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DEPARTMENT OF SCIENTIFIC AND INDUSTRIAL RESEARCH AND FIRE OFFICES' COMMITTEE
JOINT FIRE RESEARCH ORGANIZATION

THE FIRE RESISTANCE OF PRESTRESSED CONCRETE BEAMS

by

L. A. Ashton and H. L. Malhotra

Summary

The results are given of a limited programme of fire tests on prestressed concrete beams of the post-tensioned type, carried out by the Joint Fire Research Organization in co-operation with the Building Research Station.

The aim was primarily to obtain data for the design of beams to be used in buildings of high fire risk, such as large warehouses, where a fire resistance of 4 hours, as defined in B.S.476 : 1932, would be required for the structural elements. The limitations of the equipment precluded testing representative full-size beams, and it was therefore necessary to extrapolate from the results of tests on scaled-down specimens. It was shown that sudden or early failure was unlikely with this form of construction. Full-scale beams of the types tested having $2\frac{1}{2}$ in. concrete cover to the cable, should give a fire-resistance of 2 hours without recourse to special measures, but for 4 hours fire-resistance extra protection would be necessary for the cable.

This report has not been published and should be considered as confidential advance information. No reference should be made to it in any publication without the written consent of the Director of Fire Research.

May, 1953.

Fire Research Station,
Boreham Wood,
Herts.
(Telephone: Elstree 1341 and 1797).

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Introduction

The behaviour of a certain type of small prestressed concrete floor unit, which was shattered by spalling in the standard fire test (1), was the cause of concern and speculation regarding the possible effect of fire on prestressed concrete structures generally. Intensified research on the performance under fire of the various forms of prestressed concrete was urged to remove the doubts which were hampering its development. The work was undertaken by co-operation between the Building Research Station and the Joint Fire Research Organization of the Department of Scientific and Industrial Research.

Knowledge of the behaviour of post-tensioned beams in fires was very scanty and the most urgent need was for some information on the performance of this type of construction, leaving fundamental research until later. The only existing information on post-tensioned beams in a fire was from ad hoc tests on the Continent (2) made on full-size loaded beams. The tests, which are the subject of this report appear to be the first made in a systematic manner on this type of construction under controlled conditions.

It was decided to carry out a limited programme in the first instance, with the object of establishing the design requirements for post-tensioned beams for use in buildings where a fire-resistance of 4 hours is required. In order to complete the investigation in a reasonable time only those variables were taken into account which were considered likely to have most influence on performance. From the knowledge gained in this first investigation, the course of future work could be more readily planned.

Scope of programme

The capacity of the furnace used for fire tests on beams restricts the size of specimen which can be accommodated to about 10 ft. span. Prestressed concrete beams of the post-tensioned type are likely to be preferred for buildings where relatively large spans and heavy loads are encountered. With the equipment at the Fire Research Station no beam representative of present day practice in this form of construction could be tested, nor could reliable results be obtained by testing a beam of full-size section on a 10 ft. span. The adoption of a scaling technique offered the only means of discounting the limitations imposed by the present equipment. It was hoped that the results obtained by testing beams of different scales suitable to the size of furnace would enable a valid extrapolation to full-size to be made. Assuming 10 ft. as about the maximum span for a test beam, it seemed reasonable to adopt this size as $\frac{1}{2}$ scale and to regard the full-size beam, the fire-resistance of which was to be determined, as having a span of 20 ft. The scales adopted for the other beams in the series were 3/8 and 1/4, having spans of 7 ft. 6 in. and 5 ft. 0 in. respectively.

The factors affecting the fire-resistance of post-tensioned beams include the following which are important in different degrees:-

(1) size. (2) shape. (3) end conditions. (4) load.
(5) type of aggregate. (6) type of cement. (7) specified concrete strength. (8) concrete cover to the cable.
(9) initial stress in the cable. (10) additional insulation.

