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Light Gauge Structural Panel for Composite Flooring

Plateau porteur en acier formé à froid pour planchers mixtes

Leichtbausystem für Verbunddeckenkonstruktionen

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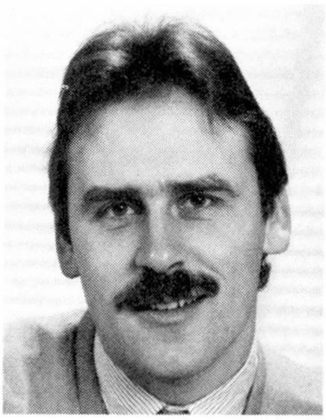
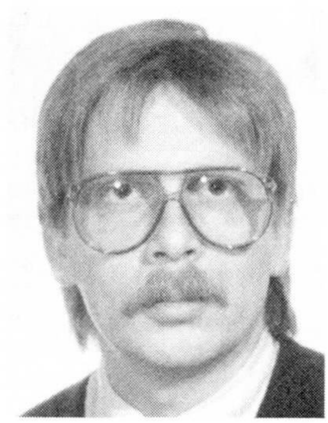


Jonathan Ramsden has been involved in research into composite flooring at the Swedish Institute of Steel Construction since 1981. Having completed his doctors' degree he is currently employed by a Swedish steel fabricator as research and development manager.



Anders Segerlind, civil engineer, was employed for the first 4 years after his degree by a firm of consulting engineers. Since 1982 he has been employed by Dobel AB, manufacturers of coated light gauge metal products for the Building Industry as development engineer.

SUMMARY
This paper describes the background to and the testing of a new Swedish composite flooring system, which offers up to 7,5 m span, complete composite action, 2 hours fire resistance without additional reinforcement, and a finished soffit ready for painting. The floor has been tested on two sites in Stockholm and has created considerable interest in the Swedish Construction Industry.

RÉSUMÉ
Cet article relate le développement et les essais d'un nouveau système de plancher mixte suédois qui permet de franchir des portées jusqu'à 7,5 m, dont l'interaction acier-béton est complète, dont la résistance au feu sans armature supplémentaire est de 2 heures et dont la face inférieure peut être facilement peinte. Le plancher a été testé sur deux chantiers à Stockholm et a suscité un intérêt considérable dans l'industrie suédoise de la construction.

ZUSAMMENFASSUNG
Der Artikel beschreibt Hintergründe und Versuche zur Entwicklung eines neuen schwedischen Verbunddeckensystems, das Spannweiten bis zu 7,5 m zulässt. Das System ermöglicht vollständige Verbundwirkung, 2-stündigen Feuerwiderstand ohne zusätzliche Bewehrung und eine anstrichfertige Unterseite. Das System wurde auf zwei Baustellen in Stockholm erprobt und hat in der schwedischen Bauindustrie bedeutendes Interesse gefunden.

1 BACKGROUND

Composite flooring is often associated with steel frames. Steel frames in Sweden are, due to the abscence of an alternative flooring system, often constructed with a pre-fab concrete slab floor, spanning between 6 and 12 metres. Secondary beams are therefore seldom used. Floor thicknesses in Sweden vary between 150 and 300 mm, due to limitations on construction height imposed by Town Planning Leglisation. Limited floor to floor height and longspan flooring then leaves little room for a traditional primary-secondary beam system in the floor structure.

Unreinforced composite floors generally have low resistance to fire. Being as composite beams are seldom used in Sweden, the floor slab must be capable of withstanding Code fire requirements without relying upon the advantages gained by using composite beams. The composite floor must be able to fulfil the A 60 class, A = incombustible, 60 = fire resistance of 60 mins. (ISO 834) approved for pre-fab composite slabs over a span of 6 metres.

A thin steel sheet subjected to fire will suffer an extremely rapid increase of temperature unless the energy input can be diverted or absorbed by some other material. Concrete is an excellent thermal energy absorbent due to its high specific heat capacity and may therefore be used to minimize the temperature in the sheet steel panel by embedding as much of the sheet panel as possible into the concrete topping. The sheet steel will then serve as tensile reinforcement even at elevated temperatures. It is of course possible to introduce extra reinforcement as a measure by which to increase the fire rating of a composite floor. This is, however, a step back and the steel decking then tends to become an expensive way of providing formwork for a traditional in-situ concrete floor.

