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TEESSIDE LABORATORIES

Report No	T/RS/1189/28/82/C
Date	28 APRIL 1982
Classification	OPEN

The Fire Resistance of an Unprotected Steel Column Built into a Fire Resistant Wall

British Steel Corporation

Research Organisation

CIRCULATION

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BRITISH STEEL CORPORATION
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SYNOPSIS

A BS476:Part 8 fire test has been performed on a pair of BS4360: Grade 43A, 203 x 203 mm x 52 Kg/m columns built into a double skin cavity wall. The flange and part of the web of the unprotected steel sections were exposed to the fire. The sections were loaded to over 50% of the maximum design stress, and the test was discontinued after 103 minutes when the outer wall exhibited large horizontal cracks and was on the point of structural failure.

The fire resistance of the construction was much greater than expected, and at failure the flange exposed to the heating environment had reached a temperature of around 1000°C, whilst the concealed flange was significantly cooler at ~250°C.

The cooler inner flange and web made a significant contribution to the load bearing capacity of the structure.

The significance of the results for other columns in walls is considered as well as other partially exposed steel constructions.

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THE FIRE RESISTANCE OF AN UNPROTECTED STEEL COLUMN BUILT INTO A FIRE RESISTANT WALL

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THE FIRE RESISTANCE OF AN UNPROTECED STEEL COLUMN BUILT INTO A FIRE RESISTANT WALL

1. INTRODUCTION

The procedures used in the BS476:Part 8 fire testing of elements generally limit themselves to the evaluation of single elements, e.g. a beam, a column or a wall. The tests performed seldom include a combination of elements and consequently neglect the interactions of various forms of construction. Frequently, combinations of elements would be expected to have improved fire resistance over the single elements, e.g. composite steel/concrete beams, shelf angle floors or columns built into walls.

The present report describes a BS476:Part 8 fire test performed on a pair of BS4360:Grade 43A, 203 x 203 mm x 52 Kg/m columns which were built into a standard cavity wall. The columns were loaded, but no load was applied to the wall. The heating rates of the columns were monitored along with the deflections of the columns. The significance of the test results on design for columns and walls in single and multi-storey buildings are discussed.

2. THE TEST SPECIMEN

The specimen was specially constructed for testing under load in the wall furnace at FIRTO. Special design features were required to accommodate this form of specimen in the furnace.

The load is applied to a wall using a pair of hydraulic rams at either side of the furnace through pads at the top and bottom of the furnace. To ensure the load was applied axially to the steel columns it was necessary to test two identical columns located at the $^1/_3$ and $^2/_3$ positions across the furnace.

To generate some degree of base fixity it was necessary to fix the columns to a steel base plate 580 mm x 3048 mm x 20 mm thick. The columns had welded end plates 406 mm square and these base plates were welded to the base plate. The bottom of the construction was then cast into a block of lightweight concrete of size approximately 580 x 3050 x 250 mm.

A similar 580 mm x 3048 mm x 20 mm plate was used at the top of the columns to restrict lateral movement of the columns during the test, and to ensure equal load distribution between the two columns. The columns had 406 x 406 mm square plates welded to the ends tops and these plates were bolted (4 bolts per column) to the upper restraining plate.

The masonry part of the wall/column construction was intended to be fire tested with no imposed load. Under BS476:Part 8 requirements, non-load bearing constructions should be tested with the edges of the construction restrained. Restraining the edges of walls was not possible without producing a loading path to reduce the load applied to the columns, and it was necessary to check that the proposed form of construction satisfied the requirements of both the Fire Research Station and the Greater London Council for assessment purposes.

The cavity walls were constructed using conventional practices. The outer skin was of Fletton brick and a 12 mm gap was left between this outer skin and the concealed flange of the column. A 50 mm gap was left between the Fletton wall and the lightweight concrete internal block wall. Wall ties were utilised between the walls, and the joint between the blockwork wall and the web of the column was sealed with the sand/cement plaster. A 50 mm gap was left between the top of the brick and block walls and the upper restraining plate.

The outer 62.5 mm of web and flange was exposed to the fire. Figures 1-6 show the details of the construction. Figure 1 shows columns and baseplates in the early stages of fabrication, and Figure 2 shows the final wall on the furnace and unexposed sides of the structure. Figures 3 and 4 are diagrams of the construction used showing precise details of the location of steel, bricks and blocks.

To protect the upper restraining plate during the fire test, the chicken wire mineral wool arrangement, shown in Figures 5 and 6, was constructed.

From the laboratory side of the construction it was impossible to note the deflection of the columns during the test. To facilitate deflection monitoring a 100 mm long bolt, 20 mm dia. was welded to the concealed flange of the column, and this bolt protruded through the outer brick wall. The movement of the end of this bolt was monitored using a dial gauge supported from the furnace frame. This gauge measured the deflection of the columns into and away from the furnace. The standard FIRTO equipment was used to measure the vertical deflection of the columns at either side of the furnace frame.

The heating rates of the steel members were monitored in forty locations, i.e. twenty thermocouples were used on each column (all 3 mm diameter Pyrotenax thermocouples chromel/alumel with insulated hot junctions). Five thermocouples were used on each of the flanges exposed to the fire and the concealed flanges. Ten thermocouples were used on the web, five in the part of the web exposed to the fire, and five near the cavity between the block and brick walls.

Six additional thermocouples were utilised to monitor the furnace atmosphere heating rate during the test.

3. THE TEST LOAD

The load selected for this test was approximately 50% of the maximum design load.

The load in any column is made up of a number of components depending upon the situation and building type. The main components come from dead loads, superimposed roof and floor loads and wind loads. At the time of a fire it is realistic to assume that many of these will be reduced or even absent.

For a typical single-storey steel framed building the following design loadings are utilised:

	kN/m^2
Superimposed load (snow)	0.75
Cladding and purlins	0.20
Roof structure	0.15
Services	0.20
	1.30 kN/m^2

It can be seen that the superimposed loading is 58% of the total and that the allowance for services, which is rarely fully realised, is 15%. At the time of a fire the load could therefore be as low as 27% of the design load.

In multi-storey structures the reductions would be smaller and the amount of reduction would diminish as the building height increased.

It is common practice to utilise the same column size over more than one storey height. Consequently in the upper storeys the columns would be subjected to loads less than their allowable maxima.

It was therefore decided that it would not be unreasonable to understress the column down to 50% of its allowable capacity. In the event calculations made after the test show that the figure was 53%.

The loading calculations are shown below:

Applied load was 952.8 kN for 2 columns

Constants

203 x 203 mm x 52 Kg/m Universal Column, BS4360 Grade 43A.

Length = 3 m. r_{xx} = 8.9 cm. r_{yy} = 5.16 cm

For xx use 0.85 effective length (estimate)

Hence: $$\frac{\ell}{r_{xx}} = 0.85 \times 300 \times \frac{1}{8.9} = 28.7$$

For yy use 0.75 effective length (BS449 Cl. 31b and Appendix D).

$$\frac{\ell}{r_{yy}} = 0.75 \times 300 \times \frac{1}{5.16} = 43.6$$

$\therefore$ yy governs and Pc = 137 N/mm^2

BS449 max load $= 137 \times 66.4 \times 10^2 \times 10^{-3}$

$= 909.7$ kN

$\therefore$ % max $= \frac{952.8}{2 \times 909.7} \times 100$

$= \underline{52.4\%}$

4. RESULTS

The test was discontinued after 103 minutes, when it was thought that the outer Fletton brick wall was on the point of collapse. Two large horizontal cracks were present in the wall at the end of the test, and significant deformation was apparent so that collapse may have occurred in a sudden and catastrophic manner.

Figure 7 is a photograph of the wall at the end of the test showing the deformation of the wall/column construction.