In normal design some of these factors, for example, 7 and 9 would usually vary between fairly narrow limits; for factors 5 and 6 the types in most common use should be given first consideration. The factors which would have most influence on performance were considered to be those which governed the rise in temperature of the cable and the stresses in the concrete or were likely to lead to spalling of the concrete. Experience had shown that unprotected concrete beams were likely to spall in fires, especially if made with flint gravel aggregate, but that a protective encasement to the concrete would prevent spalling as long as it remained in place.

It was decided to use gravel aggregate concrete of the same composition throughout the series and to keep constant the size of wire in the cable and the initial stress in the wire. The other factors enumerated above were treated as the variables in the investigation.

Phase 1 of the programme was designed to establish the most severe loads and end conditions for unprotected beams of the type chosen as standard by making fire tests with the various relevant combinations of the following factors:-

(1) with no imposed load (2) with imposed load = $1\frac{1}{2}$ times design live load.

(3) with ends simply supported (4) with ends restrained longitudinally.

The intention was that the combination of load and end conditions giving the earliest failure should be adopted for the remainder of the programme, which was to be devided into three phases each concerned with one of the variables under investigation. Thus phase 2, using the same type of beam as in phase 1, dealt with the effect of protective encasements; phase 3 with the effect of reducing the cover to the cable; and phase 4 with effect of shape of section. The load conditions included in phase 1 gave the greatest possible range of stresses in the concrete at the soffit from high compressive stress under no load to tensile stress under $1\frac{1}{2}$ times the design load. There was no need to test with loads between these values, since it was likely that if spalling did not occur in the unloaded condition then it would not appear with intermediate loads and, therefore, the earliest failure would be given by the greatest load. The reasons for including beams under end restraint were that this was a possible condition in practice and that the additional thermal stresses induced in the concrete might impair the fire-resistance of the beam.

For investigating the effect of protective encasements in phase 2, it was desirable to choose a material which would remain in place for the greater part of the fire test at least, and which could be relied on for consistency in properties and uniformity in thickness. Vermiculite concrete in precast slabs appeared to have the required qualities and was used in three different thicknesses. This material was used for the reasons stated and its use does not necessarily imply that other materials would not serve the purpose equally well or that it would be the most suitable in practice.

One beam was to be made for each variable included in the programme with some additional beams of certain types for repeat or other tests if the need should arise. An unprotected beam of each type and scale was made at the same time for cold loading tests.

Design of beams

A design for the full-size beam, on which the scaled test beams were to be based, was first decided on. The published descriptions of buildings erected in this country having prestressed floor beams were reviewed for the purpose of selecting a representative type both for superimposed load and span. The Stationery Office Building at Sighthill near Edinburgh was at that time the only structure designed for a live load exceeding 200 lb/sq.ft. Its secondary beams of 30 ft. span carried the floor slab and were only partially prestressed before erection. Prestressing was completed when the beams were in position, this method permitting a lower cable position at mid-span with a consequent smaller concrete cover at the soffit than if the beam had been fully prestressed initially. Castellations were provided on the top surface of the beams to ensure composite action with the floor slab.

This form of construction was adopted for the fire tests, but under the conditions in the tests, no advantage could be taken of the dead weight of the beams in positioning the cables, and therefore a greater concrete cover was obtained than would be usual in practice. Since the use of curved cables was impracticable for the smallest scale beams, it was decided that a valid compromise might be made by using straight cables and providing the beams with haunches to give the steel its appropriat concrete cover throughout the span.

The clear span envisaged for the full-scale beam was 20 ft. and the test specimens were designed by linear scaling to $\frac{1}{2}$, $\frac{3}{8}$ and $\frac{1}{4}$ full-size. No attempt was made to scale the aggregate size or the wire diameter beyond a convenient reduction applicable to all scales. Thus aggregate having a maximum size of $\frac{3}{8}$ in. and wire of No.12 S.W.G. were chosen. A live load of 1·14 tons per foot was assumed for the full-size beam, with the dead load as one half the live load. Under the condition of maximum load in the fire test the beam is required to support $1\frac{1}{2}$ times the live load and with the assumption made as to the magnitude of the dead load, the concrete of the beams would be just free from cracking. An assumed dead load of smaller proportions would give more severe stress conditions with the 50 per cent overload and the resultant cracking might be objectionable from the point of view of performance in the fire test.