2 PROTOTYPE NO. 1

The three conditions mentioned previously, i.e. long span, finished ceiling surface and high resistance to fire, must be fulfilled if a composite flooring system is to succeed on the Swedish market. With these criteria in mind a sheet panel, denoted here as prototype no. 1, was developed at the Swedish Institute of Steel Construction, and tested at the Royal Institute of Technology, Dept. of Steel Structures.

2.1 Basic Concepts

The standard basic dimension for modular design in Sweden is 100 mm but most designs are based on multiples of 600 mm. The visible lined pattern created by the underside of a proposed sheet panel must combine with a multiple of 600 mm, which means that a flat bottom flange of 300 mm should be a reasonable compromise between economic sheet use, acceptable deflection and modular compatibility.

The edge stiffener on the upper flange not only gives a considerable increase in pre-composite load capacity compared to that of an unstiffened flange, but even eases the fitting together of the panels on site. No screws are necessary in order to create a safe working platform. The panels are held together just above the bottom flange by a stitch fold joint. The stiffened upper flange of the panel effectively prohibits vertical separation between the steel panel and the concrete topping, hardened concrete that flows between the panels and acts as a shear connector, ensuring composite action.

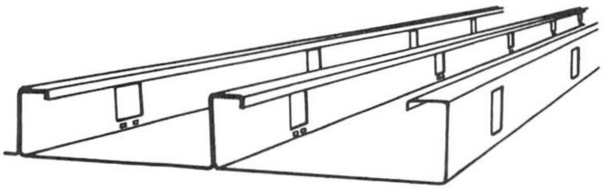


Fig 1 Prototype no. 1

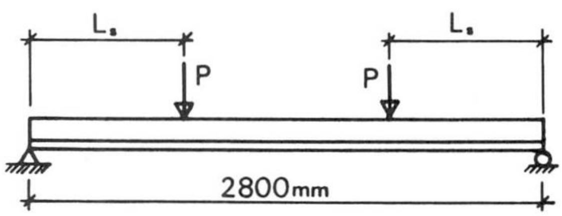


Fig 2 Sketch of the test set-up

2.2 Experimental Investigation

An experimental investigation into the function and strength of prototype no. 1 was carried out. The test specimens were 3x300 mm wide and 3000 mm long with either a 120 or 160 mm concrete topping, grade K25 (nominal cube strength 25 N/mm^2). The investigation was conducted in accordance with the recommendations in the Swedish Code for Light Gauge Metal Structures (2). The method is based on the results obtained through experimental investigation at Iowa University, Iowa, USA and is characterized by slip between the metal sheet and the concrete at ultimate loading.

2.3 Results

The results from testing show that the sheet panels performed as expected, that is to say performed in a similar way to that of a compatible reinforced concrete slab. The test specimens did however bring to light two major defects.

The most serious defect was that the panels leaked during the pouring of the concrete, leaving drops of cement paste that had adhered to the underside of the panels (the visible surface of the panels). This defect is of an aesthetic nature and has nothing to do with structural mechanics, but is of great importance if such a flooring system is to offer a finished ceiling surface.

The second defect was caused by the large proportion of the hole in the web in relation to the web itself. The hole substantially weakens the panel, especially when approaching yield loads, which is clearly demonstrated in fig 3.

The combination of these two defects indicated that the basic concept was good, but that the hole should be replaced by some other medium in order to counteract the shear forces in the panel web/concrete interface.

