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## BS476: Part 8 Fire Tests on Two Slim Floor Assemblies

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British Steel Corporation

Research Organisation



## BS476:PART 8 FIRE TESTS ON TWO SLIM FLOOR ASSEMBLIES

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SYNOPSIS

It is possible to reduce the floor depth in a steel framed multistorey building by utilising a column section as a floor beam to support precast concrete slabs on the inside of the bottom flange. Although not a common design, the assembly provides partial protection to the steelwork. BS476:Part 8 fire tests have been carried out on two loaded slim floor assemblies to establish the benefits of improved fire resistance to respectively, a 254 x 254 mm x 73 kg/m and a 254 x 254 mm x 89 kg/m BS4360 Grade 43A column.

The slim floor incorporating the 73 kg/m column was overloaded by 16% as based on BS449, assuming point loading from the concrete slabs, and exhibited a fire resistance of 44 min. The lower flange reached a temperature of 757°C whilst the centre of the web reached 191°C. The slim floor assembly incorporating the heavier column was loaded to 87% of the maximum permitted value. A fire resistance time of 93 min was measured but the rate of deflection enabled the fire test to be prolonged until L/20 had been reached after 109 min. At the 'failure time' corresponding with the L/30 criterion the lower flange temperature was 948°C but that at the centre of the web was only 242°C.

The current experiment differed from previous fire tests on steel elements in that a biaxial stress system was imposed on the lower flange.

A finite element analysis prediction of deflection gave good agreement with the experimental results on the lighter column but was less accurate for the case where the fire resistance was greater.

KEY WORDS

- |                     |                       |
|---------------------|-----------------------|
| 3. Fire Resistance  | 7. Fire Tests         |
| 4. +BS 476          | 8. Lab Reports        |
| 5. Concrete Precast | 9. +BS 4360 Grade 43A |
| 6. Floors           |                       |

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## BS476:PART 8 FIRE TESTS ON TWO SLIM FLOOR ASSEMBLIES

1. INTRODUCTION

The most common form of floor construction in steel framed buildings consists of precast concrete slabs resting on the top flange of a universal beam. Few fully loaded, bare steel beams in sizes of commercial interest achieve a fire resistance time in excess of  $\frac{1}{2}$  h when tested to BS476:Part 8 in the simply supported condition. However, research has shown that by combining steel frame members within other elements of structure it is possible to attain a fire resistance of 1 h without the added cost of fire protection. Such a design procedure is incorporated in the draft BS5950:Part 8 for the shelf angle beam. An added advantage is that the depth of the floor membrane is reduced by placing the precast concrete slabs on steel angles bolted to the web of the beam.

The floor depth in a steel framed multistorey building influences the costs of cladding and service ducting. Slim floors with minimum beam downstand allow reduced cladding area and greater freedom for service runs in any direction with the result of reduced cladding material cost and service fixing time. Current designs of slim floor utilise a column section as the floor beam to support the precast concrete slabs on the inside of the bottom flange. The assembly provides partial protection to the steelwork with the attendant benefits of improved fire resistance.

This form of floor construction is not common and initial interest in the use of 254 x 254 mm serial sizes of column stemmed from a recent enquiry relating to a new design of hospital. BS476:Part 8 indicative fire tests provided information on the rise in temperature of a number of these sections assembled with 200 mm thick concrete slabs<sup>1</sup>. The subsequent analysis of structural behaviour using a finite element model indicated that the ability of the 254 x 254 mm BS4360:Grade 43A universal columns to achieve a 1 h fire rating depended upon section weight.

The present report describes BS476:Part 8 fire tests on two loaded slim floor assemblies using, respectively, a 254 x 254 mm x 73 kg/m and a 254 x 254 mm x 89 kg/m BS4360:Grade 43A universal column section.

2. DETAILS OF CONSTRUCTION2.1 Steel Supply

The steel sections used in both assemblies were obtained from a local steel stockholder and comprised a 5.0 m length of 254 x 254 mm x 73 kg/m universal column and a 5.0 m length of 254 x 254 mm x 89 kg/m universal column. Samples were taken from each of the sections for chemical analysis. The results are given in Table 1 and show that both chemical compositions were within the limits specified for BS4360:Grade 43A steel. Tensile test results, shown in Table 2 also complied with the standard requirements.

2.2 Concrete Slabs

The concrete slabs used with the lighter column were manufactured by Richard Lees and supplied as 1550 x 600 x 200 mm deep standard 'Spiroll' units which are prestressed, precast hollow cored concrete floor slabs. Each slab had a solid but tapered end extending over a length of 250 mm, as shown in Fig. 1(a). Spiroll slabs are reported to have a fire rating of 2 h. They are manufactured by an extrusion process on a heated bed to assist with curing, producing a concrete cube strength of 38 N/mm<sup>2</sup> after 6 h treatment.

The concrete slabs used with the heavier column were identical to those used in an earlier shelf angle floor test. They were cast into 1550 x 550 x 200 mm deep blocks containing a steel reinforcement layout, as shown in Fig. 1(b), and stored indoors until the day of the test. Cube tests showed the concrete mix to be at Grade 30 strength.

### 2.3 Instrumentation

Fifteen 3 mm diameter chromel/alumel Type K thermocouples with an Inconel sheath and insulated hot junctions were embedded at mid-depth in each steel column at the positions shown in Fig. 2. Four thermocouples were fitted to the centre of the web, five to the lower flange, four to the upper flange and two at the flange/web junctions.

Six thermocouples were also used to measure the furnace atmosphere temperature. These were located 100 mm distant from the test column at regular intervals along its length and level with the lower flange.

A total of twelve single element strain gauges (Showa Type N11-FA-10-120-11) each with a 10 mm gauge length were mounted in the longitudinal direction on the lower flange of the heavier column at the positions shown in Fig. 3, seven strain gauges at the centre of the span and five at  $\frac{1}{4}$  of the span. Gauge numbers, 3, 5, 9 and 11 corresponded to the run out of the web root radius on the flange.

### 2.4 Assembly

The lighter 254 x 254 mm x 73 kg/m column was positioned on the floor furnace at the WRC to give an effective span of 4.5 m between the roller supports. The 200 mm deep Spiroll slabs were positioned along each side of the column with the tapered end resting on the inner surface of the lower flange, Fig. 4, and the opposite end resting on a blockwork wall. A total of seven slabs was positioned on either side of the column, each with a bearing length of 75 mm on the lower flange and separated from the web by a distance of approximately 50 mm. This cavity was filled with dried sand. The upper flange of the column was also covered by a 25 mm layer of sand to simulate the thermal characteristics of the screed that would be used in site practice. A 12 mm gap was left between the edge of the end slabs and the furnace wall to allow free movement as the column deflected during the test. Ceramic fibre blankets were used to cover the gaps at both ends.

Photographs of the construction during assembly are shown in Fig. 5. A similar procedure was followed in the second test using the 254 x 254 mm x 89 kg/m column, but as the solid concrete slabs were narrower, a total of sixteen were placed in position during the assembly.

### 2.5 Loading

The load to the column was applied through the concrete floor slabs to simulate service conditions but its magnitude was the subject of some debate. For the first fire test on the 254 x 254 mm x 73 kg/m column the local effects due to the fact that the load was applied to the lower flange were ignored following advice from some designers. Therefore, the total hydraulic force of 329 kN was based on the maximum uniformly distributed load of 262 kN carried by this section about its 'X-X' axis. The secondary stresses generated in the lower flange were taken into account for the fire test on the 254 x 254 mm x 89 kg/m column using the Von Mises criterion of yielding. A total hydraulic force of 340 kN was based on the assumption that the lower flange supported a uniformly distributed load and that the maximum Von Mises stress occurred at the run out of the web/flange fillet. Details of the loading calculations are given in Appendix 1.

The total load was applied to the concrete slabs by eight rams using 1 m lengths of 152 x 152 mm x 23 kg/m universal column as load spreaders, as shown in Fig. 6.

Deflection measurements were taken at the centre of each beam by Warrington Research Centre staff using a potentiometric system.

## 3. EXPERIMENTAL RESULTS

### 3.1 254 x 254 mm x 73 kg/m Column Assembly 11th July 1985

The slim floor assembly achieved a fire resistance time of 44 min at which time the L/30 failure criterion of 150 mm deflection was reached. A copy of the letter from Warrington Research Centre confirming this result is given in Appendix 2.

### 3.1.1 Deflection Behaviour

The deflection measurements made at the centre of the column are shown in Fig. 7. The vertical deflection increased steadily until the L/30 criterion was reached. Beyond this time the rate of deflection was 9 mm/min. This behaviour was similar to that observed on simply supported, unprotected beams.

### 3.1.2 Temperature Measurements

A summary of the steel temperatures and furnace atmosphere temperatures at various stages during the test is given in Table 3 and in Figs. 8-12. Figure 8 shows the average rise in temperature of the central lower flange on the test column. At 'failure', the scatter between the five temperature readings was 744-774°C with a mean value of 757°C. The average temperature profile for the lower flange/web junction is shown in Fig. 9 which reached a maximum value of 658°C. The average rise in temperature of the fully protected web is shown in Fig. 10. At 'failure', the temperatures were in the range of 182-207°C with a mean of 191°C. As shown in Fig. 11 the upper flange temperatures reached a mean temperature of 37°C.

The average furnace atmosphere heating curve is compared to the International temperature/time curve in Fig. 12 which shows that the heating rate was in accordance with the standard throughout the test.

### 3.1.3 General Observations

Shortly after the start of the test white fumes arose from the hollow concrete slabs and persisted throughout the heating period. After 17 min, longitudinal hairline cracks (Fig. 13(a)) were noticed at the ends of the slabs furthest away from the test column and became more pronounced as the test progressed (Fig. 13(b)). Several slabs developed shear cracks as shown in Fig. 14 which initiated approximately 60 mm from the flange tip and terminated at the centreline of the load spreader.

After cooling the slim floor assembly was reloaded satisfactorily before being dismantled. There was no evidence of flange distortion, Fig. 15, different from that observed in a simply supported beam.

## 3.2 254 x 254 mm x 89 kg/m Column Assembly 29th April 1986

The slim floor assembly achieved a fire resistance of 93 min at a deflection of L/30. The test was continued until the L/20 'failure' limit was reached after 109 min. A copy of the letter from Warrington Research Centre confirming this result is also given in Appendix 2.

### 3.2.1 Deflection Behaviour

The deflection measurements made at the centre of the column are shown in Fig. 16. The rate of deflection decreased after approximately 40 min but increased again after 75 min. At L/30 the rate of deflection was 3 mm/min. The test was continued until the L/20 failure limit was reached. At this time the rate of deflection was 5 mm/min which was lower than that required to satisfy BS476:Part 20. The deflection behaviour had similarities to that observed on beams following prolonged exposure to fire.

### 3.2.2 Temperature Measurements

A summary of the steel temperatures and furnace atmosphere temperatures at various stages during the test is given in Table 4 and in Figs. 17-21. Figure 17 shows the average rise in temperature of the central lower flange on the test column. At a deflection of L/30 the scatter between the five temperature readings was 937-967°C with a mean value of 947°C; at L/20 the scatter in temperature was 986-1014°C with a mean value of 999°C. The average temperature profile for the lower flange/web junction is shown in Fig. 18 which reached 849°C at L/30 and 912°C at the L/20 failure limit. The average rise in temperature of the fully protected web is shown in Fig. 19 which reached 242°C at L/30 and 293°C at L/20. As shown in Fig. 20, the average upper flange temperatures were respectively, 94 and 98°C.

The average furnace atmosphere heating curve is compared to the International temperature/time curve in Fig. 21 which shows that the heating rate was in accordance with the standard throughout the test.

### 3.2.3 Strain Gauge Measurements

The longitudinal stresses measured at the two locations on the bottom face of the lower flange are presented in Table 5. The changes in strain were obtained as the hydraulic load of 340 kN was applied to the assembly. A bending stress of 20.4 N/mm<sup>2</sup> has been added to the figures in Table 5 to allow for the dead load from the weights of the column, concrete slabs and spreader beams. The dimensions of the column were not measured accurately along its length. The magnitude of the longitudinal stresses suggested that the maximum design stress had not been applied in full. The distribution of the stresses across the lower flange implied uneven loading across the section with a tendency towards twisting.

