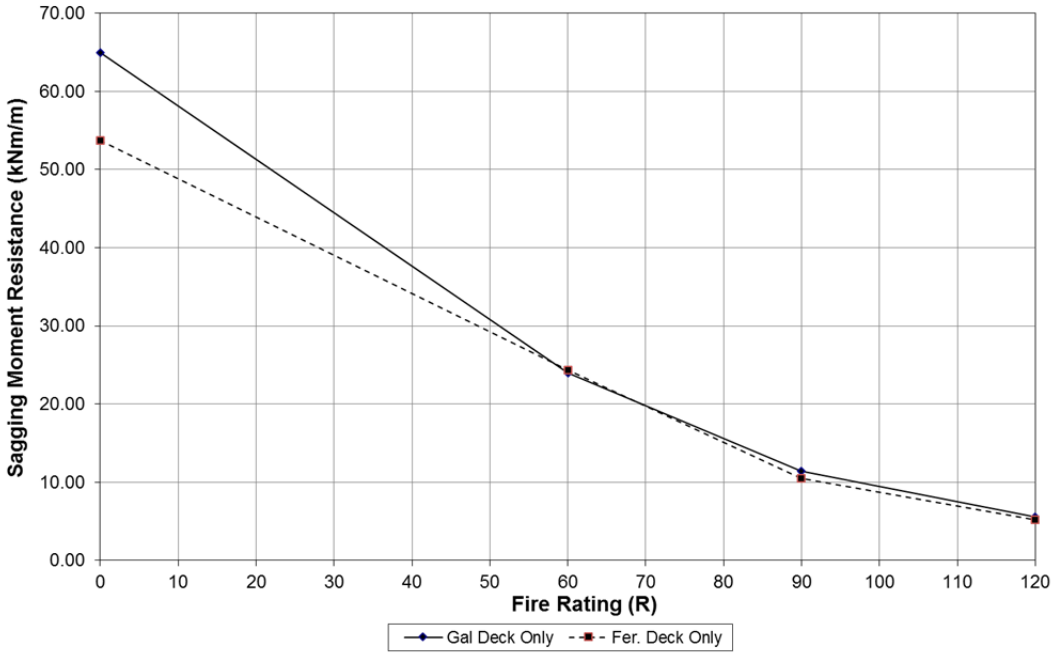

Figure 6.10 Sagging moment resistance of Cofrastra 56 (deck only) for a range of fire resistance values.

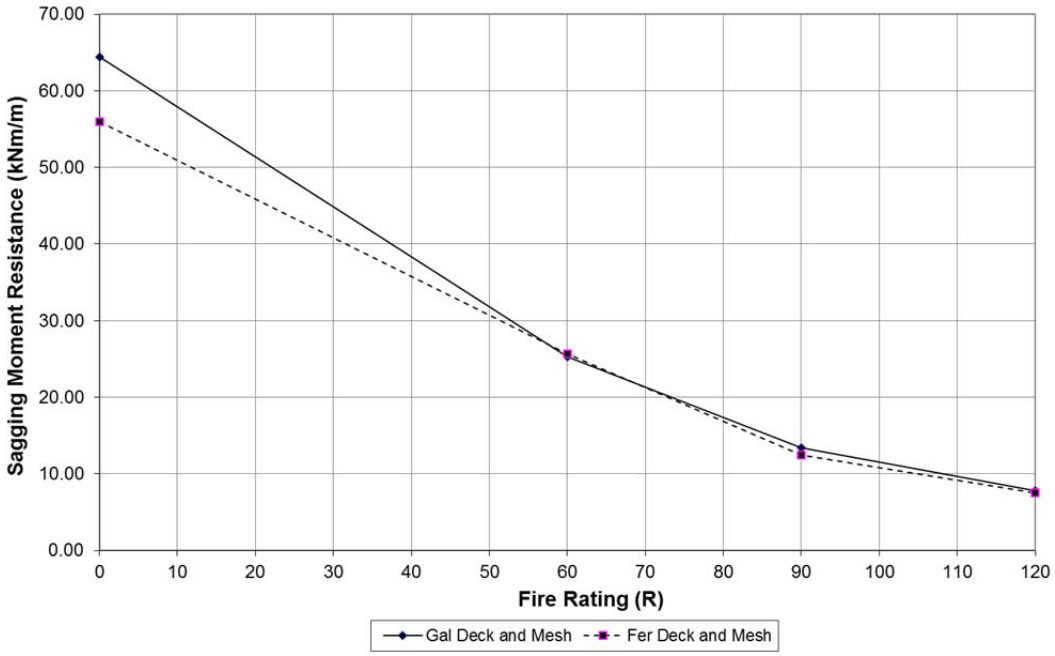

Figure 6.11 Sagging moment resistance of Cofrastra 56 (deck + top mesh in slab) for a range of fire resistance values.

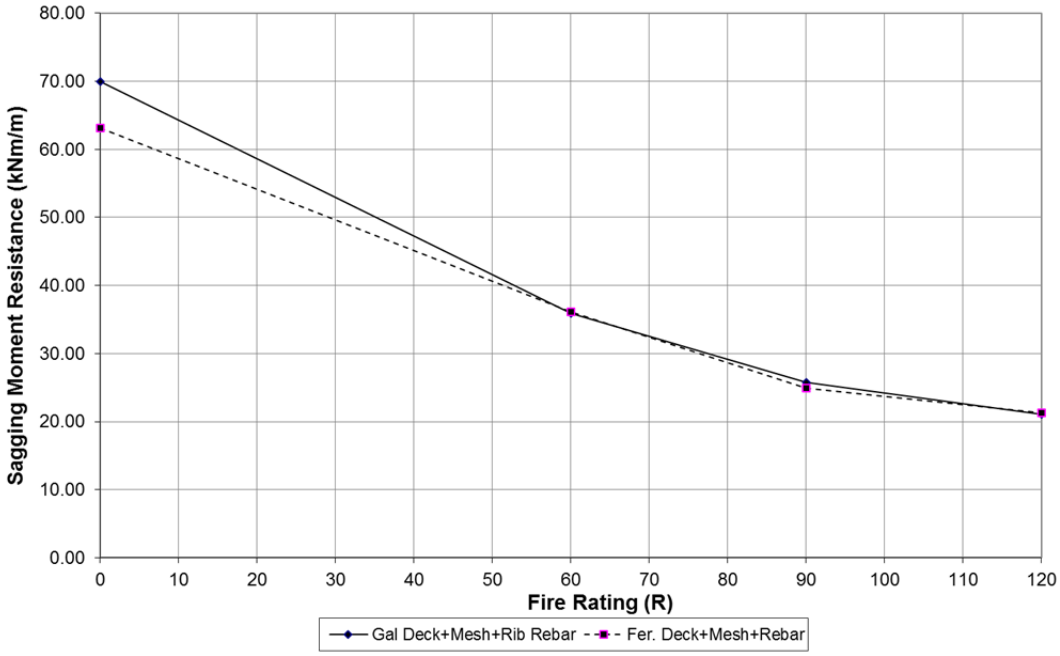

Figure 6.12 Sagging moment resistance of Cofrastra 56 (deck, top mesh and rib reinforcement in slab) for a range of fire resistance values.