The results of deflection measurements are shown in Figure 8.

The columns showed a small expansion in the early stages of the test.

The dial gauge indicated that the column moved towards the furnace in the early stages of the test, and then it moved towards the laboratory, the final central deflection being of the order of 56 mm.

The results of temperature measurement are shown in Figures 9-13.

At the end of the test the outer flanges of both columns were heated to temperatures in the range 925-1026°C, whilst the concealed flanges were much cooler with temperatures in the range 149-322°C.

At the exposed web measurement location temperatures were in the range 882-985°C, whilst in the inner web location temperatures were within the range 233-512°C.

Detailed summaries of the steel heating data are presented in Table 1.

Following the test, detailed observations were made of the extent of deformation of the columns and the relative positions of the upper plate and the block and brick walls.

A photograph of the top of the construction after the test is shown in Figure 14.

5. DISCUSSION

The fire resistance time of the construction was significantly longer than expected on the basis of single element tests.

A BS476 test on an 8" x 6" joist of similar dimensions (200 x 150 mm x 52 Kg/m) was reported(1) to fail after 11 minutes presumably when fully loaded.

Fully loaded BS4360 Grade 43A columns generally fail the BS476:Part 8 fire tests performed under full load when their temperature exceeds 550°C(2), and a specimen loaded to 50% of the maximum design stress failed when its temperature reached 650°C(3).

The present test has demonstrated the beneficial effect of the wall on the fire resistance of the column. Based upon the failure temperatures and heating rates expected for single element tests, a failure time of 13 minutes would be predicted, however a fire resistance of over 103 minutes was recorded. The wall prevents flame impingement on the concealed flange and this part of the section was only heated through conduction from the web. This heat path was not effective, and the temperatures recorded were much lower than those for the outer flange.

The thermal gradient through the sections enabled them to remain stable, the cooler concealed flanges being capable of supporting considerable loads.

Based upon the mean temperatures recorded and dividing the section into four segments as shown in Figure 15, the "approximate" load bearing capacity of the column may be calculated (based upon uniaxial compression). These calculations, shown in Appendix 1, indicated that the construction was easily capable of supporting the imposed load. In fact the calculated load bearing capacity of the column suggests that the column built into the wall could have supported the full test load (provided instability did not occur because of bending).

The bending of the column towards the laboratory in the later stages of the test meant that the column and outer wall were in direct contact, and clearly the wall prevented further bending of the column, and was supporting part of the test load at the end of the test.

This behaviour would be observed in part in a real fire, and hence the contribution of the wall towards the fire resistance of the construction should be recognised.

In a real fire any beam attached to the column would expand, and hence the column would be subjected to higher bending stresses than those encountered in this test.

It should also be recognised that it is conventional practice with single-storey portal frames to use cavity wall construction to approximately half the height of the column, and plastic coated sheet steel above this level. Therefore the significance of the outer wall would need to be considered in more detail if this result were applied to a brick/sheet steel wall.

Whilst the test proved an effective demonstration of the fire resistance of steel built into a wall, many different forms of construction could be encountered. For instance:

Different sizes of steel beam or column with different extents of exposure

Different brick or block materials or completely different wall systems

Different load levels

In order to be able to predict the stability of various different constructions it would be necessary to develop structural and thermal models to facilitate estimation of fire resistance. The thermal model would predict the temperature gradients through the section, whilst the structural model would include determination of the load bearing capacity of a column with very large temperature gradients. A key feature of the present test was the bending of the column, and the model should include assessment of the extent of bending, because of the influence of column deflection on the stability of the wall.

Some preliminary work has been completed on the development of a thermal model, and the work carried out to date will be described in a separate report to be published in the near future[4].

6. FUTURE WORK

Resources should be devoted to assessing the fire resistance of other but similar forms of construction, i.e. different sections, different degrees of exposure, different loading and different walls. These tests will provide confidence in the present result and establish an empirical base for the development of theoretical models.

Further development of the heat flow and structural models is essential to facilitate evaluation of the many and varied constructions which will be encountered.

7. OTHER FUTURE WORK

The present study has demonstrated that partially exposed constructions can have considerable fire resistance, particularly where heat transfer is restricted to thermal conduction along the web of the section.

The present test has established a general construction method for columns, but the principle established could also be utilised for beams, and the shelf angle floor unit offers a construction method which could easily be adopted to improve the fire resistance of such a member.

Further work should be directed towards establishing the fire resistance of this construction which also has the advantage of reducing storey height requirements and hence the cladding surface area for the building.

The cost implications need to be examined in detail, but when cladding and fire protection costs are considered, the extra fabrication costs could well be negated.

8. CONCLUSIONS

A BS476:Part 8 fire test has been performed on a pair of unprotected 203 x 203 mm x 52 Kg/m columns built into a double skin brick and block wall. The flange and part of the web of the columns were exposed to the fire and they were loaded to 50% of the maximum design stress.

1. The fire test was discontinued after 103 minutes when large horizontal fissures were observed on the external brick wall and its collapse appeared imminent.

2. Although the columns were deformed they were still supporting the test load when the test was discontinued.

3. The columns had reached a temperature approaching 1000°C on the flanges exposed to the fire, whilst the concealed

flanges situated in the cavity had temperatures around 300°C.

4. The slow build-up of temperature on the unexposed flange was a result of the small conduction path through the web of the section.

5. The cooler inner flanges made a major contribution towards the stability of the construction.

6. Theoretical models must be developed in order to be able to analyse the fire resistance of different constructions.

7. The results of the current test suggest that partially protected steel members can have considerable fire resistance, and future work should be directed towards establishing the fire resistance of these forms of construction, e.g. other column/wall constructions and shelf angle floors.

April 1982

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4. C Stirland
Teesside Laboratories Technical Note. To be published.

FIRTO No.: TE 4081

BSC TEST No.: 39

TEST DATE: 3.11.81

T/RS/1189/28/82/C

COLUMNS IN WALLS TEST, COMPRISING OF AN EXTERNAL FLETTON BRICK WALL AND A BLOCK-WORK BRICK WALL INTO WHICH TWO UNIFORMLY SPACED BS4360 GRADE 43A 203x203mmx52kg/m COLUMNS WERE BUILT

	WEB	FLANGE
YIELD STRESS (N/mm^2)	287	285
TENSILE STRENGTH (N/mm^2)	472	492
% ELONGATION (200mm GL)	28.0	25.5

COMPOSITION	%C	%Si	%Mn	%P	%S	%Cr	%Mo	%Ni	%V	%Ti	%Cu	%Sn	%Nb	%Zr	%Sol Al	%Tot Al	%N
RS198	.27	.041	.94	.010	.022	.01	.005	.026	.005	.005	.018	.005	.005	.005	-	.01	.0033