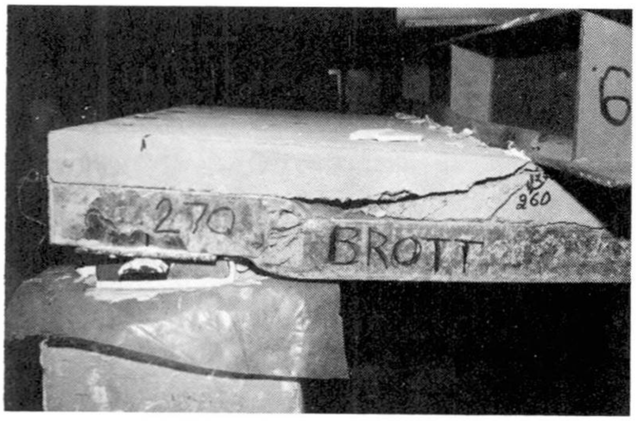


Fig 3 Panel deformation at ultimate load

3 PROTOTYPE NO. 2

The results achieved from the testing of prototype no. 1 awoke the interest of Dobel AB of Borlänge, Sweden whereby a joint project was started, based on a revised version of Prototype no. 1, hereafter referred to as Prototype no. 2.

3.1 Basic Concepts

The only way to ensure that no seepage occurs between the panels is to refrain from perforating them. If the webs are provided with embossments instead of holes, the embossed web surfaces will combine to act as shear connectors, being as the top flange overlap ensures that the adjoining webs are completely flush and are embedded in the concrete topping. The embossed webs should then act in a similar fashion to that of ribbed reinforcement bars. The web embossment pattern chosen consists of three rows of indentations 12 mm x 6 mm, 3 mm high and 6 mm between each row.

The stich fold joint was replaced by self-drilling screws, which can be quickly and effectively fitted by means of a special adapter fitted to a variable-speed electric hand-drill. The fitting of self-drilling screws requires a horizontal working surface which was achieved by the introduction of a "shelf" 20 mm wide, 25 mm above the bottom flange. The revised concept with embossed web, shelf and self-drilling screws, became prototype no. 2 and is shown in fig 4.

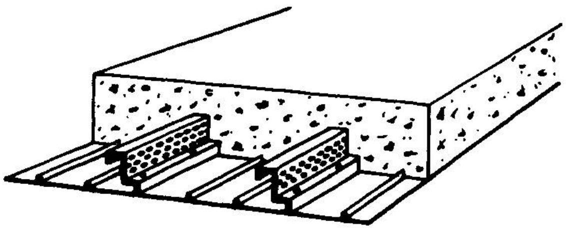


Fig 4 Sketch of prototype no. 2

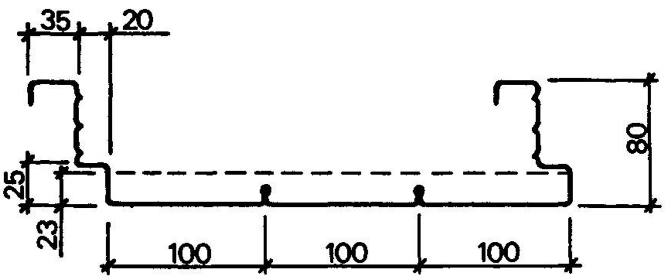


Fig 5 Section data

4 SHEAR LOAD CAPACITY

The shear load capacity of composite flooring with a regular, continuous pattern of embossment is determined by means of formula (4.1) which is stipulated in The Swedish Code for Light Gauge Metal Structures, StBK-N5 (2) and explained in (3).

$$V_d = \frac{0.8\ bd}{\gamma_n}\left(\frac{m\rho d}{L_s} + k\ f_{ct}\right) \qquad (4.1)$$

4.1 Test details, frame

The test frame was basically the same as that shown in fig 2 and consists of a simply supported composite slab, 900 mm (three panels) wide subjected to two equal knife-edge loads at a distance of L_s from each support respectively. Deflection was measured in the centre of each slab on each side. Two deflection gauges were placed at each end of the slab in order to measure the amount of slip between the concrete slab and the steel panels.

The test series consisted of the following seven tests, each test being repeated three times to check consistency:

Table 1 The test series, shear load capacity

Denomination	L (mm)	H (mm)	L (mm)	d (mm)	Number of tests	L/H
H 130-390	1650	130	390	107	3	3
H 130-780	2300	130	780	107	3	6
H 130-1170	3400	130	1170	107	3	9
H 200-600	2300	200	600	177	3	3
H 200-1200	3400	200	1200	177	3	6
H 300-900	3400	300	900	277	3	3
H 300-1800	4700	300	1800	277	3	6

4.2 Test results

The results from the tests were plotted in the design diagram in the Swedish Code (2) (The Porter-Schuster semi-empirical design method).