The beams designed for the conditions stated above and having the cable in the portion of the span between the haunches within the middle third of the section were given the designation of Type A and are shown in Figures 1 and 2. Beams of this type were used for all the tests in phases 1 and 2 of the programme. For phase 3, in which the effect of concrete cover to the cable was investigated, a modified version of Type A was used, referred to as Type B, having one half of the concrete cover. The production of large tensile stresses in the unloaded Type B beams due to the position of the cable was prevented by including a secondary cable in the upper half of the section. For the investigation of the effect of shape of section, phase 4, an I beam was used, known as Type C, in which two cables were employed as in Type B, but in slightly different positions. All beams were provided with castellations on the top surface and the width of section was increased at the ends for the anchorages. Details of the requirements for manufacture of the beams and information on the design stresses are given in Appendix 1.

A floor slab 6 in. thick was assumed for the full-size beam, since this is the minimum recommended in reinforced concrete for a fire resistance of 4 hours (3). Mild steel reinforcement was included in the slab near the upper surface as in normal construction. The width for the slab was chosen somewhat arbitrarily, but the requirements of the test apparatus had some influence Fully dimensioned half-scale beams of the three types are shown in Figures 1 and 2 together with a schedule of the number of wires in the cables for all scales. A Type A beam of each scale is shown in Figure 3. The design loads for the beams and their test loads are given in Table 1, and the estimated stresses in the concrete under the unloaded and the fully loaded conditions are shown in Table 2. The loss of prestress assumed in the cable between stressing and testing was 20 per cent.

For ease of reference the beams were given a code symbol besides a series number, denoting type, scale, thickness of encasement, load and end conditions. Thus A / $\frac{1}{2}$ / 1 - L / R refers in the first part to details of construction (Type A, $\frac{1}{2}$-scale, with protection 1 in. thick) and in the second part to test conditions (with specified test load and ends restrained). B / $\frac{1}{4}$ / 0 - U / U is the symbol for a Type B beam of $\frac{1}{4}$-scale, without protection, tested with no imposed load and the ends unrestrained.

Table 1
Design loads and test loads for beams

Type and scale	Design load		Test load (total) (=D.L + $1\frac{1}{2}$xL.L + wt.of Beam) tons
	Dead load tons	Live load tons	
A/$\frac{1}{2}$	2·9	5·7	11·4
A/$\frac{3}{8}$	1·6	3·2	6·4
A/$\frac{1}{4}$	0·71	1·43	2·87
B/$\frac{1}{2}$	2·9	5·8	11·7
B/$\frac{1}{4}$	0·73	1·45	2·94
C/$\frac{1}{2}$	2·6	5·2	10·4
C/$\frac{3}{8}$	1·5	3·1	6·1
C/$\frac{1}{4}$	0·65	1·3	2·66

The $^3/8$-scale was omitted from the Type B series, since these beams were only intended to show the effect of cover to the cable and it was considered that this information could be obtained from the $\frac{1}{4}$ and $\frac{1}{2}$ sizes.