FAILURE TIME: 103 MINUTES

THERMOCOUPLE LOCATION	TEMPERATURE (°C) AFTER VARIOUS TIMES (MIN)																	
	5	10	20	25	30	40	45	50	55	60	65	70	75	80	85	90	95	100	103
INNER FLANGE 21	33	56	92	112	129	161	176	190	202	213	228	241	254	267	279	290	300	313	322
INNER FLANGE 22	19	34	60	75	95	136	155	172	186	199	217	234	246	258	269	279	288	298	304
INNER FLANGE 23	18	21	41	53	69	105	122	138	151	163	174	186	195	204	213	220	226	232	234
INNER FLANGE 24	17	19	34	46	60	92	108	123	137	148	162	176	187	197	207	215	223	230	233
INNER FLANGE 25	16	17	23	28	36	54	63	71	78	86	95	104	112	120	127	134	140	146	149
AVGE. INNER FLANGE	21	29	50	63	78	110	125	139	151	162	175	188	199	209	219	228	235	245	248
OUTER FLANGE 36	155	283	513	604	668	743	772	804	836	862	887	902	914	928	944	957	970	983	989
OUTER FLANGE 37	133	284	559	665	727	801	838	866	889	908	933	944	953	966	981	993	1004	1014	1018
OUTER FLANGE 38	158	319	570	660	716	777	812	847	873	894	918	927	936	950	968	982	994	1005	1010
OUTER FLANGE 39	148	290	563	664	723	787	821	852	875	896	922	931	939	954	974	989	999	1010	1013
OUTER FLANGE 40	90	172	415	584	677	739	764	790	815	838	874	893	904	919	939	957	969	981	980
AVGE. OUTER FLANGE	137	270	524	635	702	769	801	832	858	880	907	919	929	943	961	976	987	999	1002
INNER WEB 26	37	66	125	150	174	220	241	259	276	292	309	326	340	354	367	379	391	403	411
INNER WEB 27	27	54	118	154	190	250	274	295	314	332	358	378	393	409	424	437	450	464	474
INNER WEB 28	20	33	82	115	144	190	210	228	244	259	277	295	307	320	331	341	350	358	364
INNER WEB 29	20	32	76	106	132	178	199	220	236	251	278	300	315	330	343	355	366	376	382
INNER WEB 30	18	23	43	61	80	112	125	137	148	158	169	181	191	200	209	218	226	234	238
AVE. INNER WEB	24	42	89	117	144	190	210	228	244	258	278	296	309	323	335	346	357	367	374
OUTER WEB 31	121	217	400	478	542	639	670	700	733	765	797	824	843	861	881	899	915	931	940
OUTER WEB 32	101	210	431	535	615	707	747	786	818	845	874	892	906	920	937	952	964	977	983
OUTER WEB 33	94	213	437	537	607	687	722	758	792	821	853	870	883	899	919	935	948	960	966
OUTER WEB 34	74	170	385	486	559	653	694	731	762	791	831	853	867	886	909	926	939	952	958
OUTER WEB 35	51	100	250	365	454	550	584	615	642	671	713	748	768	790	816	840	860	887	882
AVE. OUTER WEB	88	182	381	480	555	647	683	718	749	779	814	837	853	871	892	910	925	941	946
FCE ATM 1	572	652	786	814	845	880	893	910	923	937	946	956	964	980	993	1004	1017	1027	1031
FCE ATM 2	628	687	820	823	862	892	908	924	939	953	963	971	978	997	1012	1022	1034	1047	1045
FCE ATM 3	661	717	833	862	889	900	912	926	938	950	970	976	982	999	1013	1027	1035	1045	1042
FCE ATM 4	565	646	785	797	837	866	882	900	915	928	940	952	960	976	989	1000	1014	1026	1028
FCE ATM 5	441	521	795	817	841	863	875	891	908	918	955	960	966	984	998	1012	1017	1027	1017
FCE ATM 6	666	720	830	836	863	891	908	924	938	951	953	960	970	989	1003	1016	1027	1039	1037
AVE FCE ATM	589	657	808	825	856	882	896	912	927	939	954	962	970	988	1001	1013	1026	1035	1033
ISO CURVE RT19°C	575	677	780	814	841	884	901	917	931	944	956	967	978	987	996	1005	1013	1021	1025

TABLE 1

FABRICATION OF COLUMNS AND BASEPLATES

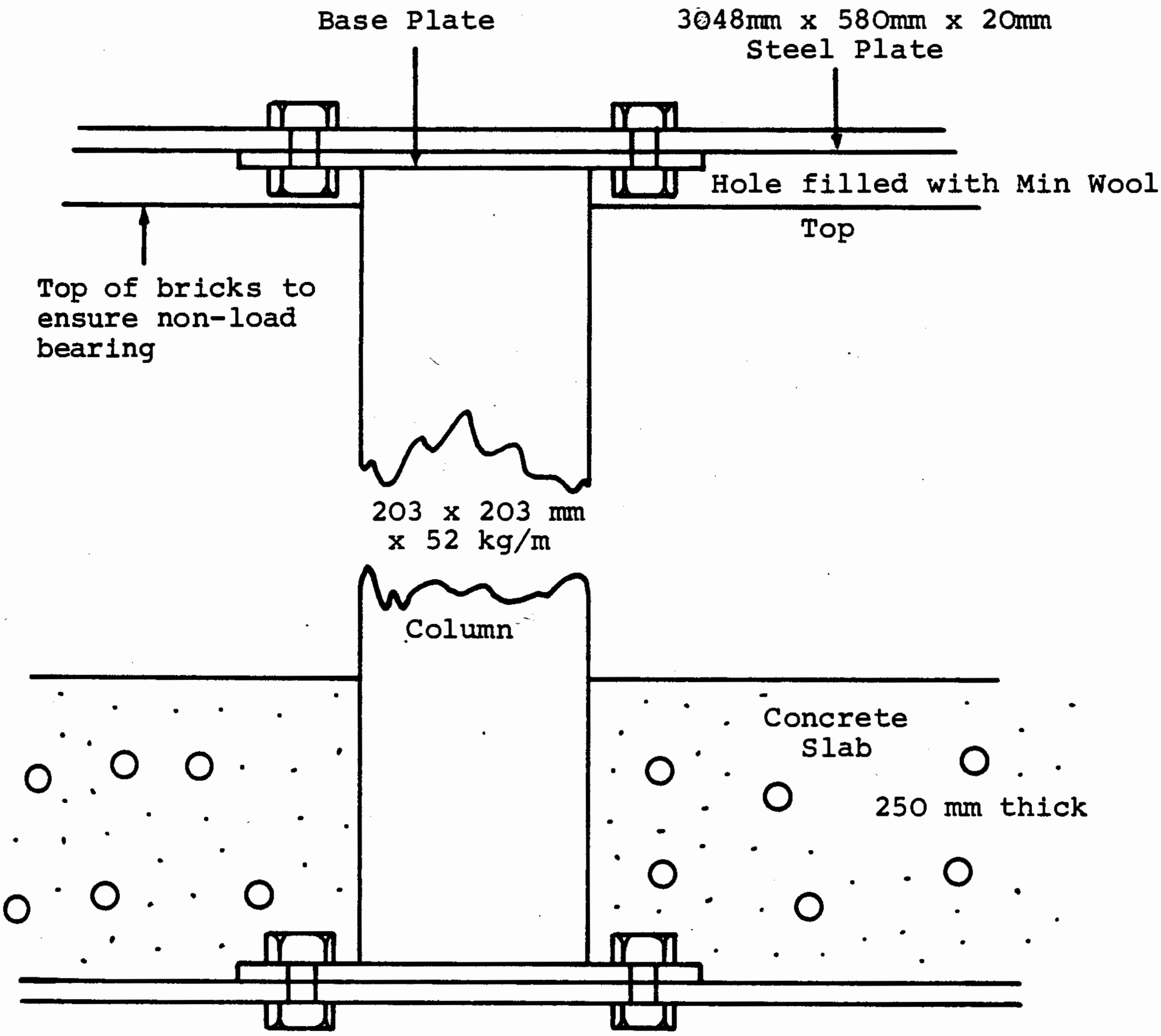
FIGURE 1.