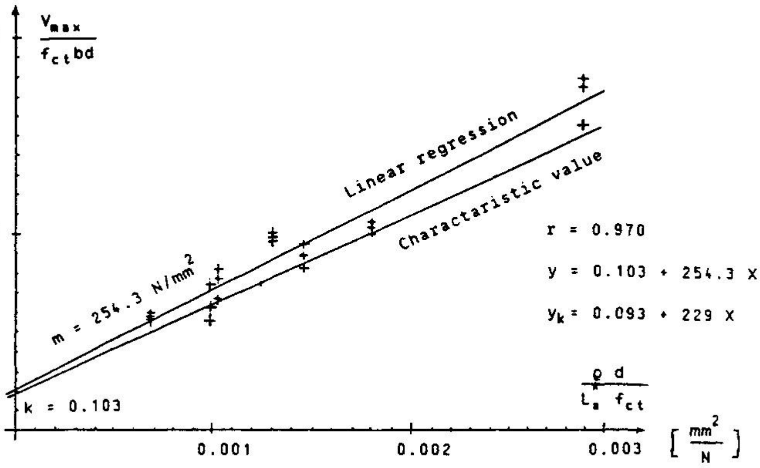


Fig 6 The plotted regression line giving the coefficients m and k

The regression line determined in fig 6 fits the equation

$$y = 0.103 + 254.3\,x \tag{4.2}$$

The regression line for design is determined by a 10 % reduction in the values in equation (4.2), being as 21 tests had been carried out. The equation for the design curve is then

$$y = 0.093 + 229\,x \tag{4.3}$$
$$k = 0.093 \tag{4.4}$$
$$m = 229\ \mathrm{N/mm^2} \tag{4.5}$$

for the composite panel BLK 300/80

The equation for the calculation of the shear capacity of composite flooring using the panel BLK 300/80 is, after the substitution of $m = 229\ \mathrm{N/mm^2}$, $k = 0.093$ and $\rho = A_s / bd$

$$V_d = \frac{0.8\,d}{\gamma_n}\left(229\,\frac{A_s}{L_s} + 0.093\,b\,f_{ct}\right) \tag{4.6}$$

4.3 Bending moment capacity

The bending moment capacity of fully composite flooring with profiled steel sheeting is given in StBK-N5 section 34:265 (2). Formula (4.7) applies to under-reinforced slabs.

$$M_d = A_s f_{ty} d \left(1 - \frac{\varrho}{2} \frac{f_{ty}}{f_{cc}}\right) \tag{4.7}$$

and formula (4.8) to over-reinforced slabs

$$M_d = f_{cc} b d^2 \eta (1 - 0.5 \eta) \tag{4.8}$$

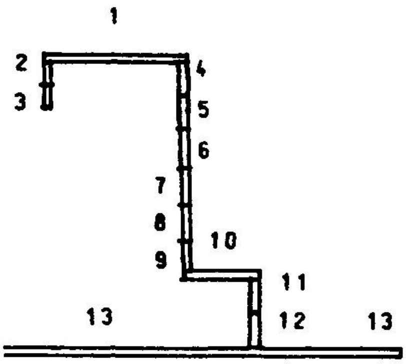
where

$$\eta = 0.5 \left(\sqrt{(4\beta + \beta^2)} - \beta \right) \quad \text{and} \quad \beta = \frac{E_s A_s \varepsilon_{cu}}{f_{cc} b d} \tag{4.9}$$

The slab is classified as under-reinforced when $\varrho < \varrho_b$ and as over-reinforced when $\varrho > \varrho_b$ where

$$\varrho_b = \frac{h_s}{d} \frac{f_{cc}}{f_{ty}} \frac{1}{(1 + f_{ty} / 700)} \quad \text{and} \quad \varrho = \frac{A_s}{b d} \tag{4.10}$$

When producing theoretical design data for the panel BLK 300/80 it was found by Anders Segerlind (4) that formula 4.4 did not give sufficiently correct design values for the under-reinforced slab, presumably due to the fact that the formula is directly derived from equivalent reinforced concrete formulae, which do not take the height and the stiffness of the sheet web into considera- tion. A new design model was established which divided the panel section into a number of sub-sections, each of which was theoretically allowed to plasticise in turn until a balance in stresses was obtained in the compression (concrete) and tension (steel) zones. This model is shown in fig 7.