Table 2

Estimated concrete stresses at commencement of test

Type and scale	Estimated stresses in concrete with beam ready for test lb/sq.in.		
	At soffit of beam		At top of slab under test load
	Without load	With test load	
A / $\frac{1}{2}$	1650		
A / $\frac{3}{8}$	1650	- 600	1200
A / $\frac{1}{4}$	1700		
B / $\frac{1}{2}$	1650	- 600	1250
B / $\frac{1}{4}$	1700		
C / $\frac{1}{2}$	1650	- 550	1100
C / $\frac{3}{8}$	1750	- 600	1150
C / $\frac{1}{4}$	1700	- 550	1100

Manufacture of beams

The rectangular or I-section members, representing the precast elements of the beams, were made with circular ducts for the cables and with castellations on the upper surface. The flange of reinforced concrete representing a section of the floor slab, was cast in position after the cable was stressed. To avoid loss of tension in the wires by "draw-in" of the wedges in the anchorages a double tensioning operation was used. The method adopted consisted in making up the cable by passing the wires round a pin which formed the anchorage at one end of the beam so that a single loop became one pair of wires. Tensioning was carried out on four wires at a time, which were anchored with a single wedge in a conical hole in a plate at one end of the beam. The correct cable tension was established by attaching a jack to the anchorage pin at the other end of the beam and inserting the appropriate shims behind the pin. A grout of rapid hardening Portland cement with a water/cement ratio of 0·4 was injected in the duct when tensioning was complete.

The insulation for the protected beams was in the form of slabs of three different thicknesses (1 in., 2 in. and $2\frac{1}{2}$ in.) consisting of cement and exfoliated vermiculite in a 1 : 5 mix, with a central membrane of light wire mesh included chiefly for handling purposes. All the slabs were made of the same size and the various shapes required to encase each beam were cut as required. After cutting to size the slabs were used to line the moulds, the rather open texture of the surface providing a good key for the concrete.

Thermocouples for measuring temperatures of selected wires in the cables were fixed during manufacture in all beams except those which were to be subjected to cold loading tests. One unprotected Type A beam of each scale was made in addition to the beams in phase 1 of the programme with a large number of thermocouples in the concrete for the purpose of exploring the temperature distribution over the section. These beams were provided with their correct cables which were only lightly tensioned, as no load was to be applied in the tests.

To supplement the tests on the vermiculite-encased beams, three of the spare unprotected beams were plastered, following the brush application of a keying agent on the smooth concrete surfaces, with either gypsum/sand or cement/lime/sand.

After casting, the unprotected beams were allowed to mature under cover without special facilities for drying, until the [illegible] the beams with vermiculite protection were stored until required in a constant temperature and humidity room at 65°F and 65 per cent relative humidity.

Testing equipment

For the fire tests the beams were mounted on brackets fixed to a loading bridge constructed from 24 in. x $7\frac{1}{2}$ in. x 90 lb. British Standard Beams and having a clear space inside the vertical legs of 11 ft. as shown in Figure 4. The $\frac{1}{2}$-scale beams were supported with a clear span of 10 ft. on brackets bolted to the vertical legs of the bridge, but it was necessary to house the bracket for the beams of smaller span in refractory concrete blocks, since they projected into the furnace.

The load was applied at four points on the slab, at 1/8, 3/8, 5/8 and 7/8 of the span, each point taking one quarter of the total load. Hydraulic jacks were used for loading, mounted on the bridge as shown in Figure 5 because of restricted space between the beam and the underside of the bridge. Two jacks were necessary for $\frac{1}{2}$ and 3/8-scale beams, since the clearance between the top of the beam and the bridge was insufficient for the size of load distributors which would be required with one jack and four loading points. In testing $\frac{1}{4}$-scale beams, however, loading could be effected with one jack. The jacks were connected by flexible hose to a high pressure oil system and loads were measured by a pendulum dynamometer.

The gas-fired furnace in which the tests were made was the floor furnace, shown in Figure 6 with a beam in the loading bridge being placed on the walls. This furnace is, in effect, a rectangular box open at the top and is described in detail elsewhere (4). The specimen under test forms the upper side of the furnace so that heat is applied to the soffit, representing the conditions with a fire underneath the beam. Since a beam in position for test occupied only a narrow strip of the furnace, the opening on each side was closed by removable refractory concrete covers. The slab of the beam and the covers were on approximately the same level, with a small gap between, which was sealed with asbestos rope lagging in a manner to prevent heat loss but not to impede free deflection of the beam.