### 3.2.4 General Observations

The concrete slabs showed no evidence of cracking during the fire test. Closer inspection on the following day showed the presence of vertical cracks following the steel reinforcement in the solid concrete slabs at the ends of the column. Other slabs in this vicinity developed shear cracks.

After cooling the slim floor assembly was reloaded satisfactorily before being dismantled. As shown in Fig. 22 there was no evidence of flange distortion different from that experienced by a simply supported beam except for local indentations from isolated concrete slabs.

## 4. DISCUSSION

The columns studied in these experiments represented the lightest sections in the 254 x 254 mm serial size range. Neither column had previously been subjected to the BS476:Part 8 fire test as an unprotected beam in the simply supported condition. As their respective  $H_p/A$  values for 3-sided exposure were only 110 and 130 m<sup>-1</sup>, a fire resistance less than 30 min would be expected.

The fire resistance of the slim floor assembly using the 254 x 254 mm x 73 kg/m was 44 min for an imposed load of 328 kN. Once the L/30 failure limit had been reached the rate of deflection exceeded the value set by the new BS476:Part 20 standard for the L/20 criterion. The fire resistance of the slim floor assembly using the 254 x 254 mm x 89 kg/m was 93 min at the L/30 failure limit for an imposed load of 340 kN. At this time the rate of deflection was 3 mm/min. The test was continued until the L/20 criterion was satisfied after 109 min at a rate of deflection of 5 mm/min which was 3.8 mm/min below the limit set by the expression  $L^2/9000d$ , where  $L$  = effective span (mm) and  $d$  = distance from the top of the structural section to the bottom of the design tensile zone. The improved fire resistance compared to a bare section was due principally to the partial protection provided by the concrete floor slabs.

The concrete cover slabs resisted the heat flow to the upper parts of the steel section. For instance, at the end of the test on the lighter section, when the lower flange had reached a temperature of 757°C, the upper flange was still only 37°C. The corresponding temperatures for the heavier section were 948°C and 94°C. The marked temperature gradient enhanced the load bearing capacity of each section during the test above the strength behaviour expected from the unprotected member. Similarly, the temperature distribution would be expected to reduce the rate of deflection at the higher combustion gas temperatures generated by a fire.

A standard fire test in which the load was imposed onto the lower flange of the section had never previously been carried out. The loads in a shelf angle beam had been applied to a secondary angle section attached to the web. Therefore, the influence of the additional stresses in the lower flange on the fire resistance of the slim floor were not known. Preliminary discussions with design engineers failed to produce any clear advice. Yielding would first occur in the lower flange in the locality of the root radius with the web. One recommendation was to ignore this effect in comparison with the overall plastic behaviour of this part of the section. An alternative approach was to evaluate the lateral bending stresses at

the run out of the web/flange fillet that were generated either, by uniform loading over the area of overlap with the concrete floor slabs, or by point loading at the leading edge of the slab.

The biaxial stress system existing on the lower flange face in the slim floor assembly is different from the simple bending situation and can be quantified by using the Von Mises yield criterion. The combined stress has a greater effect at the fillet radius if the uniform loading case is adopted. From an experimental point of view the point loading case is considered to be the more realistic because the flange would deflect to a greater extent than the concrete cover slab. On this basis, therefore, the calculated Von Mises stress in the locality of the fillet radius on the lower flange of the 73 kg/m section was 191 N/mm<sup>2</sup> and on the lower flange of the 89 kg/m section was 154 N/mm<sup>2</sup>. The 254 x 254 mm x 73 kg column was therefore overloaded in the standard fire test by 16% using the point loading approach or by 26% using a uniform distributed load.

The longitudinal stresses measured across the lower flange of the 89 kg/m section on application of the load suggested that the column had been subjected to torsional forces. The average longitudinal stress at the centre of the beam was 130.7 N/mm<sup>2</sup> and 128.8 N/mm<sup>2</sup> in the vicinity of the fillet radii. To a first approximation the Von Mises stress in the lower flange is proportional to the magnitude of the longitudinal stress. On this basis, 87% of the maximum design stress was generated in the heavier column section using the point load calculation.

If the benefits offered by this form of partial protection are to be recognised in design it is necessary to evaluate the behaviour of the complete range of slim floor assemblies likely to be encountered in practice. The most cost effective approach is to complement a limited number of fire tests by mathematical modelling. Progress in the development of a two stage finite element programme for the prediction of the deflection characteristics of bare and partially protected steel sections in the BS476:Part 8 test has been described elsewhere<sup>2</sup>.

A comparison between the predicted and measured deflections of the slim floor assemblies studied in this exercise is made in Fig. 23. The predicted fire resistance of the 254 x 254 mm x 73 kg/m, loaded to 116% of the maximum design stress allowed in BS449 was 41.6 min, whereas a measured deflection of L/30 occurred after 44 min. By reducing the imposed load to the maximum permitted value the predicted fire resistance increased to 51 min.

The predicted fire resistance of the 254 x 254 mm x 89 kg/m, loaded to 87% of the maximum design stress (based on the strain gauge measurements) was 66 min whereas the measured deflection of L/30 occurred after 93 min. Such a wide difference in fire resistance time was surprising, particularly in view of the better correlation with experimental observations in other partially protected floor assemblies. The thermal model predicted temperatures in the lower flange that agreed closely with measurements but indicated a greater rise in temperature at the centre of the web except at the end of the test. In the latter position, the absolute values of temperature were insufficient to influence significantly the load carrying capacity of the section. The FASBUS II analysis of deflection uses as a temperature model absolute values at the quartile positions with linear interpolation between these points. Where the change in temperature across the lower flange and the lower part of the web is not extreme, as in shelf angle floor beams, this assumption is acceptable. However, in the case of slim floors where the temperature gradient is high in the lower part of the steelwork the model can overestimate the temperature. In the latter part of the fire test this portion of the beam carries a high load and is therefore the critical zone. A sensitivity analysis of element size in this region is to be carried out.

During the fire test it was noticed that individual concrete slabs formed a stepwise pattern as the column deflected. Based on earlier fire tests, it is not considered that this interaction could provide a significant contribution to the load bearing capacity of the assembly.

## 5. CONCLUSIONS

A BS476:Part 8 fire test on a slim floor assembly using a 254 x 254 mm x 73 kg/m BS4360 Grade 43A universal column gave a fire resistance time of 44 min. Based



on BS449 the assembly was overloaded to a design stress of 16% above the maximum, assuming point loading from the leading edge of the concrete slabs.

A BS476:Part 8 fire test on a slim floor assembly using a 254 x 254 mm x 89 kg/m BS4360:Grade 43A universal column gave a fire resistance time of 93 min at the L/30 deflection limit. The test was prolonged until a deflection of L/20 had been reached after 109 min. At this time the rate of deflection was still less than the maximum permitted by BS476:Part 20. It was calculated that 93% of the maximum design stress had been applied to the section but longitudinal stresses measured on the lower flange suggested that only 87% of the maximum design stress had been generated.

As the loads had been applied through the concrete slabs the influence on fire resistance of the additional stresses in the lower flange of the column were unknown. Load calculations were based on the yielding of the lower flange in the vicinity of the fillet radius with the web using the Von Mises yield criterion to quantify the effect of longitudinal and lateral bending. Point loading from the leading edge of the slab was considered to be applicable.

At the L/30 failure criterion the average lower flange temperature of the 73 kg/m column was 757°C and the temperature at the centre of the web was 191°C; the respective temperatures for the heavier column were 948°C and 242°C. Neither column showed any evidence of lateral bending on the lower flange.

A finite element analysis predicted a fire resistance of 41 min for the 254 x 254 x 73 kg/m column at 116% of the permitted design stress and 51 min for the same section loaded to the maximum design stress. A fire resistance of 66 min was predicted for the 254 x 254 mm x 89 kg/m column at 87% of the maximum design stress (BS449). The agreement between the calculated and actual fire resistance periods was very close for the lighter column but the analysis was more conservative for the heavier column. The explanation for the discrepancy is not known but might be linked to the sensitivity of the FASBUS II model.

## 6. REFERENCES

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2. Preston, R.R. and Kay, T.R., 'Deformation of Steel Beams During BS476:Part 8 Fire Tests', To be Published, Metals and Materials.

## 7. ACKNOWLEDGEMENT

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TABLE 1 CHEMICAL COMPOSITION OF TEST BEAM

Code No.	C	Si	Mn	P	S	Cr	Mo	Ni	Al	N	Nb	Sn	Ti	V
5J865P 254 x 254 mm x 73 kg/m	0.20	0.03	0.97	0.022	0.028	0.02	<0.005	0.03	<0.005	0.0055	<0.005	<0.005	<0.005	<0.005
6J162P 254 x 254 mm x 89 kg/m	0.15	0.19	0.95	0.009	0.014	0.02	<0.01	0.02	0.032	0.0048	<0.005	<0.005	<0.005	<0.005
BS4360 Grade 43A Product	0.30 max.	0.55 max.	1.70 max.	0.06 max.	0.06 max.									

TABLE 2 TENSILE TEST RESULTS FROM TEST BEAM

Code No.	Yield Stress N/mm <sup>2</sup>	Tensile Strength N/mm <sup>2</sup>	El., %
5J865P 254 x 254 mm x 73 kg/m	287	478	34
6J162P 254 x 254 mm x 89 kg/m	299	465	36
BS4360 Grade 43A Specification	255 min.	430/540	20

TABLE 3 SLIM FLOOR TEST - TEMPERATURE DATA SHEET

Section: 254 x 254 mm x 73 kg/m UC

Date: 11.7.85

Slabs: 200 mm thick Spiroll (Hollow)

Failure Time: 44 min

Thermocouple Position		Temperatures (°C) After Various Times (min)														
		3	6	9	12	15	18	21	24	27	30	33	36	39	42	44
Lower Flange (Exposed)	1	84	142	230	326	402	463	521	567	605	641	671	699	723	746	758
	2	79	138	224	317	395	456	513	561	602	638	671	700	726	748	759
	4	111	169	253	336	405	456	507	551	586	620	651	680	705	729	744
	6	111	176	269	363	433	493	548	592	632	664	693	719	744	765	774
	7	60	115	186	286	370	439	501	556	599	636	668	697	722	741	751
	$\bar{X}$	89	148	232	326	401	461	518	565	605	640	671	699	724	746	757
Web (Unexposed)	1	20	22	28	36	48	62	78	94	113	128	143	156	168	179	186
	2	20	23	29	39	54	71	89	109	130	146	161	174	187	199	207
	3	21	23	29	38	51	67	83	100	121	134	146	159	171	183	190
	4	20	23	28	36	49	63	79	95	117	129	141	153	164	175	182
	$\bar{X}$	20	23	28	37	50	66	82	99	120	134	148	160	172	184	191
Upper Flange	3	19	20	20	20	20	20	21	22	24	25	27	30	32	35	37
	5	20	20	20	20	20	20	21	21	22	24	25	27	29	31	33
	8	20	20	20	20	21	21	22	23	25	27	29	31	34	37	39
	9	20	20	20	20	20	21	21	22	24	26	28	30	32	35	37
	$\bar{X}$	20	20	20	20	20	21	21	22	24	26	28	30	32	35	37
Lower Flange/Web Junction	R1	45	84	137	216	289	352	408	459	504	543	577	607	633	656	670
	R2	44	83	133	205	272	329	381	430	473	510	545	576	605	631	647
	$\bar{X}$	44	83	135	210	280	340	394	444	488	526	561	591	619	643	658
Mean Atmosphere ISO Curve 22°C Deflection, mm		496	562	711	734	751	769	802	819	833	849	863	876	891	902	909
		504	605	665	707	740	768	791	810	828	844	858	871	883	894	901
		2	9	15	29	41	51	60	68	74	80	92	100	114	133	150

TABLE 4 SLIM FLOOR TEST - TEMPERATURE DATA SHEET

Section: 254 x 254 mm x 89 kg/m Date: 29.4.86  
 Slabs: 200 mm thick solid Failure Time: 93 min (L/30)  
 109 min (L/20)