UNEXPOSED SIDE OF CONSTRUCTION

SIDE EXPOSED TO FURNACE ATMOSPHERE

FINAL CONSTRUCTION POSITIONED IN FURNACE SHOWING THE EXPOSED AND UNEXPOSED SIDES

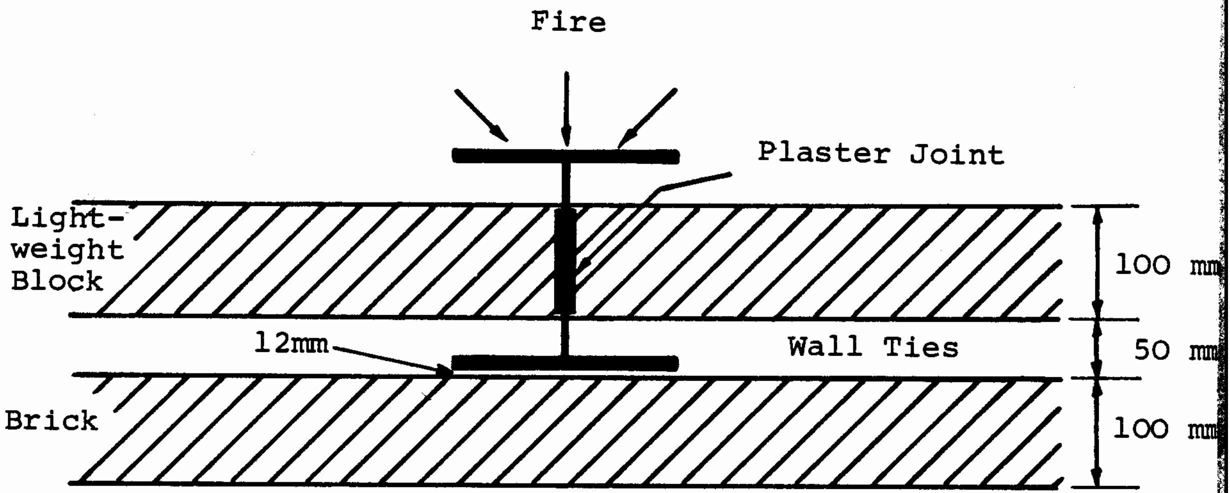
FIGURE 2.