Furnace temperatures were obtained by thermocouples and measured on continuous recorders. Temperatures in the beams were measured on multi-point indicators. Deflections were measured on the top of the slab of the beams by gauges specially devised for remote operation and accurate to 0.03 in., which was sufficient for the purpose. The gauges were located at the centre of the span and over each support.

Test methods

The procedure followed in the fire tests was that prescribed for fire-resistance tests in general in B.S.476 : 1932, Definitions for Fire-resistance etc. of Building Materials and Structures. A standard rate of heating is specified, defined by a time-temperature curve, which must be developed in the furnace within certain limits. Points on the curve which determine its character are:-

At the end of	5 minutes	1000°F	(538°C)
" " " "	10 "	1300°F	(704°C)
" " " "	30 "	1550°F	(843°C)
" " " "	1 hour	1700°F	(927°C)
" " " "	2 hours	1850°F	(1010°C)
" " " "	4 "	2050°F	(1121°C)
" " " "	6 "	2200°F	(1204°C)

For elements of structure such as beams and columns B.S.476 has only one requirement of performance in the test - the element must remain rigid and not collapse while subjected to a load equal to $1\frac{1}{2}$ times the design superimposed load. The definition of "rigid" is not clear, for a certain amount of deflection can be tolerated. It has been interpreted for floors in houses to mean that a deflection exceeding 1/20 of the span constitutes failure (3). In the present tests failure was considered to have occurred when it was impossible to maintain the test load. The time elapsing from the start of the test until failure occurs is defined as the fire-resistance of a structural element.

The age of the beams when tested varied between about six months to over a year. The moisture content of the unprotected beams was about 5 per cent. Cubes of both beam and slab concrete for crushing on the day of test were stored with the beams. After a beam had been mounted on its supports in the bridge, the bearing plates at each load point were bedded on the surface of the slab with aluminous cement. In the protected beams, where a high fire-resistance was expected, a pad of asbestos millboard was interposed for heat insulation between the bearing plate and the bed.

Test results

(a) Cold load tests. The results of the strength tests made at the Building Research Station on beams of each type and scale are summarised in Table 3.

Table 3

Load tests on beams

Type and scale	Total load - tons		
	For application in fire test	At first cracks in load tests	At failure in load tests
A / $\frac{1}{2}$	11·4	12·3	21·5
A / $3/8$	6·4	6·9	12·1
A / $\frac{1}{4}$	2·87	2·75	5·43
B / $\frac{1}{2}$	11·7	11·2	21·6
B / $\frac{1}{4}$	2·94	3·31	5·56
C / $\frac{1}{2}$	10·4	11·5	19·6
C / $3/8$	6·1	6·8	12·2
C / $\frac{1}{4}$	2·66	2·52	4·91

(b) Fire tests. The results of all the tests are summarised in Tables 4, 5 and 6; certain of the results are shown graphically in Figures 7 to 13.

(1) Phase 1. In this series of the tests the fire-resistance of Type A unprotected beams was determined under conditions of load and restraint which would produce the greatest extremes of concrete stress possible with a maximum value for the applied load of dead load + $1\frac{1}{2}$ times the live load and a minimum value of the weight of beam only. From the results of these tests the conditions for the remainder of the programme were decided.