Thermocouple Position	Temperatures (°C) After Various Times (min)																																	
	0	3	6	9	12	15	18	20	22	24	27	30	33	36	39	42	45	48	51	54	57	60	65	70	75	80	85	90	93	95	100	105	109	
Lower Flange	10	60	126	192	271	337	399	451	467	496	535	571	606	637	665	691	715	735	753	766	782	799	826	851	877	900	920	938	947	954	971	986	997	
	2	8	76	163	226	302	358	414	461	476	504	544	579	611	639	668	693	716	736	757	771	787	804	835	863	893	919	939	957	967	974	989	1003	1014
	4	8	52	109	165	231	294	353	396	411	441	484	522	556	588	620	649	676	700	722	740	754	769	800	828	856	883	906	926	937	944	961	977	986
	6	7	65	130	192	265	326	385	438	455	485	526	560	590	614	639	662	686	708	728	746	759	773	803	832	860	887	912	932	943	951	968	984	994
	7	10	52	127	185	259	317	376	425	441	471	511	546	577	605	631	657	681	703	725	743	758	772	802	832	860	889	914	935	947	955	972	988	999
$\bar{X}$	9	61	131	192	266	326	385	434	450	479	520	556	588	617	645	670	695	716	737	753	768	783	813	841	869	896	918	938	948	956	972	988	998	
Flange/Web Junction	R1	10	30	66	115	169	222	272	320	336	364	404	442	476	507	537	565	591	615	637	657	675	690	710	731	755	781	807	832	846	856	877	897	911
	R2	10	29	65	117	175	231	283	331	346	375	413	448	481	512	540	565	589	611	631	649	666	679	701	726	754	783	811	838	852	861	882	900	913
Central Web	1	10	11	13	17	24	33	45	57	61	70	83	95	102	106	111	115	121	127	135	143	151	158	171	183	195	206	218	229	237	242	257	272	284
	2	10	11	13	19	28	40	54	69	75	87	98	100	100	101	101	101	102	102	103	106	109	120	132	151	177	203	225	237	245	265	285	301	321
	3	10	10	12	18	27	37	51	65	70	82	95	101	106	109	113	119	127	136	144	152	161	168	179	194	210	226	243	259	269	275	292	308	321
	4	11	12	13	16	22	29	39	49	53	60	71	81	89	95	99	101	106	114	122	130	139	147	160	172	183	194	205	217	224	228	241	254	265
$\bar{X}$	10	11	13	17	25	35	47	60	65	75	87	94	99	102	106	109	114	120	126	132	139	145	157	170	186	201	217	232	242	247	264	280	293	
Upper Flange	3	8	10	10	10	10	11	12	13	13	14	17	20	23	27	31	36	41	46	52	58	64	73	80	87	90	92	93	94	99	95	96	97	98
	5	8	10	10	10	10	11	11	12	12	14	15	17	20	23	28	33	40	45	50	55	60	67	75	83	88	92	94	96	96	96	97	98	99
	8	10	11	12	12	12	13	14	15	17	20	23	28	33	39	49	61	72	80	85	88	92	94	95	95	95	95	96	96	96	97	98	99	99
	9	11	12	12	12	12	13	13	13	14	15	16	18	19	21	23	25	28	32	35	39	43	50	57	65	73	80	87	90	92	96	98	99	99
$\bar{X}$	9	11	11	11	11	11	12	12	13	13	15	17	19	22	26	30	36	42	49	54	59	64	70	76	82	86	90	92	93	94	96	97	98	
Ave. Atmosphere		505	638	664	720	733	760	774	776	774	802	820	832	845	861	873	881	893	905	916	922	933	948	964	978	991	1001	1015	1024	1028	1037	1047	1055	
Standard Curve 17°C		499	600	660	702	735	763	778	792	805	823	834	853	866	878	889	899	909	918	926	935	942	954	965	976	985	994	1003	1008	1011	1019	1026	1032	
Deflection, mm	0	2	7	14	22	30	40	43	47	51	57	63	67	71	75	79	82	85	88	91	93	96	100	105	112	120	130	143	150	155	179	204	224	

**TABLE 5**      **LONGITUDINAL STRESSES ON BOTTOM FACE**  
**OF LOWER FLANGE OF A 254 x 254 mm x**  
**89 kg/m COLUMN IN A SLIM FLOOR ASSEMBLY**

Position	Longitudinal Stress, N/mm <sup>2</sup>		
	1st Loading	2nd Loading	Average
1	148.4	143.9	146.2
2	142.4	137.9	140.2
3	131.4	130.4	130.9
4	131.4	128.9	130.2
5	127.4	127.4	127.4
6	124.4	122.4	123.4
7	117.4	116.4	117.2
8	97.9	95.9	96.9
9	101.4	99.4	100.4
10	107.4	105.9	106.7
11	115.4	110.9	113.2
12	118.4	111.0	114.7

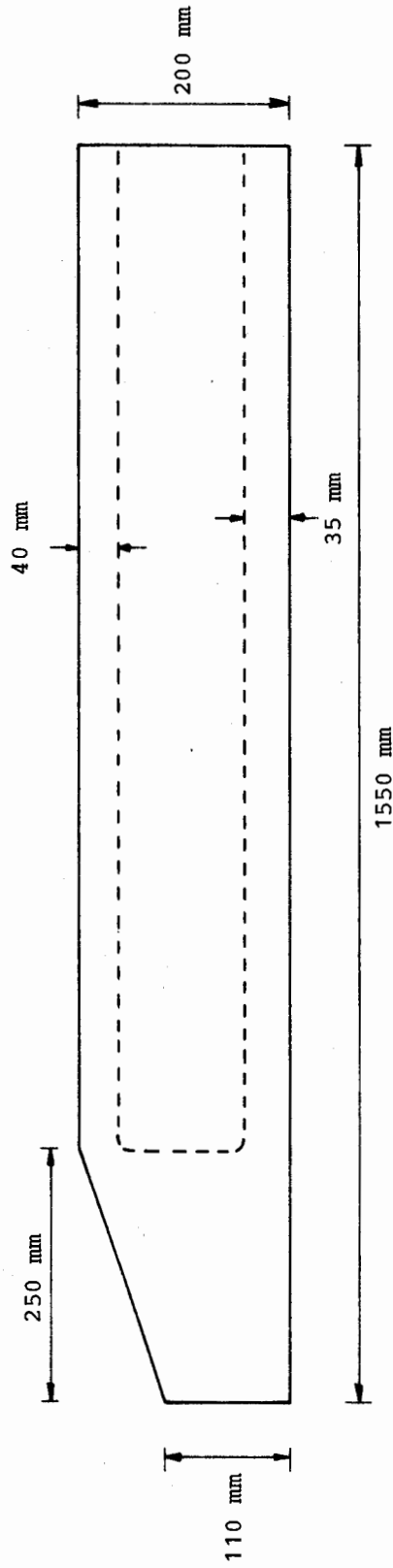


FIG. 1 (a)  
(R2/5469)

SCHEMATIC ILLUSTRATION OF THE 600 mm WIDE  
SPIROLL CONCRETE FLOOR SLAB

All dimensions in mm

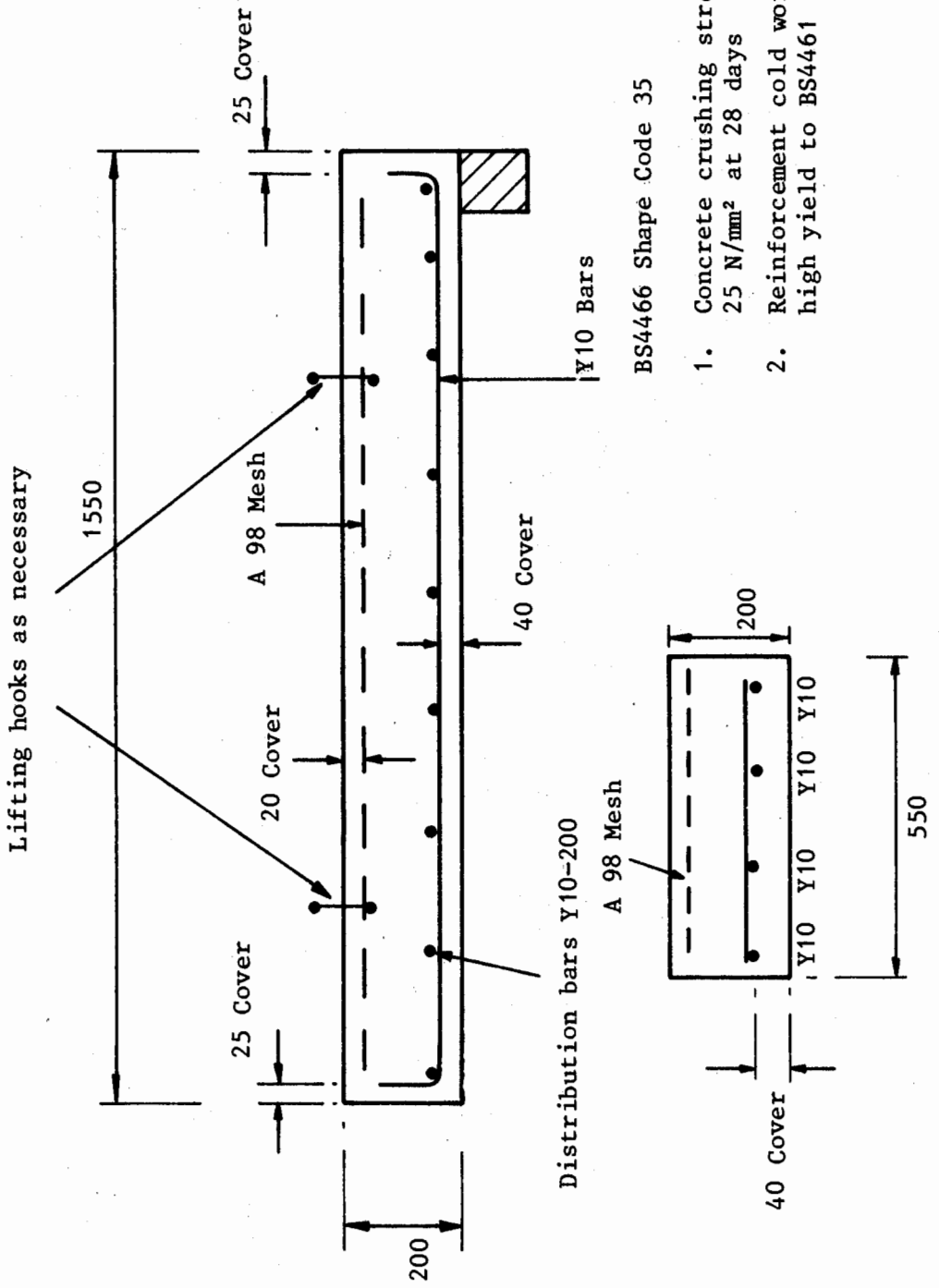
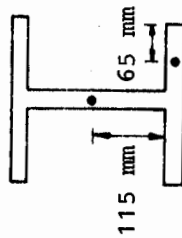
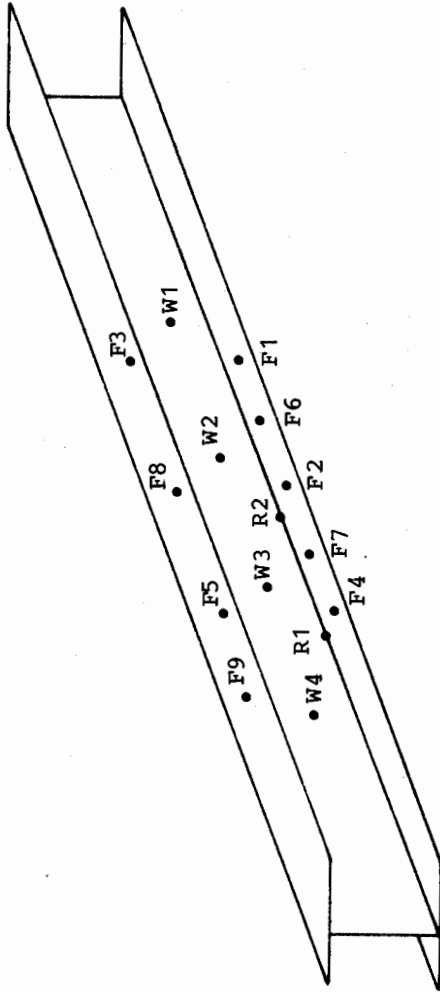


FIG. 1(b)  
(R2/919)