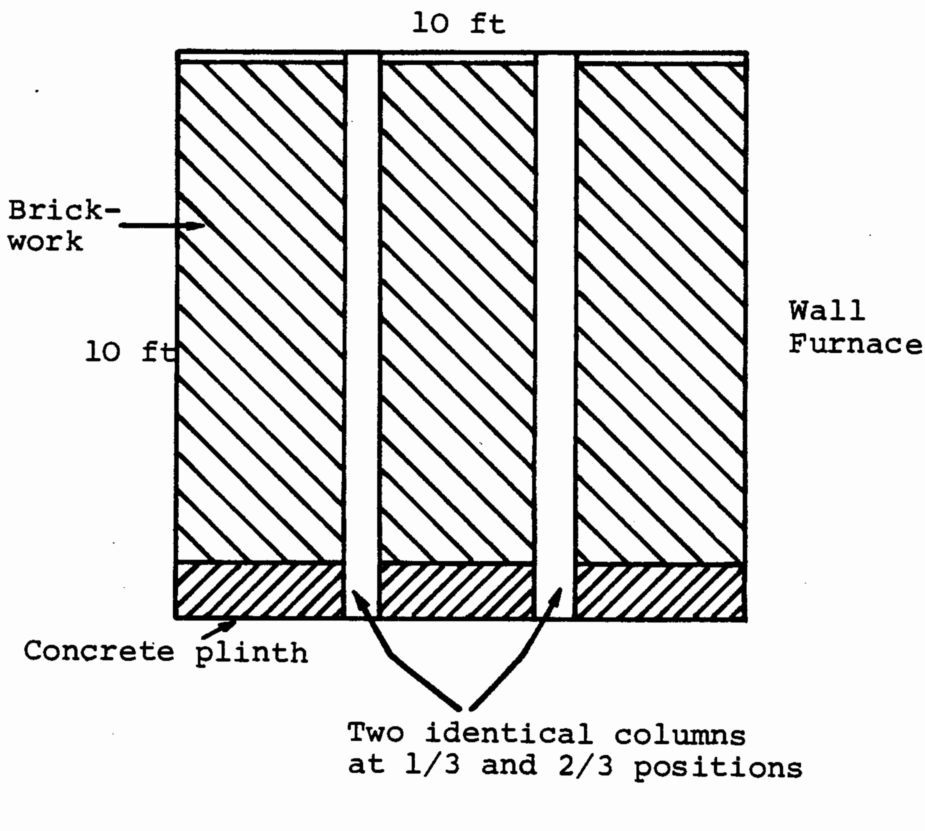


TOP AND BOTTOM DETAIL OF THE COLUMNS

FIGURE 3

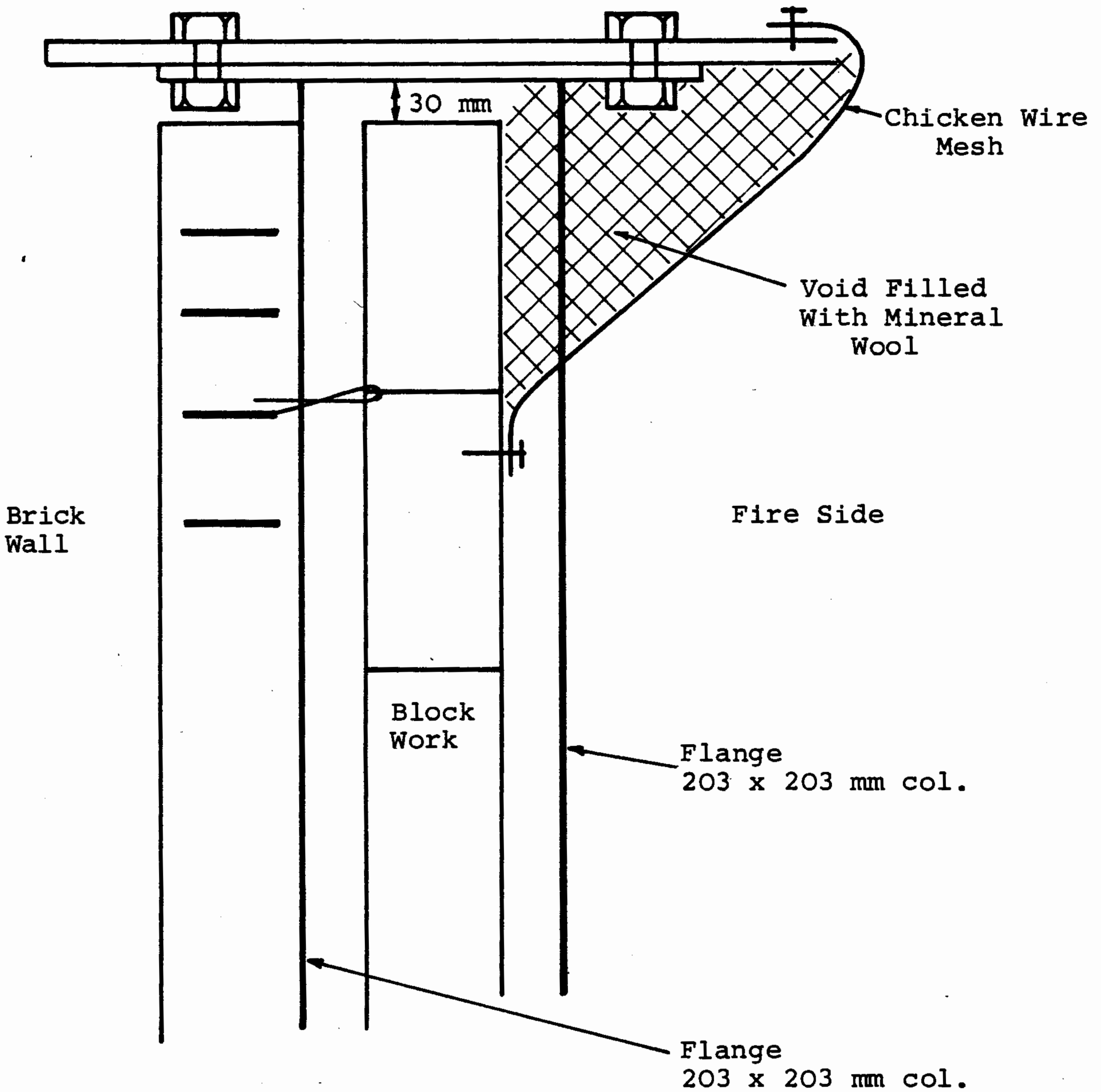
Column Size 203mm x 203mm x 52 kg/m





DETAILS OF THE CONSTRUCTION

FIGURE 4



MINERAL WOOL ARRANGEMENT TO PROTECT UPPER RESTRAINING PLATE