It was realised that the unloaded condition was not a practical one, but it was included because it was considered likely to produce spalling of the concrete. If spalling did not occur it could be assumed that it was unlikely to appear with intermediate loads. The distinctive feature of the tests without imposed load was the failure of the $\frac{1}{2}$-scale beams by upward deflection; the times at which failure occurred, both with restrained and freely supported ends, was less than the time to collapse when fully loaded. With the $\frac{1}{4}$ and $3/8$-scale beams, however, although there was upward deflection the force in the cable was reduced by increase in temperature so that it did not produce a tension failure in the slab and the tests were stopped when the concrete cover fell from the cable. That the application of the dead load assumed in design was sufficient to prevent the upward failure of $\frac{1}{2}$-scale beams was shown in the test on beam No.667, which deflected upwards initially but had returned to its original position after a time greater than that at which failure of the unloaded beam occurred. Again with load and longitudinal restraint a difference in behaviour was obtained between the $\frac{1}{2}$-scale beam and the two smaller scale beams, for whereas the fire-resistance of the first was 16 per cent lower than when simply supported, the $\frac{1}{4}$ and $\frac{3}{8}$-scale specimens gave increases of 58 and 26 per cent respectively. Consideration of the plotted results for the loaded/...

TABLE 4

SUMMARY OF RESULTS FOR UNPROTECTED BEAMS

TYPE 'A'

No.	SYMBOL FOR BEAM & TEST CONDITION	AGE AT TEST	CUBE STRENGTH Lbs / in^2. (1)		LOAD	TESTING PERIOD	TYPE OF FAILURE	CENTRE DEFLECTION (2)		MEAN CABLE TEMPERATURE		REMARKS
		Months	BEAM	SLAB	Tons	Hr - Min		Hr-Min	Ins.	Hr-Min	°C	
675	A/½/0-U.U.	6½	5615 (28d) 9845 (6½m)	– 8200 (5m)	NIL	1 - 10	Collapse by upward deflection.	0-30 1-00 1-10	-0.24 -.96 -1.33	0 - 30 1 - 00 1 - 10	109 176 225	
671	A/½/0-U.R	6½	5670 (28d) 8045 (6½m)	8022 (28d) 10100 (4m)	NIL	1 - 02	Collapse by upward deflection.	0-30 1-00	- 0.20 - 0.38	0 - 30 1 - 00	109 171	
681	A/½/0-L.U	9	6472 (5d) 10300 (9m)	7420 (29d) 9400 (8m)	10.9	1 - 38	Fracture of concrete and yield of cable at centre.	0-10 0-30 1-00 1-30	0.57 0.93 1.12 3.00	0 - 30 1 - 00 1 - 35	106 200 475	
678	A/½/0-L.U	12½	6430 (5d) –	6830 (28d) –	10.9	1 - 46	Shear failure of concrete near one end.	0-10 1-00 1-45	0.66 0.94 3.16	0 - 30 1 - 00 1 - 45	104 147 478	Repeat test.
664	A/½/0-L.R	10½	8086 (28d) 9850 (10½m)	8208 (28d) 10600 (7½m)	10.9	1 - 29	Fracture of concrete and yield of cable.	0-10 1-00 1-25	0.27 0.32 0.66	0 - 30 1 - 00 1 - 25	110 177 341	
667	A/½/0-L.U	11½	3660 (4d) 9700 (11½m)	6515 (29d) 9150 (9½m)	2.4	2 - 00	NO FAILURE	0-10 1-00 2-00	0.07 -0.32 0.38	0 - 30 1 - 00 1 - 30 2 - 00	101 155 356 544	Test stopped at 2 hours, there was no sign of failure.
665	A/⅜/0-U.U	7	6010 (6d) 10110 (7m)	6870 (28d) 8640 (5½m)	NIL	1 - 25	Fall of concrete cover to the cable.	0-10 1-00 1-20	0.05 -0.81 -0.79	0 - 30 1 - 00 1 - 20	133 420 572	
676	A/⅜/0-U.R	6½	5615 (7d) 9820 (6½m)	6048 (28d) 8020 (5½m)	NIL	1 - 48	Fall of concrete cover tb the cable.	0-10 1-00 1-45	-0.02 -0.13 -1.00	0 - 30 1 - 00 1 - 45	147 361 785	
682	A/⅜/0-L.U	10½	5610 (5d) 10000 (10½m)	7390 (28d) 9610 (8½m)	6.2	1 - 06	Fracture of concrete and cable near centre.	0-10 0-30 1-05	0.29 0.44 3.08	0 - 30 1 - 05	158 483	
695	A/⅜/0-L.U	12	6780 (5d) 10000 (12m)	8210 (28d) 10900 (11m)	6.2	1 - 08	Fracture of concrete and cable near centre.	0-10 0-30 1-05	0.36 0.63 1.83	0 - 30 1 - 05	133 435	
672	A/⅜/0-L.R	11½	5550 (5d) 8800 (11½m)	7040 (28d) 7810 (10m)	6.2	1 - 24	Fracture of concrete and yield of cable.	0-10 1-00 1-20	0.13 0.10 0.25	0 - 30 1 - 00 1 - 20	131 355 555	
677	A/⅜/0-L.U	10½			6.2	0 - 45	NO FAILURE	0-10 0-45	0.34 0.95	0 - 10 0 - 45	95 281	Beam loaded to failure when cold. Ultimate load 9.9 tons.