PRECAST CONCRETE SLAB DESIGN USED IN TEST



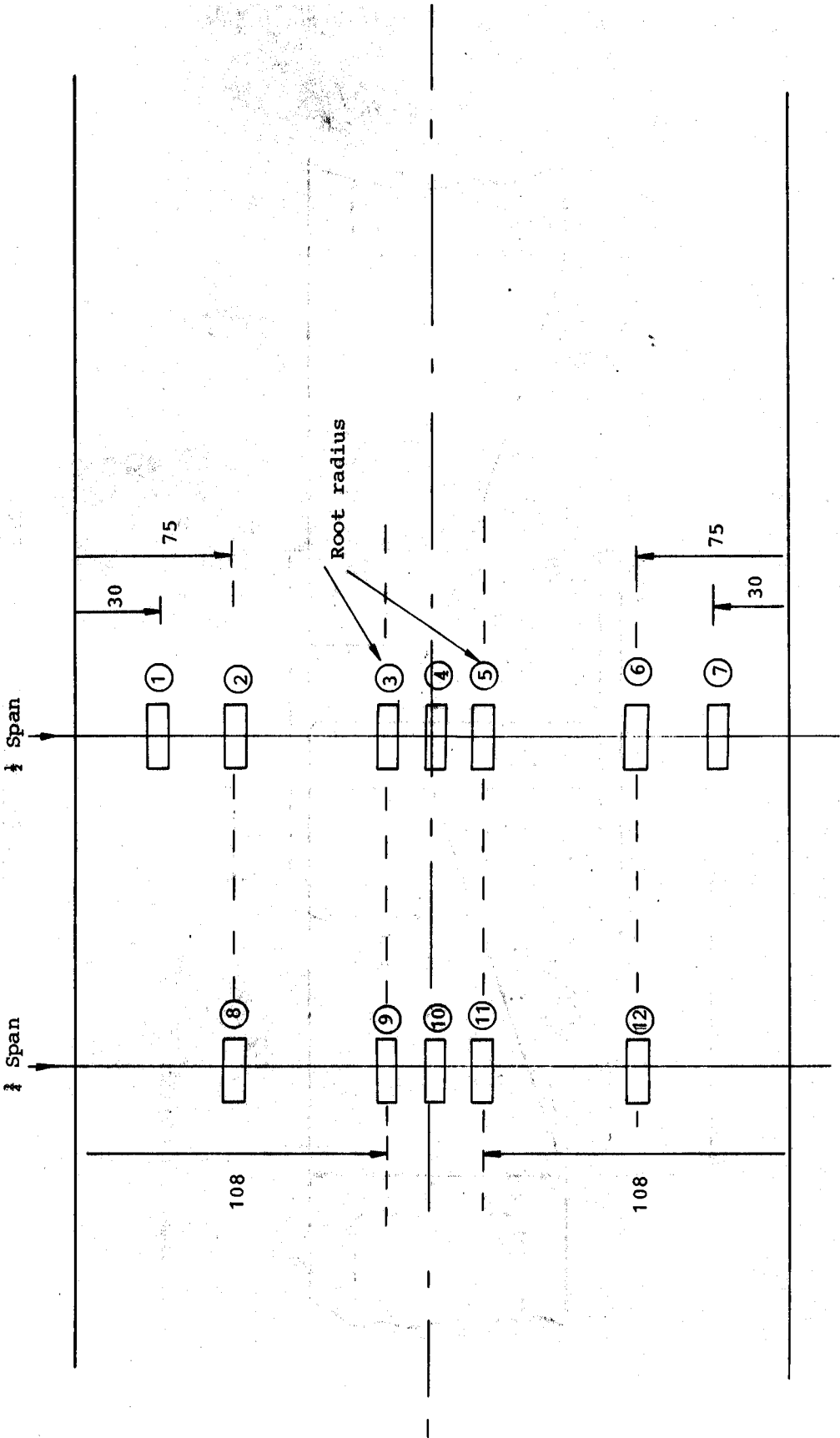
Distance from end of beam to thermocouple

W1	3.44 m
F3, F1	3.14 m
W2, F6	2.82 m
F2, R2	2.52 m
W3, F7	2.20 m
F4, F5, R1	1.90 m
W4	1.59 m
End of beam	5.02 m

POSITION OF THERMOCOUPLES

FIG. 2  
(R2/5470)

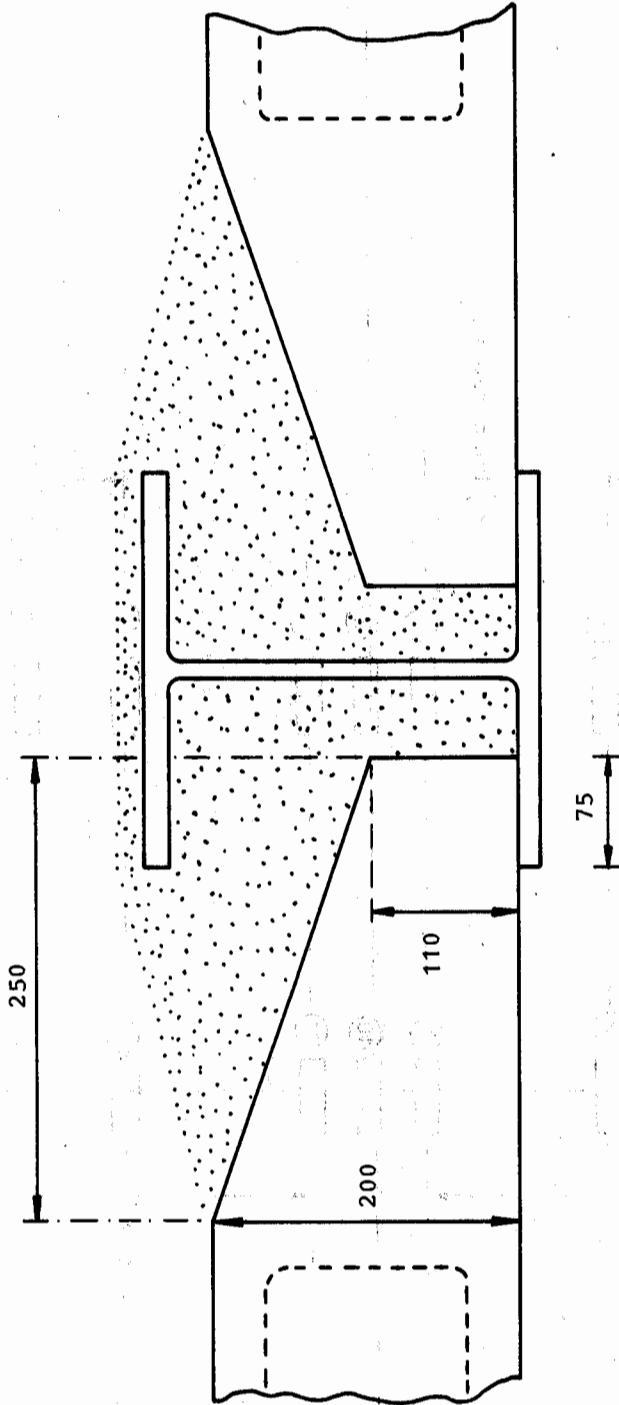
254 x 254 mm x 89 kg/m Column



POSITION OF THE LONGITUDINAL STRAIN GAUGES MOUNTED ON THE LOWER FLANGE

FIG. 3  
(R2/5471)

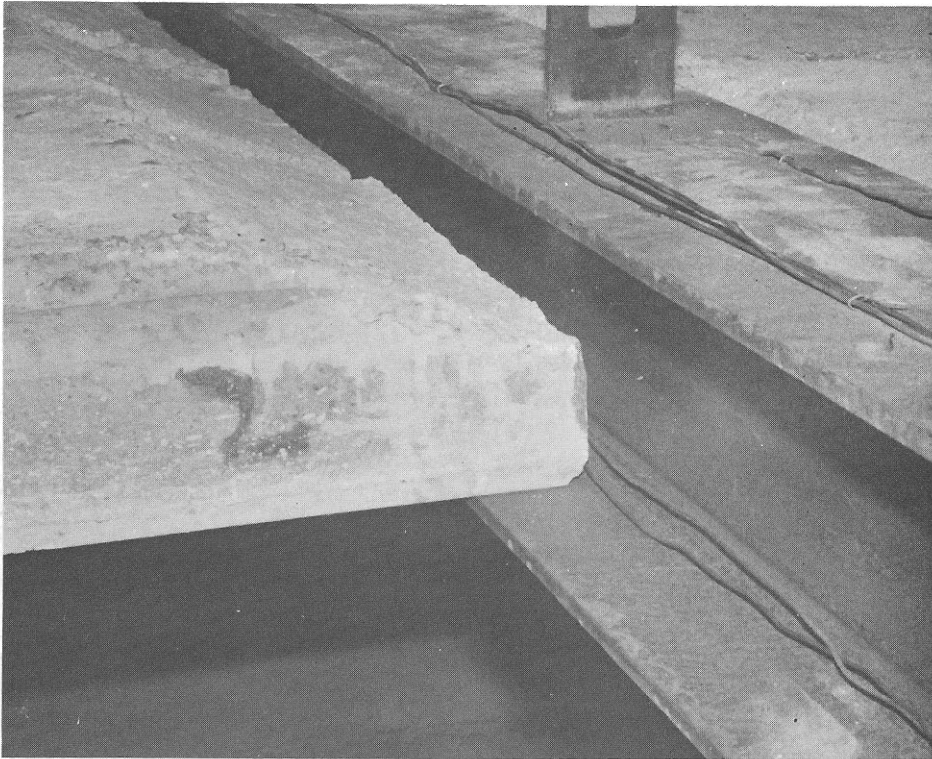




All dimensions in mm

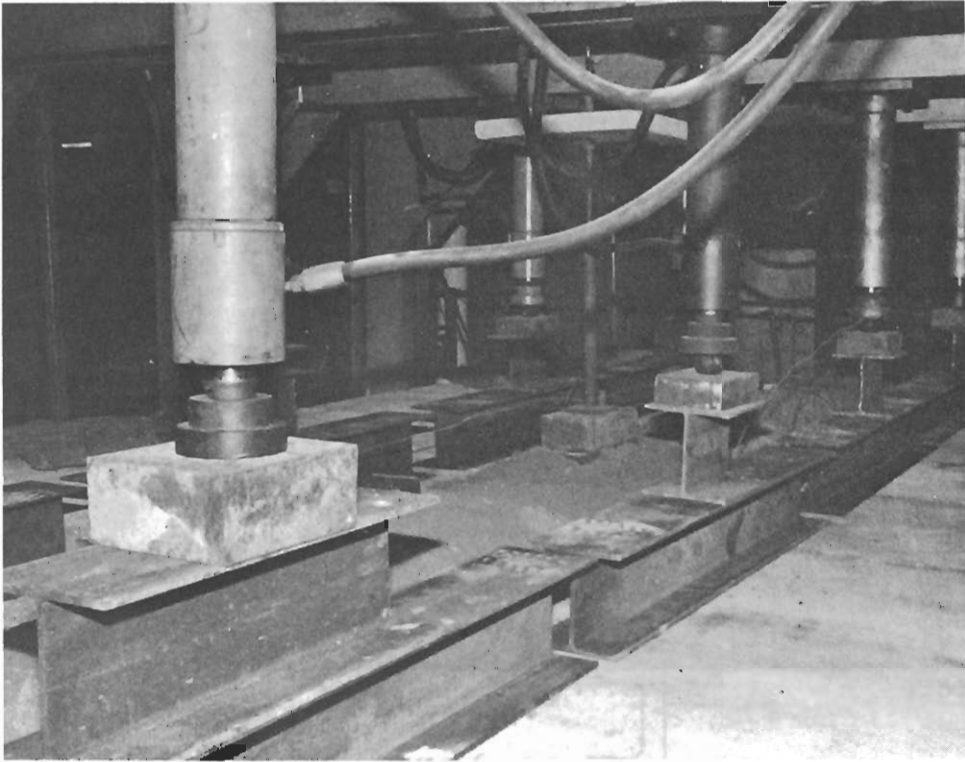
FIG. 4  
(R2/5472)