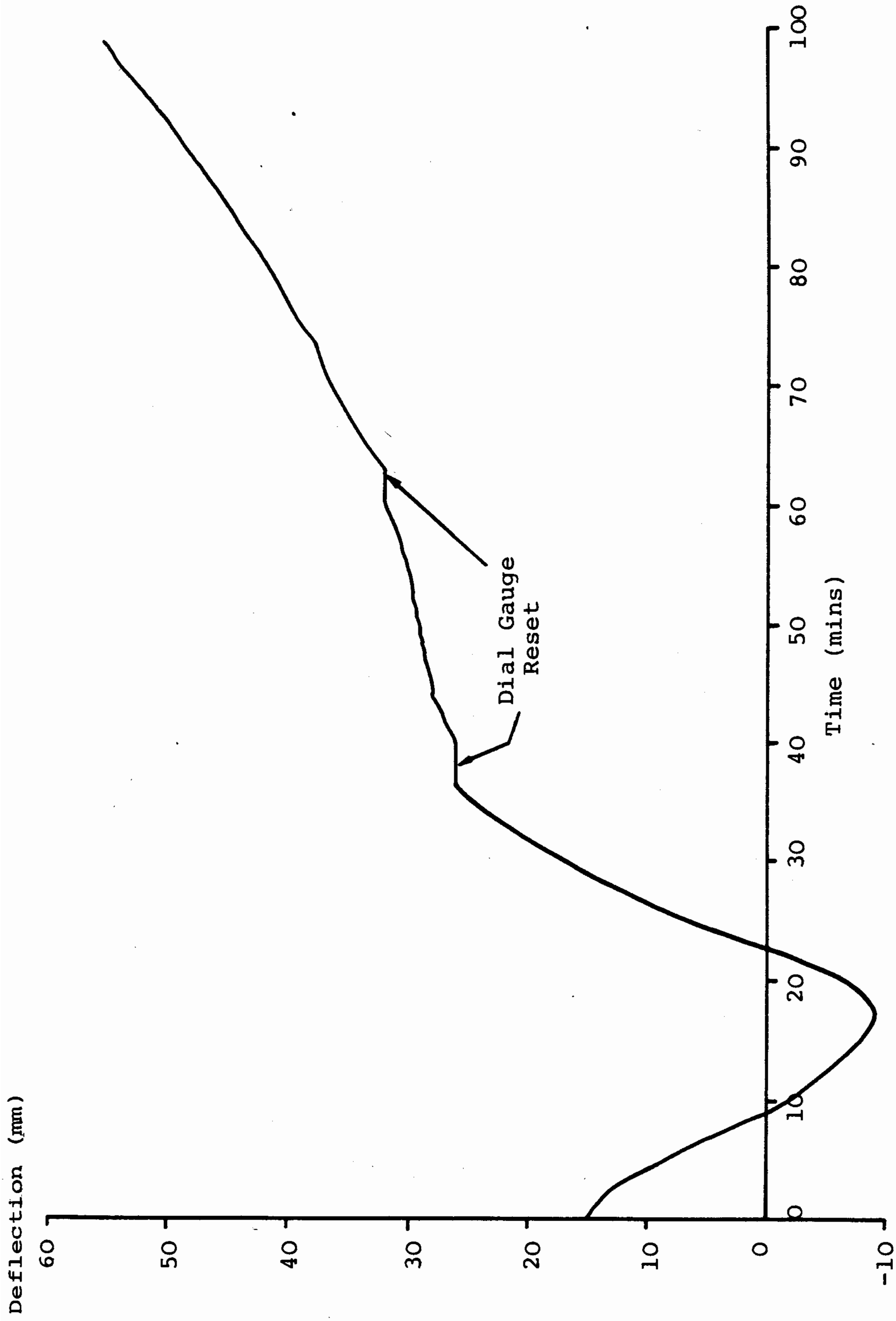
FIGURE 5

THE WALL AT THE END OF THE TEST

FIGURE 7.

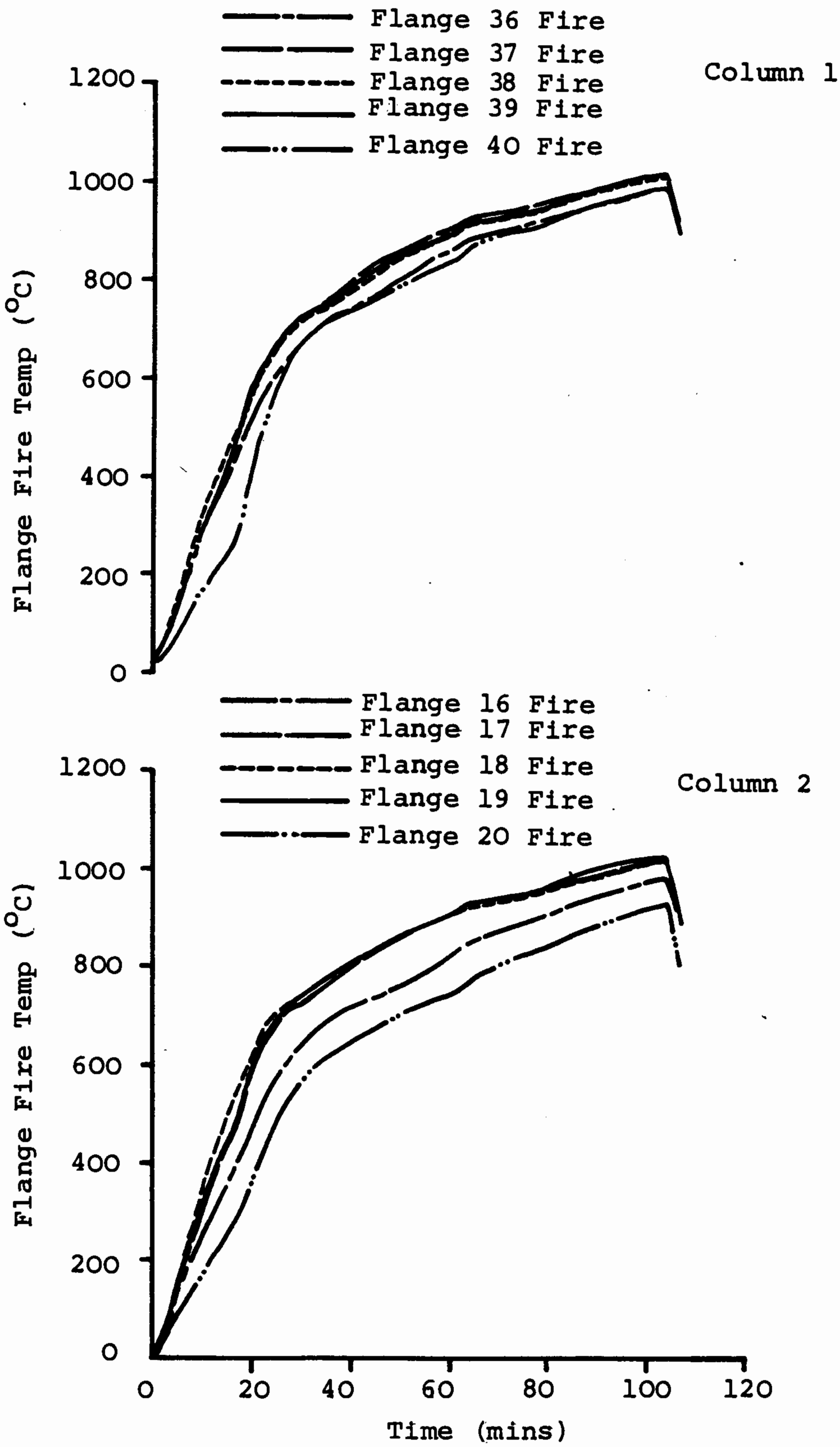
CONSTRUCTION OF CHICKEN MESH AND MINERAL WOOL
TO PROTECT UPPER PLATE

FIGURE 6.