$$F_s = \Sigma A_i \sigma_i$$
$$F_c = x B \sigma_c$$
$$F_s = F_c$$
$$\varepsilon_{cu} = 0.35\%$$

over-reinforced : $F_s > F_c$

under-reinforced : $F_s < F_c$

$$d' = \frac{\Sigma A_i \sigma_i}{F_s t}$$
$$M_d = (H - d' - 0.4X) F_s$$

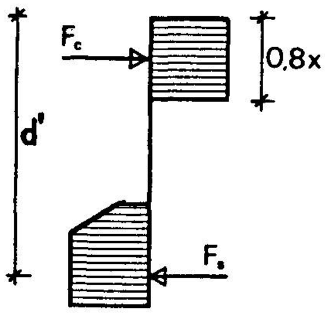


Fig 7 The sub-divided panel for the calculation of the moment capacity of the under-reinforced panel section.

This new design model gave considerably improved design values which correlate well with the test results obtained, see Table 2.

4.4 Theoretical deflection of the cracked slab

The handbook "BYGG" part T section A24:35 gives the deflection of a simply supported slab subjected to two downward point loads that are of an equal dis- tance, Ls, from each support as

$$y = \frac{P L_s (3L^2 - 4L_s^2)}{24 EI} \tag{4.11}$$

The moment at midspan is

$$M = P L_s \tag{4.12}$$

and the curvature of the slab

$$\frac{1}{r} = -\frac{d^2y}{dx^2} = \frac{M}{EI} \tag{4.13}$$

the substitution of (4.13) and (4.12) into (4.11) give

$$y = \frac{1}{r}\,\frac{(3L^2 - 4L_s^2)}{24} \tag{4.14}$$

If the effects of creep are ignored then the curvature of a concrete slab is given in (1) p. 34 as

$$\frac{1}{r} = \frac{\sigma_s}{E_s\, d(1-x/d)} \tag{4.15}$$

This straight line is shown in fig 8.

4.5 Theoretical deflection of the uncracked slab

The curvature of the slab is given in (4.15). Where the height of the compression zone, x, is

$$x = \frac{(\alpha-1)A_s\, d + 0.5bH^2}{(\alpha-1)A_s + bH} \tag{4.16}$$

and the second moment of area of the uncracked composite section is given in (1) p 36 as

$$I_{id} = \frac{bH^3}{12} + bH(x-H/2)^2 + (\alpha-1)A_s\,(d-x)^2 + \alpha I_s \tag{4.17}$$

The flexural stress in the concrete as shown in (1) p. 36 is

$$\sigma_{cb} = \frac{M}{I_{id}}(H-x) \tag{4.18}$$

$$\frac{1}{r} = \frac{\sigma_{cb}}{E_c\, d(1-x/d)} \tag{4.19}$$

which may be substituted into (4.14). An example of the resulting straight line is shown in fig 8.

4.6 Calculation of the load at which the first crack occurs

Section 4.4.2 in (1) gives

$$M_{max,test} = PL_s + qL^2/8 \tag{4.20}$$

and 4.18 rewritten gives

$$M_{max,material} = \frac{\sigma_{cb}\, I_{id}}{H-x} \qquad M_{max,test} = M_{max,material} \tag{4.21}$$

which gives

$$P_{crack} = \left[\frac{\sigma_{cb}\, I_{id}}{H-x} - qL^2/8\right]\frac{1}{L_s} \tag{4.22}$$

which is shown in fig 8.