TEST ARRANGEMENT



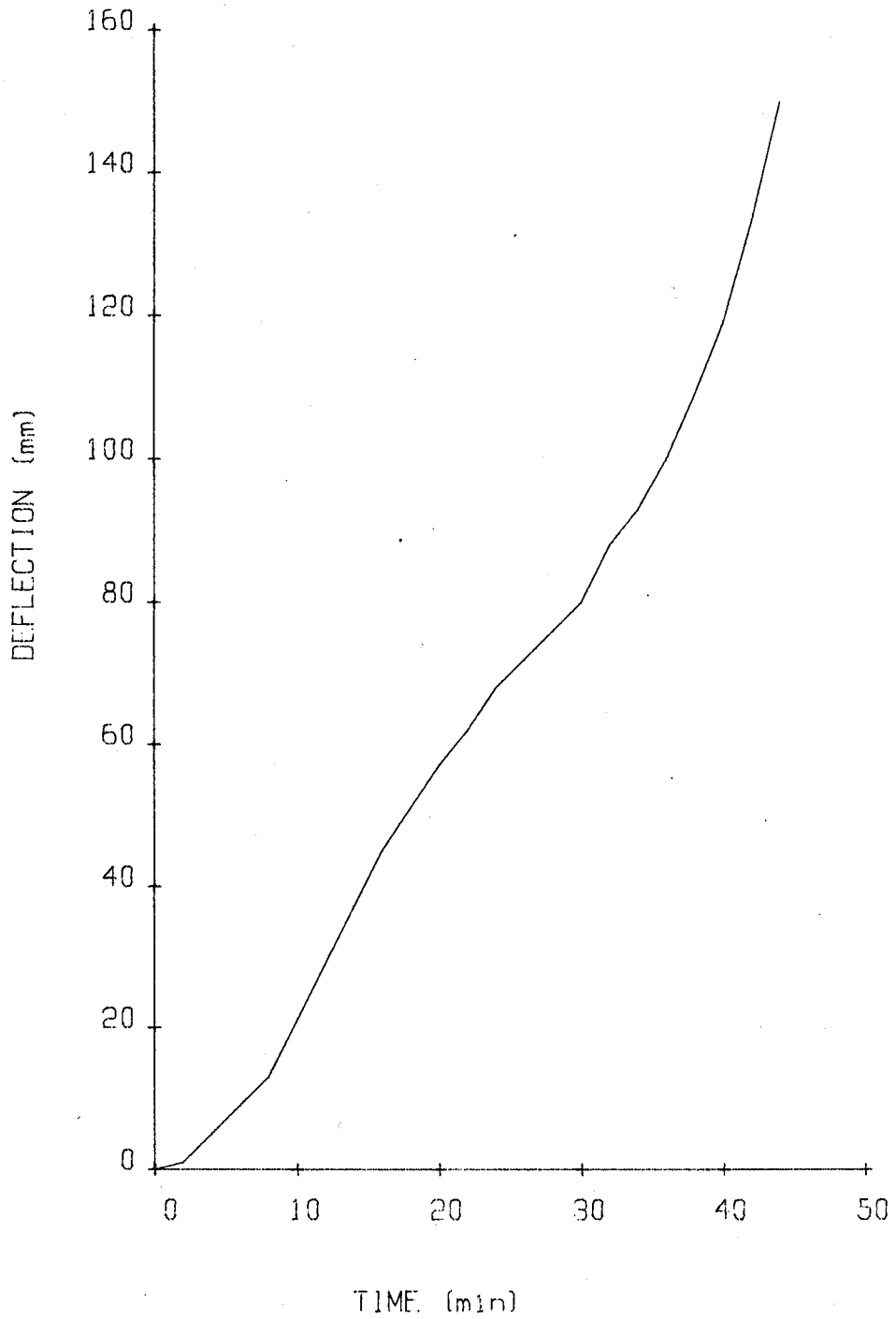
POSITION OF SPIROLL FLOOR SLABS  
ON THE 73 kg/m TEST COLUMN

FIG. 5



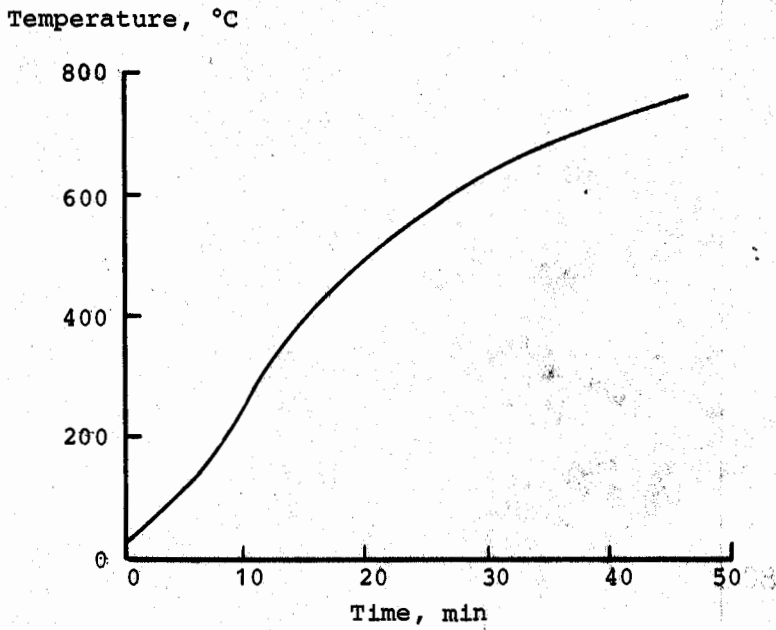
COMPLETED TEST ASSEMBLY

FIG. 6



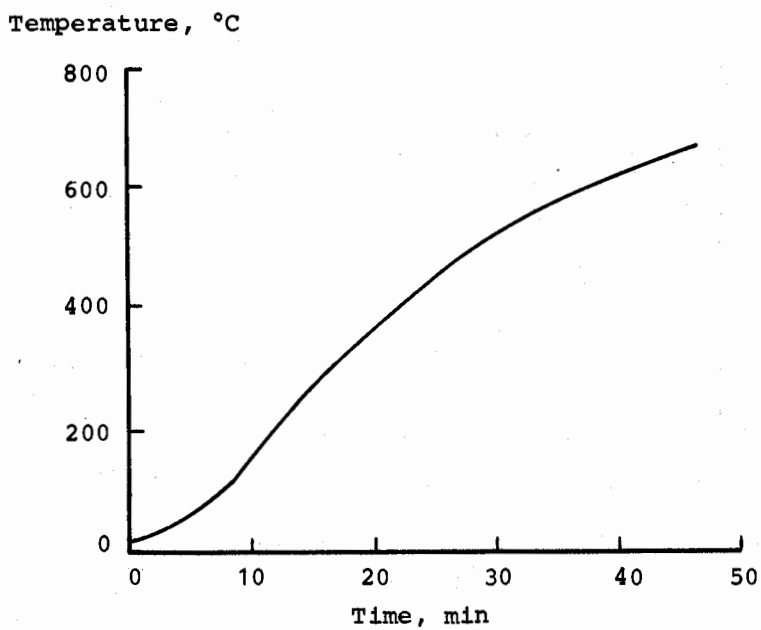
CENTRAL VERTICAL DEFLECTION RECORDED DURING THE TEST  
ON THE 254 x 254 mm x 73 kg/m COLUMN

FIG. 7



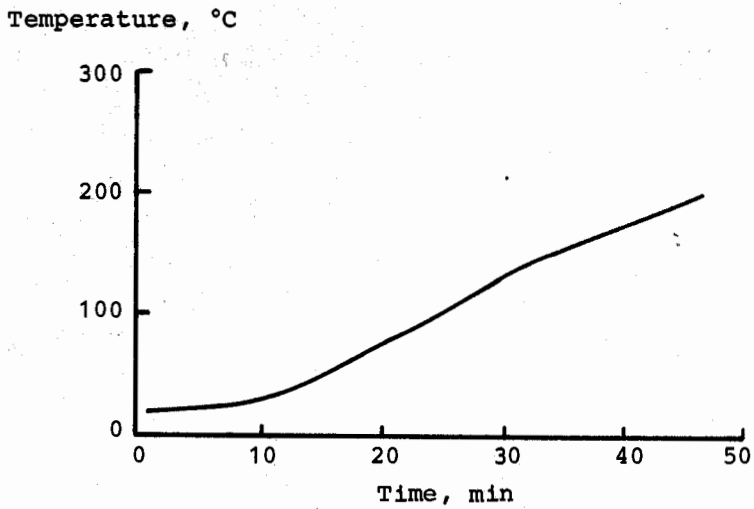
AVERAGE LOWER FLANGE TEMPERATURE RECORDED  
ON THE 73 kg/m COLUMN

FIG. 8



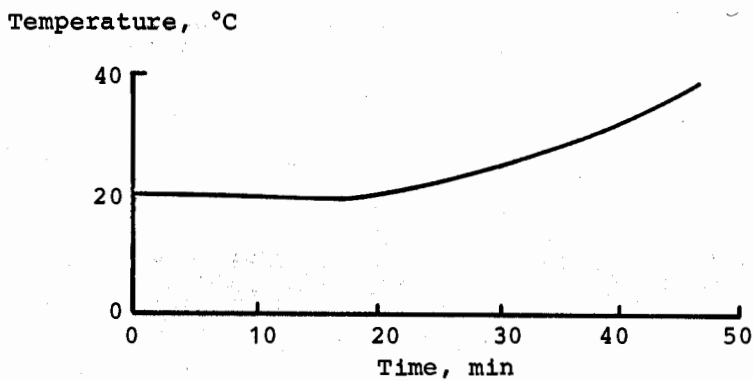
AVERAGE TEMPERATURE AT THE FLANGE/WEB JUNCTION  
RECORDED ON THE 73 kg/m COLUMN

FIG. 9  
(R2/5473)



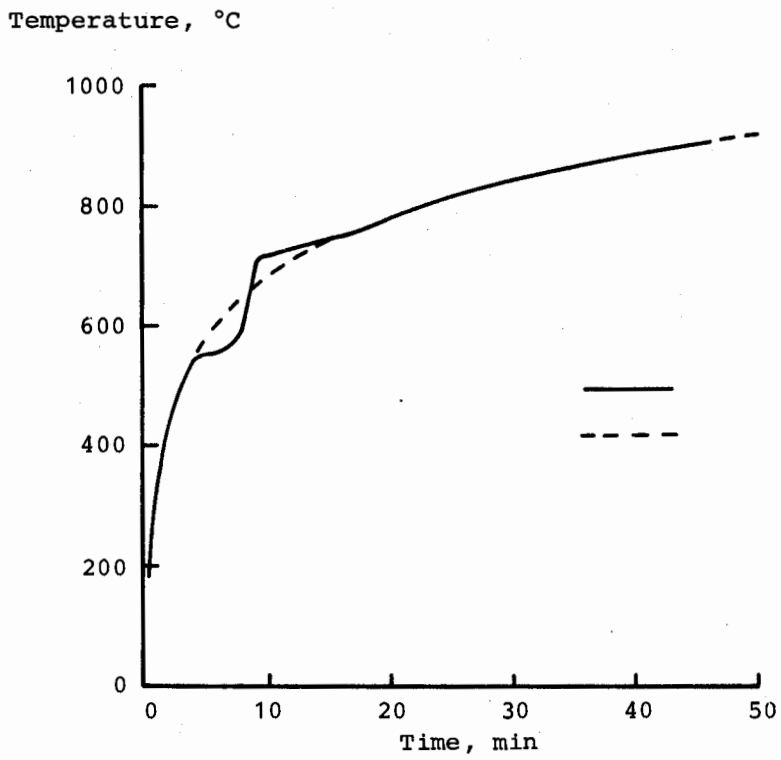
AVERAGE TEMPERATURE OF THE FULLY PROTECTED WEB  
RECORDED ON THE 73 kg/m COLUMN

FIG. 10



AVERAGE TEMPERATURE OF THE UPPER FLANGE  
RECORDED ON THE 73 kg/m COLUMN

FIG. 11  
(R2/5474)



COMPARISON OF FURNACE ATMOSPHERE TEMPERATURE WITH  
THE INTERNATIONAL TEMPERATURE/TIME CURVE

FIG. 12  
(R2/5475)