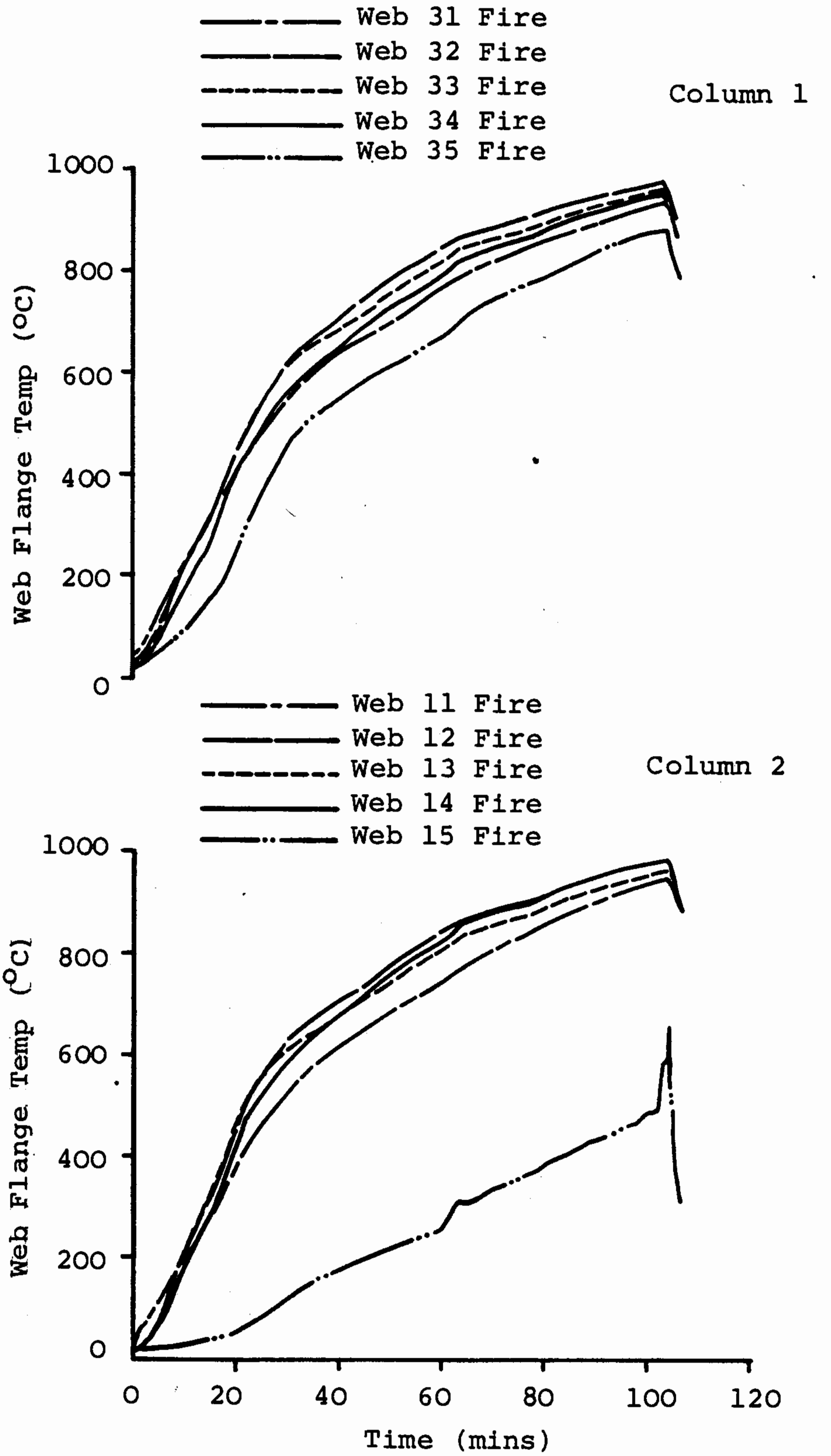
DEFLECTION OF COLUMNS MEASURED DURING THE TEST

FIGURE 8



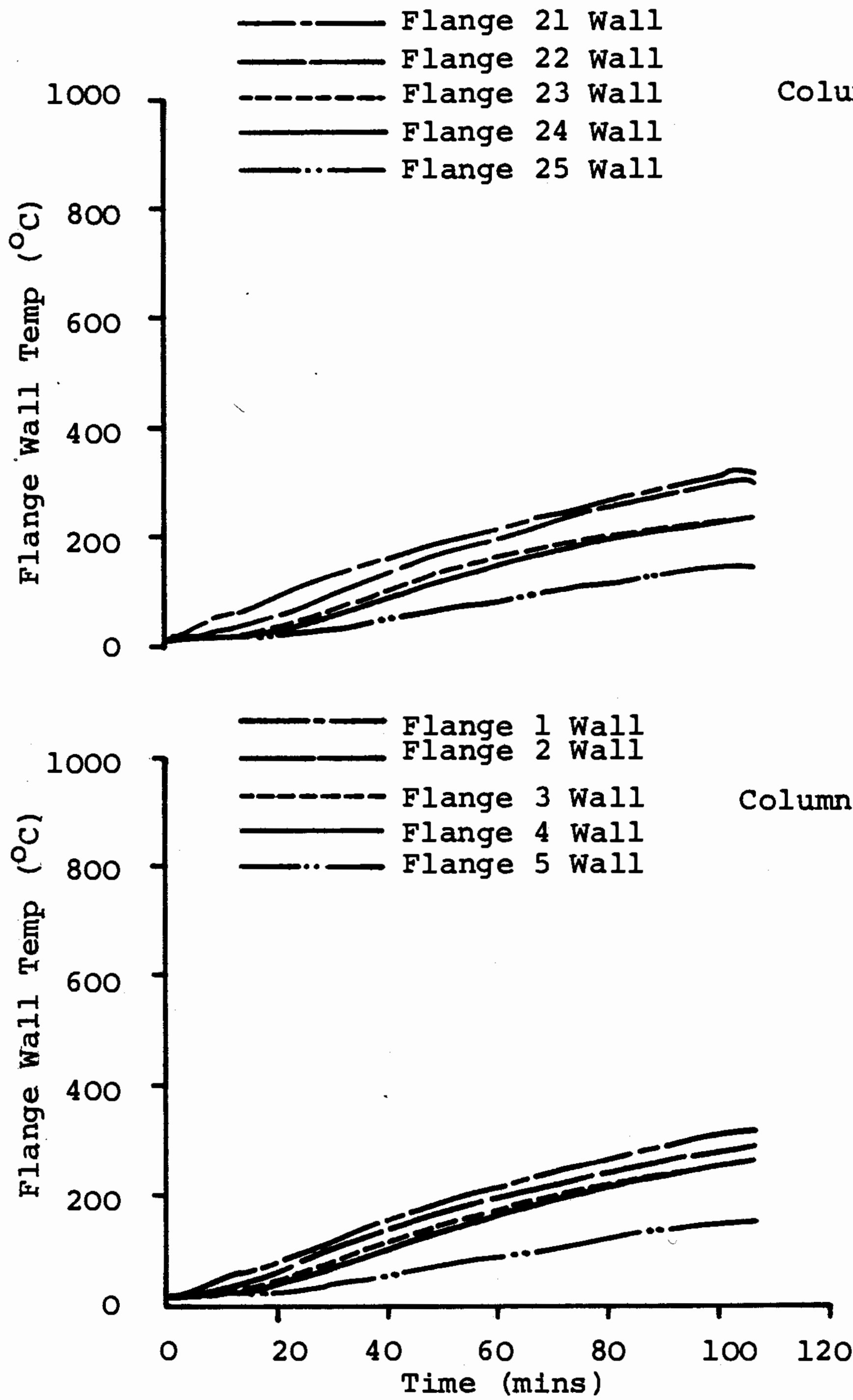
TEMPERATURES RECORDED ON THE OUTER FLANGES OF BOTH COLUMNS

FIGURE 9



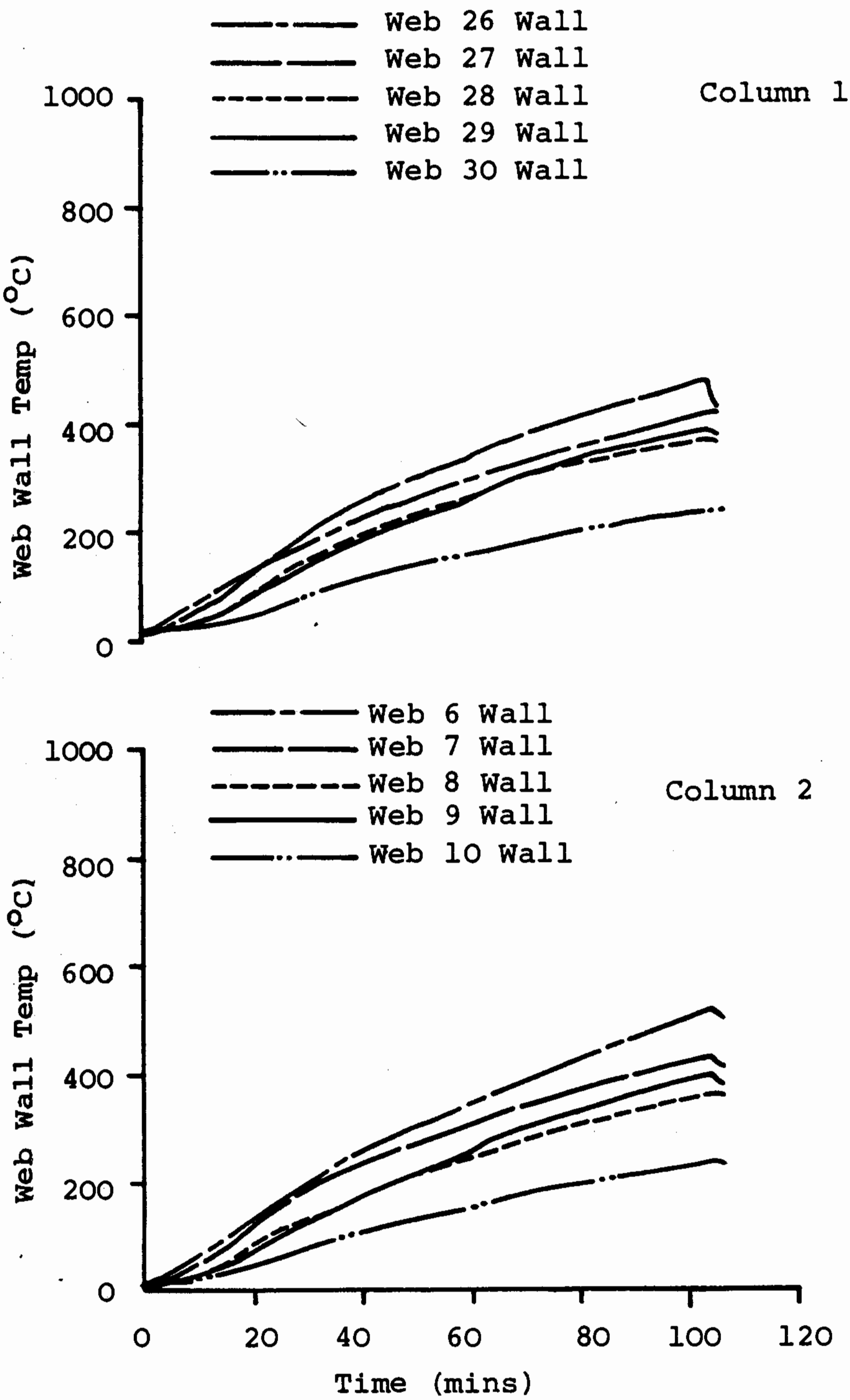
TEMPERATURES RECORDED ON THE OUTER WEBS OF BOTH COLUMNS

FIGURE 10



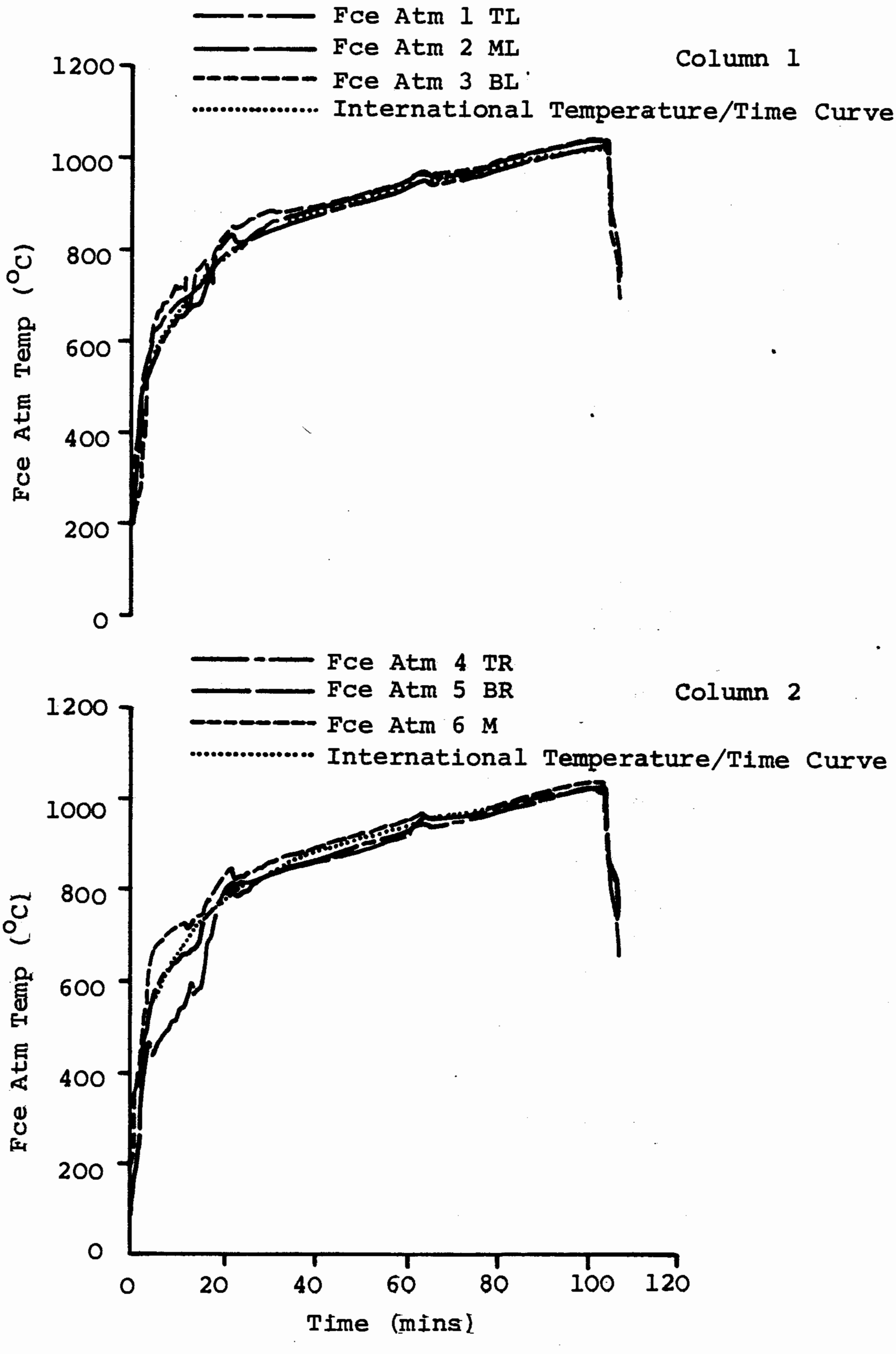
TEMPERATURES RECORDED ON THE INNER FLANGES OF BOTH COLUMNS

FIGURE 1



TEMPERATURES RECORDED ON THE INNER WEBS OF BOTH COLUMNS

FIGURE 12

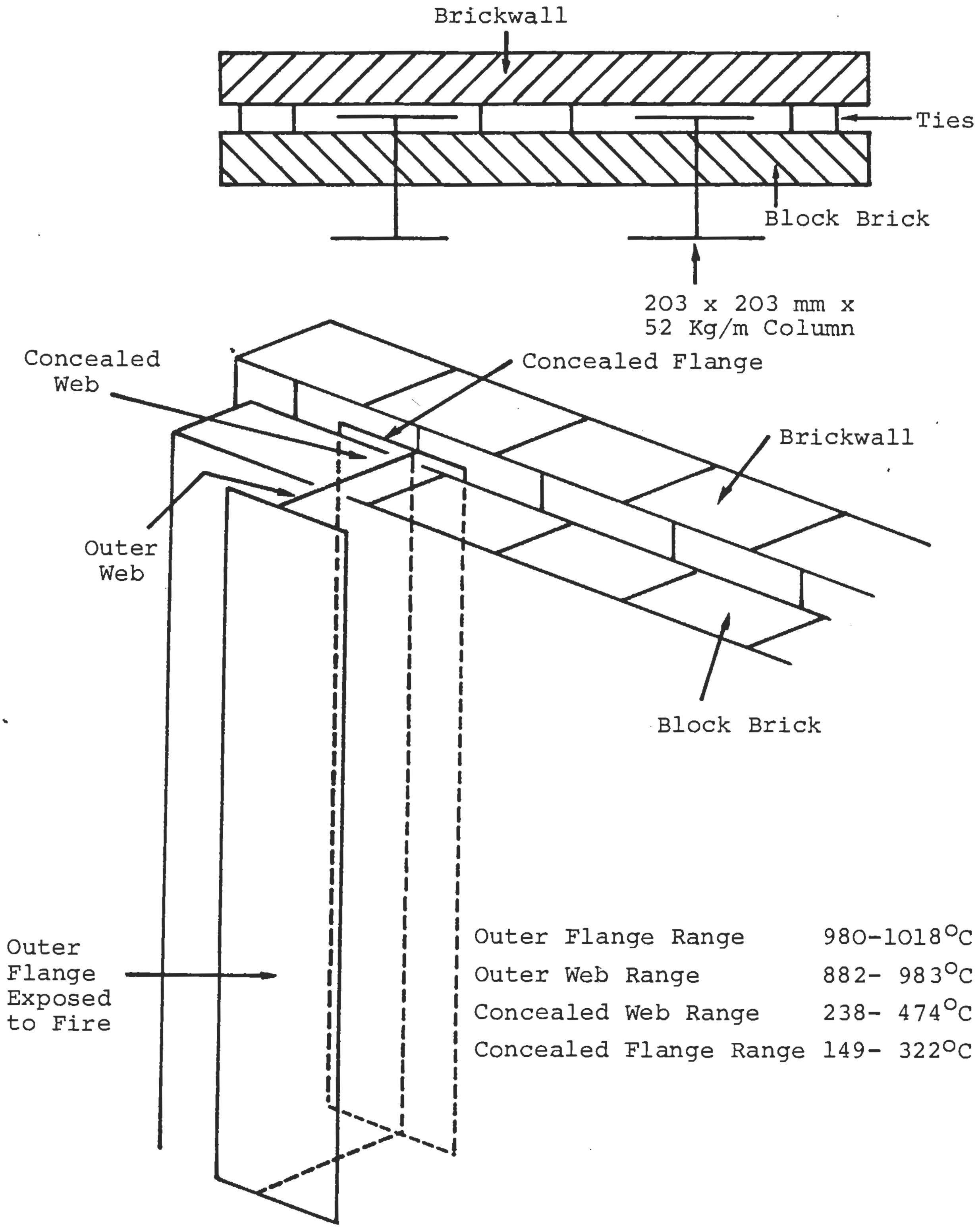


FURNACE ATMOSPHERE TEMPERATURES RECORDED DURING TEST

FIGURE 13

TOP OF THE SPECIMEN AFTER THE TEST, AND REMOVAL OF THE MINERAL FIBRE PROTECTION. NOTE THE GAP BETWEEN THE STEEL PLATE AND THE FURNACE FRAME

FIGURE 14.



STEEL TEMPERATURE OF COLUMN 1

FIGURE 15

APPENDIX

LOAD BEARING CAPACITY OF A COLUMN UNDER AXIAL COMPRESSION AT ELEVATED TEMPERATURE

Divide the column into 4 elements, i.e.

Outer flange
Outer web
Inner web
Inner flange

The areas, temperatures and strengths were as follows:

The 203 x 203 mm x 52 Kg/m column has a flange width of 204 mm and flange thickness of 12.5 mm. The web depth is 206 mm and web thickness is 8.0 mm.

Outer Flange

Area = 204 x 12.5 mm = 2550 mm^2
Temperature = 1000°C
Strength = 18 N/mm^2 (from T/RS/1189/11/80/C)

Load Bearing Capacity = 45.9 kN

Outer Web

Area = 90.5 x 8 mm = 724 mm^2
Temperature = 930°C
Strength = 26 N/mm^2 (from T/RS/1189/11/80/C)

Load Bearing Capacity = 18.8 kN

Inner Web

Area = 724 mm^2
Temperature = 356°C
Strength = 213 N/mm^2 (from T/RS/1189/11/80/C)

Load Bearing Capacity = 154 kN

Inner Flange

Area	= 2550 mm^2
Temperature	= 235.5^{o}C
Strength	= 240 N/mm^2 (from T/RS/1189/11/80/C)
Load Bearing Capacity	= 612 kN

Total Load Bearing Capacity = 831 kN

The imposed test load per column was 476 kN whilst this calculation indicates a load bearing capacity of 831 kN i.e. the columns are capable of supporting the test load.

The maximum permissible load per column is 910 kN, and clearly failure would have occurred after 103 minutes if this load had been applied to the test columns.