Notes:- (1) Figures in brackets after the cube strengths denote the age of the cubes at test. 'd' stands for days and 'm' for months.

(2) - ve sign before deflection readings indicates upward movement.

TABLE 4 (Contd.)

No.	SYMBOL FOR BEAM & TEST CONDITIONS	AGE AT TEST	CUBE STRENGTH (1) Lb / In^2.		LOAD	TESTING PERIOD	TYPE OF FAILURE	CENTRE DEFLECTION (2)		MEAN CABLE TEMPERATURE		REMARKS
		Months	BEAM	SLAB	Tons	Hr - Min		Hr - Min	Ins.	Hr - Min	°C	
683	A/$\frac{1}{4}$/0-U.U	$6\frac{1}{2}$	6200 (5d) 10000 ($6\frac{1}{2}$m)	5530 (28d) 8020 ($5\frac{1}{2}$m)	NIL	1 - 05	Fall of concrete cover to the cable.	0 - 10 0 - 30 1 - 05	-0.07 -0.53 -1.01	0 - 30 1 - 05	342 803	
684	A/$\frac{1}{4}$/0-U.R	$6\frac{1}{2}$	5265 (5d) 10200 ($6\frac{1}{2}$m)	5530 (28d) 9500 ($5\frac{1}{2}$m)	NIL	1 - 18	Fall of concrete cover to the cable.	0 - 10 0 - 30 1 - 15	-0.08 -0.15 -1.03	0 - 30 1 - 00 1 - 15	317 715 820	
686	A/$\frac{1}{4}$/0-L.U	$9\frac{1}{2}$	4935 (5d) 9620 ($9\frac{1}{2}$m)	7210 (35d) 9700 ($7\frac{1}{2}$m)	2.8	0 - 40	Fracture of concrete and cable.	0 - 10 0 - 38	0.26 1.60	0 - 20 0 - 35	189 421	
693	A/$\frac{1}{4}$/0-L.U	$11\frac{1}{2}$	6630 (5d) 16500 ($11\frac{1}{2}$m)	7210 (35d) 9100 ($10\frac{1}{2}$m)	2.8	0 - 36	Fracture of concrete and yield of cable.	0 - 10 0 - 30	0.35 1.04	0 - 20 0 - 35	200 481	Repeat test to check failure time.
690	A/$\frac{1}{4}$/0-L.R	$8\frac{1}{2}$	7020 (5d) 10600 ($8\frac{1}{2}$m)	7210 (35d) 9700 ($7\frac{1}{2}$m)	2.8	0 - 57	Fracture of concrete and yield of cable.	0 - 10 0 - 55	0.10 -0.23	0 - 20 0 - 55	147 658	

Notes:- (1) Figures in brackets after the cube strengths denote the age of the cubes at test. 'd' stands for days and 'm' for months.

(2) - ve sign before deflection readings indicates upward movement.