Longitudinal hairline cracks

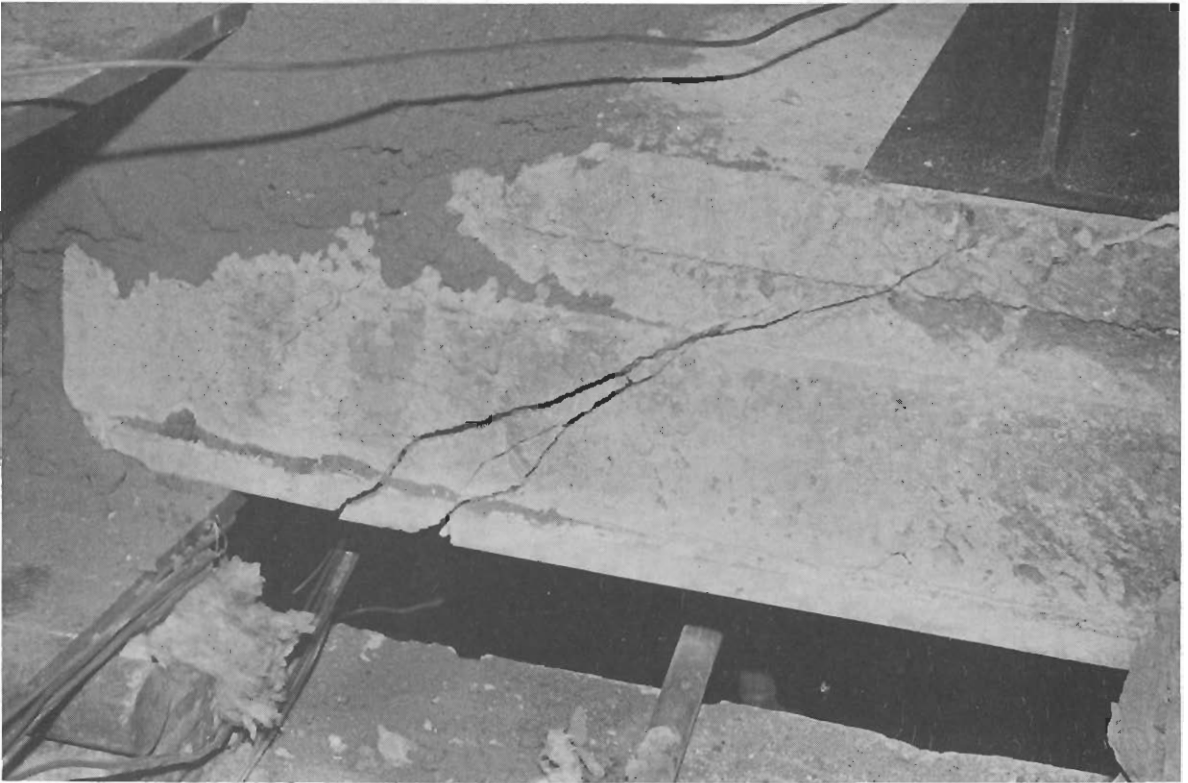


Longitudinal shear cracks

FORMATION OF LONGITUDINAL CRACKS IN THE CONCRETE SLABS  
DURING THE FIRE TEST ON THE 73 kg/m COLUMN

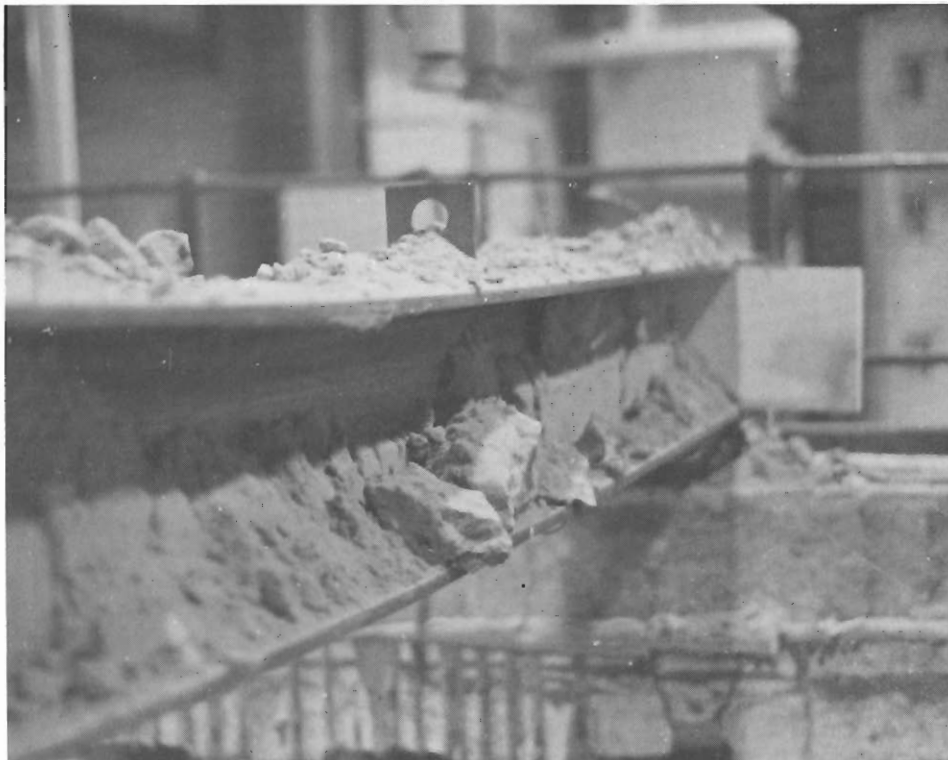
FIG. 13





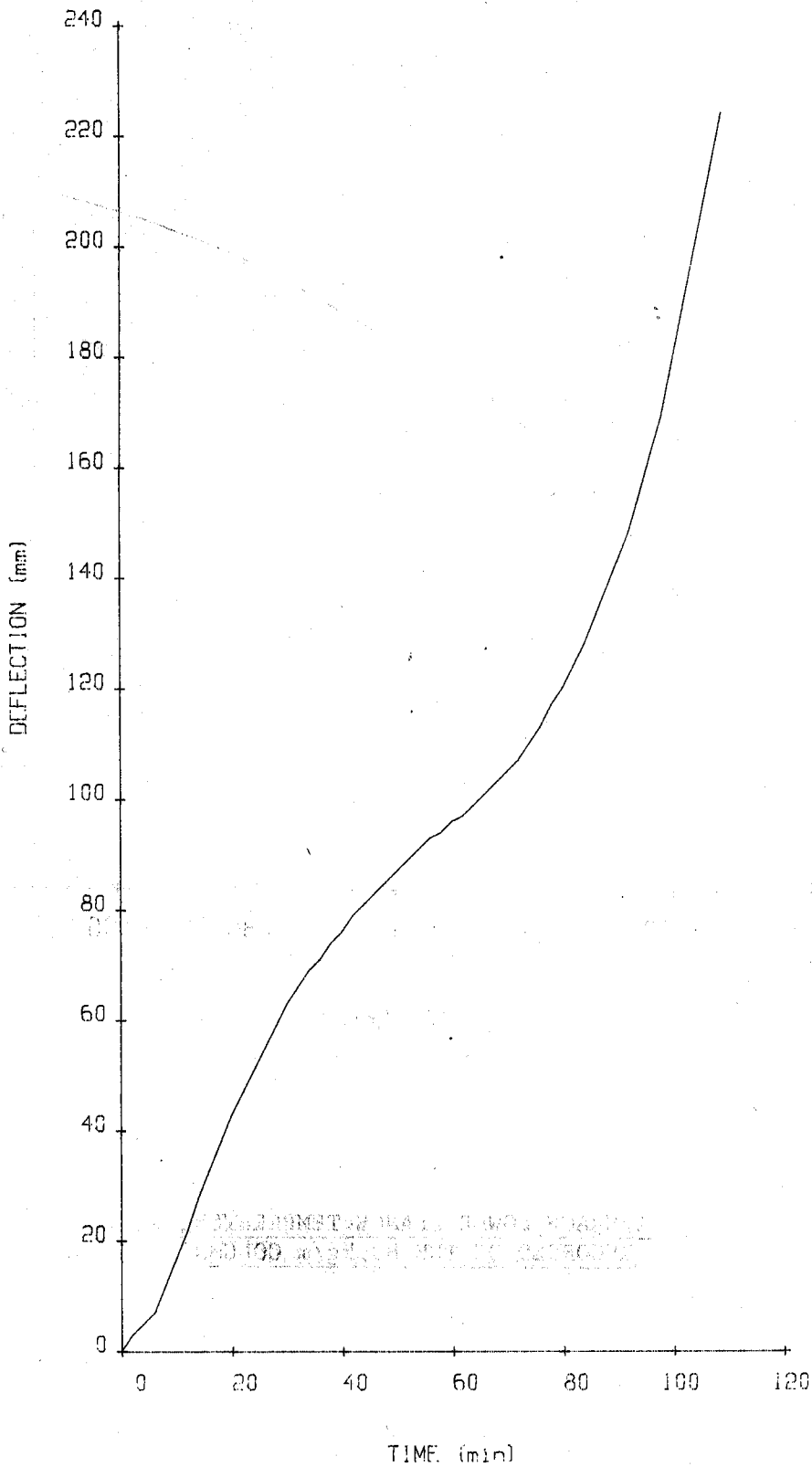
ANGULAR CRACK FORMATION EXPERIENCED BY SOME OF THE CONCRETE SLABS DURING THE TEST ON THE 73 kg/m COLUMN

FIG. 14



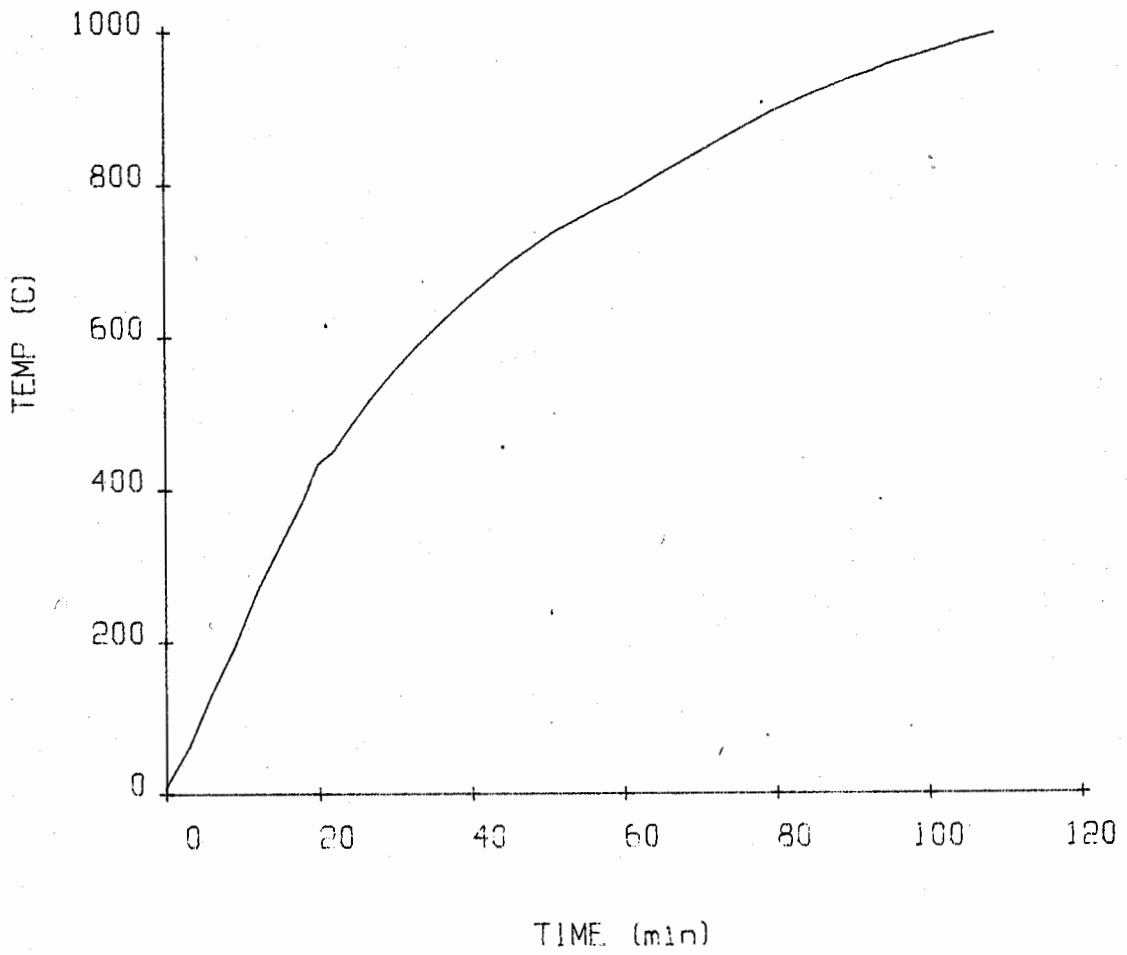
CONDITION OF THE 73 kg/m COLUMN AFTER TESTING