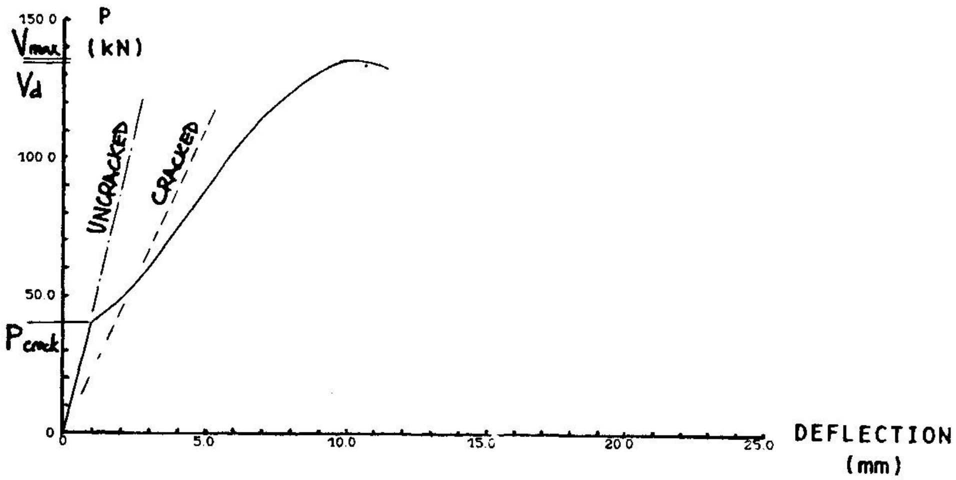


Fig 8 An example of the load deflection curve from testing

4.7 Conclusion

The regression line for the calculation of the coefficients k and m in formula (4.1) shows excellent consistency and may therefore be taken as a reliable basis for the calculation of the shear capacity of a concrete slab using the Dobel BLK 300/80 sheet steel composite panel. The ratio between V_{max} (the maximum load values during testing) and V_d (the design values for shear capacity using formula (4.6)) is shown in table 2 below where the highest value is 0.755, giving a minimum of a 32 % safety margin in design before the introduction of material and load reduction factors. The safety margin in design is then in the order of 35-45 %, which is more than sufficient, perhaps even conservative. The corresponding values for moments, M_{max} and M_d are shown for the cube strength measured in the tests. M is calculated according to Segerlind's design table (4), which includes a material load factor. The design strength of the sheet steel was at a nominal value of f_{by} = 350 N/mm^2. In reality (that is to say during testing) γ = 1.0 and f_y = 390-400 N/mm^2.

Table 2 Test values and design values for shear and bending

Test	V_{max}	f_{ct}	V_d	V_d/V_{max}	M_{max}	Cube strgth	M_d	M_d/M_{max}	Fail mode
	(N) (*10^3)	(MPa)	(N) (*10^3)		(Nmm) (*10^6)	(MPa) (*10^3)	(Nmm) (*10^6)		
H130-390	177.0	2.16	120.9	0.683	69.0	37.8	54.1	0.784	SPLIT
H130-780	93.0	2.16	68.3	0.734	72.5	37.8	54.1	0.746	SPLIT
H130-1170	85.4	2.15	50.6	0.592	99.9	37.6	54.0	0.541	BEND
H200-600	185.7	2.26	140.3	0.755	111.4	40.4	115.0	1.032	SHEAR
H200-1200	120.6	2.02	80.7	0.669	144.7	34.4	112.5	0.778	BEND
H300-900	253.8	2.08	156.9	0.618	228.4	35.7	187.0	0.819	SHEAR
H300-1800	148.4	2.07	97.6	0.658	267.1	35.5	187.0	0.700	BEND

Tests H130-1170, H200-1200 and H300-1800 failed in bending. See fig 9. Tests H130-390 and H130-780 failed by the slab splitting along the line of the upper flange of the panels as shown in fig 9. This form of failure may easily be remedied by placing additional reinforcement at right-angles to the direction of the panels in areas of high shear. As may be seen in table 2 the load values at failure are far higher (+ 30 %) than the calculated values without load factors so that addition reinforcement is not really required. It is however reassuring to note that additional reinforcement at right angles to the panels could increase the shear capacity of the panels even further. This is a possible area for continued research.