FIG. 15



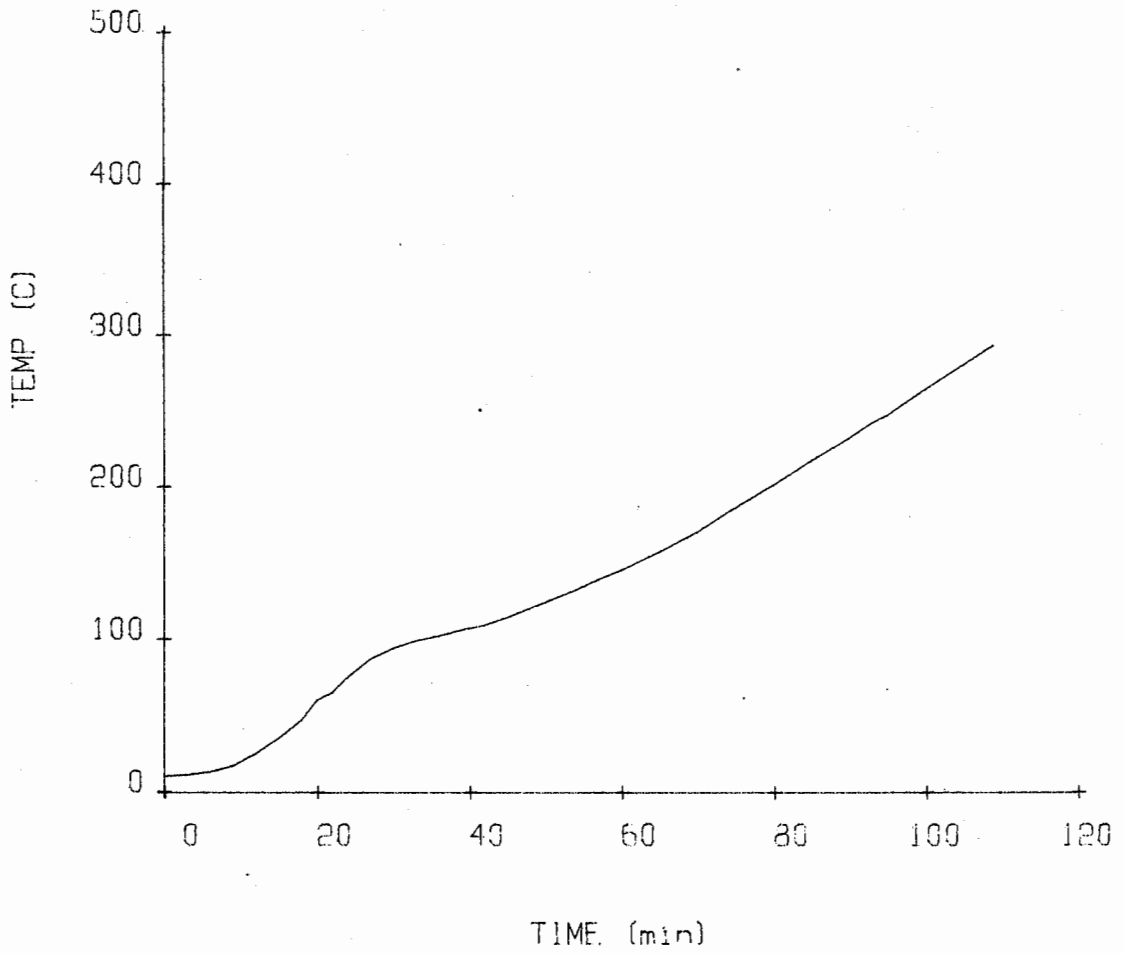
CENTRAL VERTICAL DEFLECTION RECORDED DURING THE TEST  
ON THE 254 x 254 mm x 89 kg/m COLUMN

FIG. 16



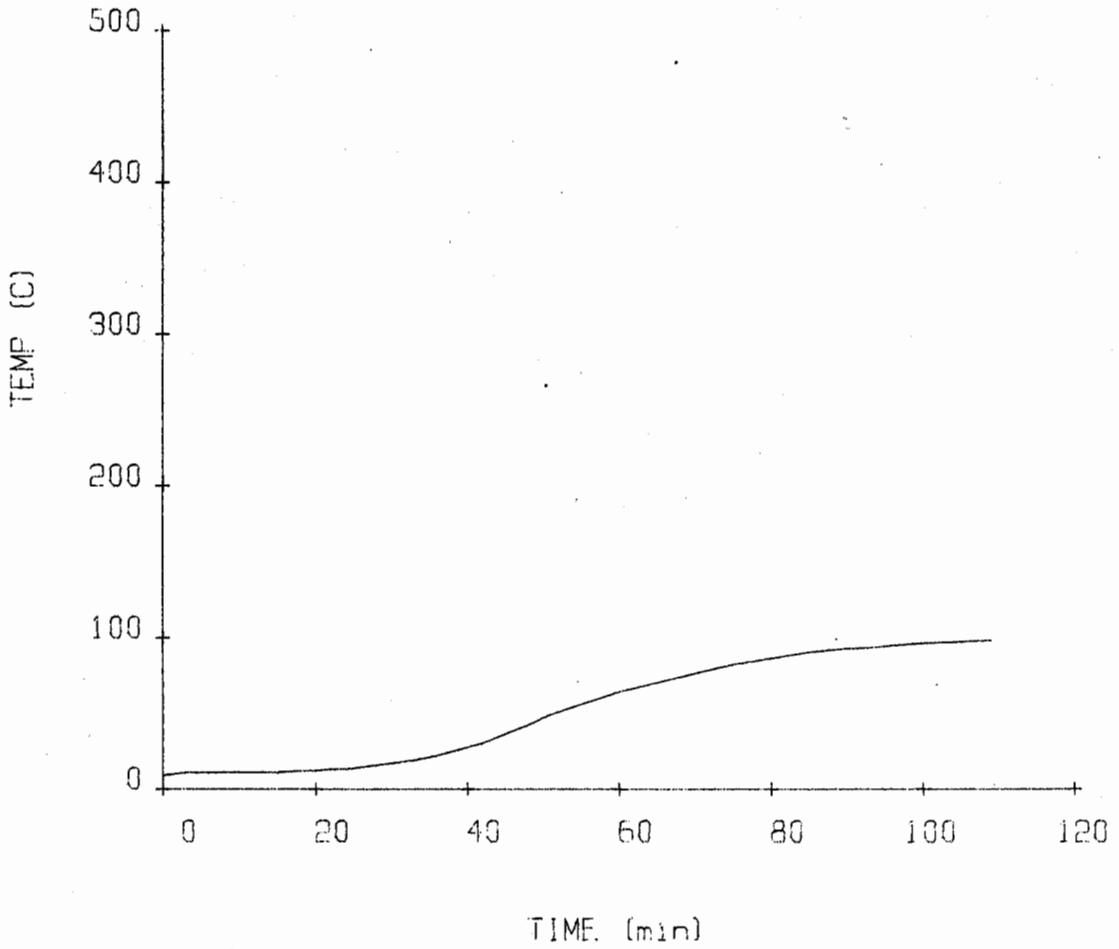
AVERAGE LOWER FLANGE TEMPERATURE  
RECORDED ON THE 89 kg/m COLUMN

FIG. 17



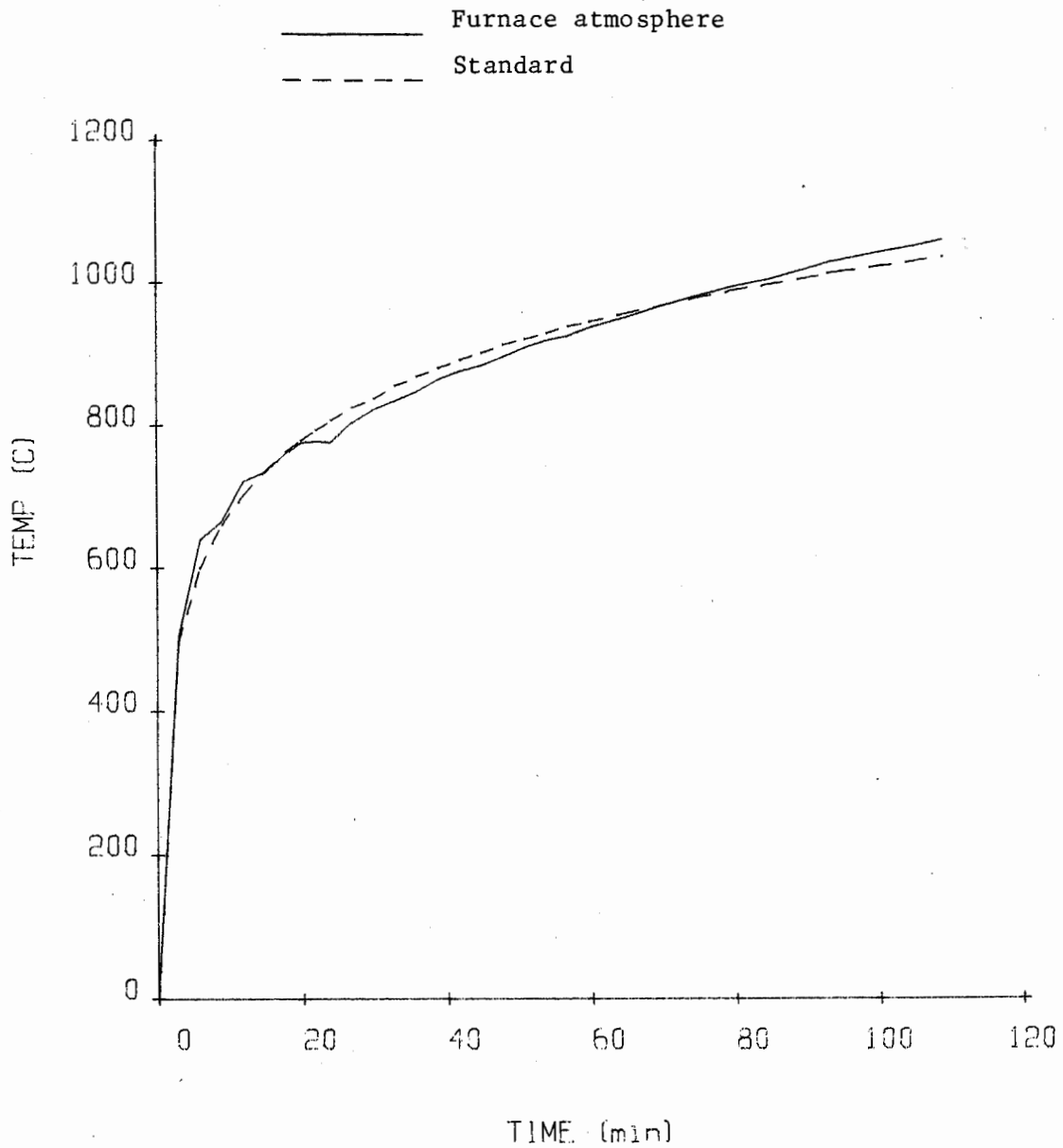
AVERAGE TEMPERATURE OF THE FULLY PROTECTED WEB  
RECORDED ON THE 89 kg/m COLUMN

FIG. 19



AVERAGE UPPER FLANGE TEMPERATURE  
RECORDED ON THE 89 kg/m COLUMN

FIG. 20



COMPARISON OF THE FURNACE ATMOSPHERE TEMPERATURE  
WITH THE INTERNATIONAL TEMPERATURE/TIME CURVE

FIG. 21



CONDITION OF THE 89 kg/m COLUMN  
AFTER THE TEST

FIG. 22

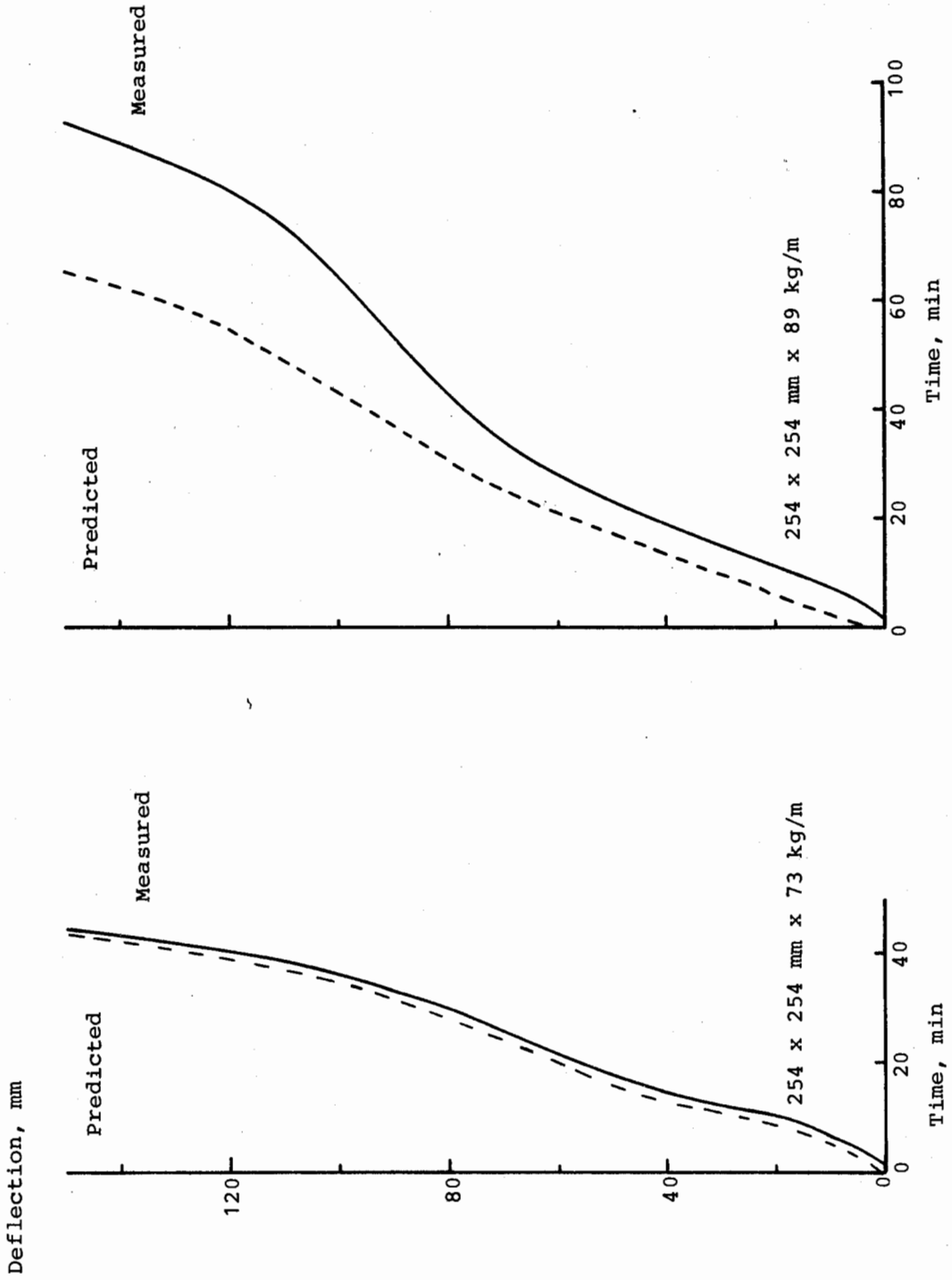


FIG. 23  
(R2/5476)

COMPARISON BETWEEN PREDICTED AND MEASURED DEFLECTIONS OF  
SLIM FLOOR ASSEMBLIES IN BS476:PT. 8 FIRE TESTS



APPENDIX 1A1.1 SLIM FLOOR - 254 x 254 mm x 73 kg/m - TEST CONDITIONS

Span = 4.5 m

I = 11 360 x 10<sup>4</sup> mm

Distance from neutral axis to lower flange = 130.2 mm

Weight of beam = 3.22 kN

Weight of concrete and spreader beams = 72 kN

Live load applied to each flange = 164.5 x  $\frac{1.1}{1.6}$  = 113.1 kN

Dead load on each flange = 18 kN

(a) UDL Case

Compressive bending stress/flange due to live load

$$\frac{113.1 \times 73 \times 6 \times 1000}{4.5 \times 1000 \times (14.2)^2} = -54.6 \text{ N/mm}^2$$

Compressive bending stress/flange due to dead load

$$\frac{18 \times 73 \times 6 \times 1000}{4.5 \times 1000 \times (14.2)^2} = -8.7 \text{ N/mm}^2$$

$$\text{Total} = -63.3 \text{ N/mm}^2$$

$$\text{Longitudinal stress at mid-span} = \frac{wy L^2}{8I} = \frac{365.4 \times 130.2 \times (4500)^2}{1000 \times 4.5 \times 8 \times 11\,360 \times 10^4}$$

$$= 171.1 \text{ N/mm}^2$$

∴ Combined stress on lower flange =

$$\sigma_o = \frac{1}{\sqrt{2}} [(234.4)^2 + (63.3)^2 + (171.1)^2]^{\frac{1}{2}}$$

$$= 209 \text{ N/mm}^2$$

(b) Point Load CaseCompressive bending stress/flange = -36.0 N/mm<sup>2</sup>

∴ Combined stress on lower flange =

$$0.707 [(207.1)^2 + (36.0)^2 + (171.1)^2]^{\frac{1}{2}}$$

$$= 191 \text{ N/mm}^2$$

A1.2 SLIM FLOOR - 254 x 254 mm x 89 kg/m BEAM - TEST CONDITIONS

Span = 4.5 m

 $I = 14\,307 \times 10^4 \text{ mm}^4$ 

Distance from neutral axis to lower flange = 130.2 mm

Weight of beam = 3.93 kN

Weight of concrete and spreader beams = 72 kN

Live load applied to each flange =  $\frac{340}{2} \times \frac{1.1}{1.6} = 116.895 \text{ kN}$ Dead load on each flange =  $\frac{72}{2} \times 0.5 = 18 \text{ kN}$ (a) UDL Case

Compressive bending stress/flange due to live load

$$\frac{116.875 \times 73 \times 6 \times 1000}{4.5 \times 1000 \times (17)^2} = 39.36 \text{ N/mm}^2$$

Compressive bending stress/flange due to dead load

$$\frac{18 \times 73 \times 6 \times 1000}{4.5 \times 1000 \times (17)^2} = 6.06 \text{ N/mm}^2$$

$$\text{Total} = -45.4 \text{ N/mm}^2$$

Longitudinal stress at mid-span =

$$\sigma = \frac{wy L^2}{8I} \frac{\left(\frac{273.67}{4500}\right) \times 130.2 \times (4500)^2}{8 \times 14\,307 \times 10^4}$$

$$= 140.1 \text{ N/mm}^2$$

∴ Combined stress on lower flange

$$\begin{aligned} \sigma_o &= \frac{1}{\sqrt{2}} [(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2]^{\frac{1}{2}} \\ &= 0.707 [(185.5)^2 + (45.4)^2 + (140.1)^2]^{\frac{1}{2}} \\ &= 167.5 \text{ N/mm}^2 \quad (174.1) \end{aligned}$$

(b) Point Load CaseCompressive bending stress/flange = -25.74 N/mm<sup>2</sup>

∴ Combined stress on lower flange =

$$\begin{aligned} &0.707 [(165.8)^2 + (25.74)^2 + (140.1)^2]^{\frac{1}{2}} \\ &= 154.5 \text{ N/mm}^2 \quad (156.4) \end{aligned}$$

## APPENDIX 2



# WARRINGTON RESEARCH CENTRE

Fire Research, Testing and Consultancy

Warrington Research  
Consultants (Services) Limited  
Holmesfield Road  
Warrington WA1 2DS  
Tel: Warrington (0925) 55116  
Telex: 626743 WARRES G

WARRES No. 36438 - LH/LMC  
16 July 1985

**British Steel Corporation**  
Sheffield Laboratories  
Swindon House  
Moorgate  
ROTHERHAM.

Dear Sirs,

### FIRE RESISTANCE TEST RESULTS

We confirm the results of a fire resistance test carried out on your behalf in accordance with B.S. 476 : Part 8 : 1972 on a steel beam of serial size 254 mm by 254 mm by 73 kg/m, Grade 43A which supported precast prestressed concrete slabs, supplied by Richard Lees, of overall size 1550 mm long by 590 mm wide by 200 mm deep, which contained three circular shaped hollow cores of approximate size 130 mm diameter along the length of the slabs. The upper edge of the concrete slabs adjacent to the steel beam was tapered to 100 mm deep over a distance of 250 mm. The concrete slabs were positioned on each side of the beam. The concrete slabs were supported on the lower flange of the steel beam over a distance of 75 mm. A total load of 328.6 kN was applied to the concrete slabs at 1/8, 3/8, 5/8 and 7/8 span positions. The load was calculated by the sponsor to be 100% of the maximum allowable for the beam. The loading was applied at a distance of 500 mm away from the centre line of the beam on each side of the beam. The gap between the ends of the concrete slabs and the web of the steel beam was filled with a dry sand. The soffit of the steel beam was unprotected. The test results were as follows:

Stability : 44 minutes (Test discontinued)  
Re-load Test : Satisfied  
Date of Test : 11 July 1985

A survey of the specimen was performed prior to the test being conducted, but, if you have not already done so, you are asked to provide an accurate written specification of the specimen tested together with detailed drawings to supplement the survey information.

A FULL REPORT IS UNABLE TO BE PROVIDED UNLESS A DETAILED SPECIFICATION OF THE TEST SPECIMEN HAS BEEN PROVIDED.

Yours sincerely,

**L. HEALEY**  
Technical Officer



# WARRINGTON RESEARCH CENTRE

Fire Research, Testing and Consultancy

Warrington Research  
Consultants (Services) Limited  
Hornesfield Road  
Warrington WA1 2DS  
Tel: Warrington (0925) 55116  
Telex: 628743 WARRES G

OUR REF: 36155 - LH/LMC  
1 May 1986

British Steel Corporation  
Sheffield Laboratories  
Swindon House  
Moorgate, ROTHERHAM

Dear Sirs,

## FIRE RESISTANCE TEST RESULTS

We confirm the results of a fire resistance test carried out on your behalf in accordance with BS 476: Part 8: 1972 on a slim floor construction comprising a steel beam of serial size 254 mm by 254 mm by 89 kg/m, Grade 43A which supported precast concrete slabs, of overall size 1550 mm long by 550 mm wide by 200 mm deep. The concrete slabs were positioned on each side of the beam over a distance of 75 mm. A total load of 342 kN was applied to the concrete slabs at  $1/8$ ,  $3/8$ ,  $5/8$  and  $7/8$  span positions. The load was calculated by the sponsor to be 93% of the maximum allowable for the beam, assuming point loading on the lower flange. The loading was applied at a distance of 500 mm away from the centre line of the beam on each side of the beam. The gap between the ends of the concrete slabs and the web of the steel beam was filled with a dry sand. The soffit of the steel beam was unprotected. The test results were as follows:

Stability : To L/30	: 93 minutes
Stability : To L/20	: 109 minutes (Test discontinued)
Re-load Test	: Satisfied
Date of Test	: 29 April 1986

A survey of the specimen was performed prior to the test being conducted, but, if you have not already done so, you are asked to provide an accurate written specification of the specimen tested together with detailed drawings to supplement the survey information.

A FULL REPORT IS UNABLE TO BE PROVIDED UNLESS A DETAILED SPECIFICATION OF THE TEST SPECIMEN HAS BEEN PROVIDED.

Yours faithfully

L. HEALEY  
Technical Officer -  
Structural Fire Protection  
WARRINGTON RESEARCH CENTRE

WP Ref. 155

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Swinden Laboratories

General Steel Products Group  
Standard Circulation  
Dr. B.R. Kirby

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Mr. G. Hogan  
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Mr. J.T. Robinson

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