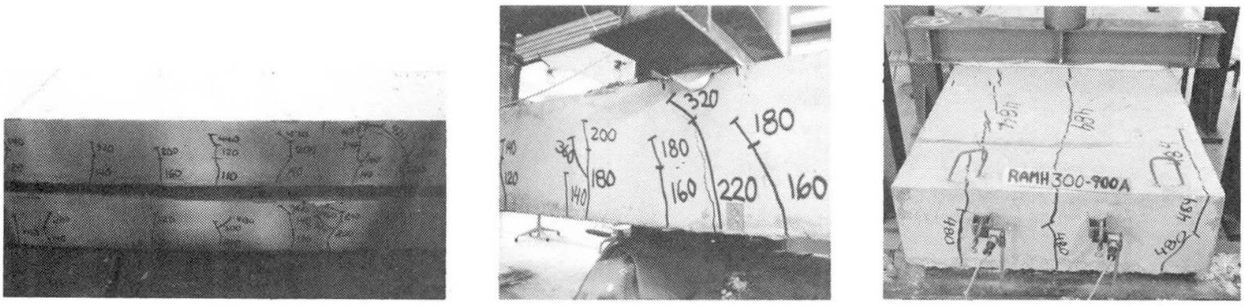


Fig 9 Bending failure Shear failure Failure due to splitting

5 FIRE PERFORMANCE

The BLK 300/80 composite panel was designed to withstand 60 minute standard fire (ISO 834). Fire tests were carried out at the National Testing Station in Borås in Jan 1985 in order to verify the preliminary computer calculations. The time - temperature curves for the computer analysis and the actual results obtained during testing are shown in fig 10. These results may be used for calculation in the fire engineering design of the BLK 300/80 composite slab. The results obtained from such an analysis vary from case to case, but it may generally be concluded that a 200 mm thick slab spanning 6 m without support restraint will sustain a working office load of 2 kN/m^2 for 90 mins and a reduced load of 1,8 kN/m^2 for 120 mins. Complete fire engineering design tables and details are available from Dobel AB.

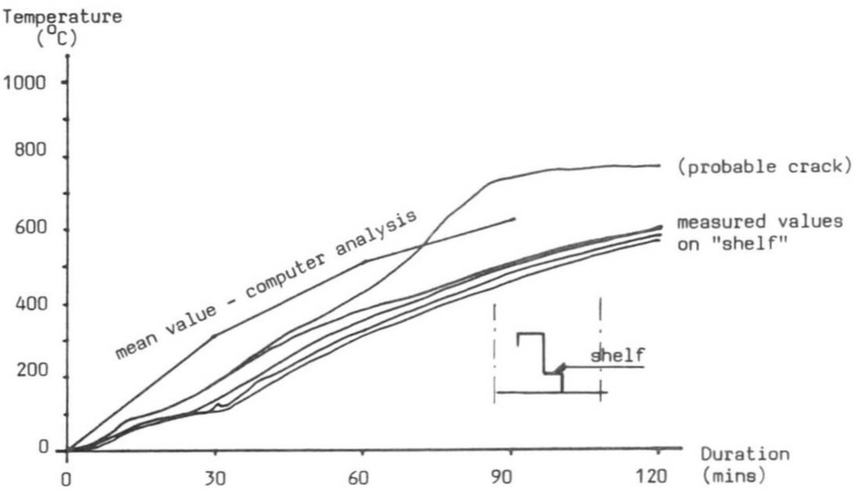


Fig 10 An example of the theoretical and test temperatures in the composite slab

SYMBOLS

A_s	Crossectional area of tensile reinforcement
B	Width of test specimin
b	Width of panel
d	Depth of compressive reinforcement
E_c	Modulus of elasticity of concrete
E_s	Modulus of elasticity of steel
f_{cc}	Cylinder strength of concrete
f_{ty}	Tensile yeild strength of steel
H	Total height of test specimin
h	Effective height from neutral axis of tensile reinf.
I	Second moment of area
L	Span
l	Length
M_d	Design value of strength with respect to bending moment
M_u	Ultimate value of strength with respect to bending moment
P_d	Design load
P_u	Theoretical ultimate load
q_D	Self weight per unit of length
q_L	Live load per unit of length
t, t_{nom}	Thickness of sheet incl zinc, coating, etc.
t_{core}	Thickness of steel within the sheet
x	Depth of compression zone
y	Deflection of test specimin
ε_{cu}	Ultimate strain in concrete
σ_s	Strain in steel reinforcement
σ_{cb}	Calculated compressive strength due to bending

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