No.	SYMBOL FOR BEAM	AGE AT TEST	CUBE STRENGTH (1) lb / in²		LOAD	TESTING PERIOD	TYPE OF FAILURE	CENTRE DEFLECTION (2)		MEAN CABLE TEMPERATURE		REMARKS
		Months	BEAM	SLAB	Tons	hr - min		hr-min	Ins.	hr-min	°C	
687	A/$\frac{1}{2}$/1	10½	6750 (6d) -	9675 (33d) 11000 (9½m)	10.9	3 - 51	Fracture of concrete and yield of cable	1-00 3-00 3-45	0.25 1.34 4.09	1-00 3-00 3-45	105 275 382	Spalling at soffit near the ends at 1½ hours.
691	A/$\frac{1}{2}$/2	10½	6777 (6d) 10850 (10½m)	8085 (28d) 10850 (9½m)	10.9	6 - 00	NO FAILURE	2-00 4-00 6-00	0.10 0.52 2.36	2-00 4-00 6-00	97 165 384	Test stopped at 6 hrs and a water test applied for 6 mins. Permanent deflection after 16 hrs was 2.05 in.
694	A/$\frac{1}{2}$/2½	10½	6635 (5d) 10800 (10½m)	8085 (28d) 10350 (9½m)	10.9	6 - 00	NO FAILURE	2-00 4-00 6-00	-0.18 -0.11 0.30	2-00 4-00 6-00	79 110 297	Cold loading tests carried out - See Fig.17.
692	A/$\frac{3}{8}$/1	10½	6750 (5d) 9620 (10½m)	7852 (28d) -	6.2	2 - 38	Fracture of concrete and cable at the place of spalling	1-00 2-00 2-37	0.18 0.45 2.15	1-00 2-00 2-30	138 261 347	Spalling at soffit near one end at 1¼ hours.
689	A/$\frac{3}{8}$/1	11	6795 (5d) -	9290 (21d) -	6.2	3 - 37	Fracture of concrete and cable near centre	1-00 2-00 3-30	0.17 0.46 2.77	1-00 2-00 3-30	114 229 494	Repeat test. No spalling in this test.
696	A/$\frac{3}{8}$/2	13½	7360 (5d) 10050 (13½m)	9290 (21d) 11950 (13m)	6.2	5 - 41	Fracture of concrete and cable near centre	2-00 4-00 5-40	-0.03 0.27 3.34	2-00 4-00 5-38	103 283 473	
697	A/$\frac{1}{4}$/1	10	7360 (5d) 10000 (10m)	7210 (35d) 8700 (9½m)	2.8	2 - 34	Fracture of concrete and yield of cable	1-00 2-00 2-29	0.71 1.82 4.16	1-00 2-00 2-30	125 238 293	
685	A/$\frac{1}{4}$/$\frac{1}{2}$	17	5775 (5d)	5530 (28)	2.8	1 - 07	Fracture of concrete and yield of cable	0-30 1-00	1.59 2.15	0-30 1-00	134 391	Beam protected with ½ in thick cement/lime/sand plaster.
702	A/$\frac{1}{4}$/2	10	6692 (5d) 9550 (10m)	8140 (28d) -	2.8	4 - 24	Fracture of concrete and yield of cable	1-00 3-00 4-10	0.17 0.86 4.73	1-00 3-00 4-20	84 257 433	

Notes:- (1.) Figures in brackets after the cube strength denote the age of the cubes at test. 'd' stands for days and 'm' stands for months.

(2) - ve sign before deflection readings indicates upward movement.

TABLE 5

SUMMARY OF RESULTS FOR PROTECTED BEAMS

TYPE 'A'

TABLE 6

SUMMARY OF RESULTS FOR BEAMS TYPE 'B' & TYPE 'C'

BEAM No.	SYMBOL FOR BEAM	AGE AT TEST Months	CUBE STRENGTH (1) Lb / in^2. BEAM	SLAB	LOAD Tons	TESTING PERIOD Hr - Min	TYPE OF FAILURE	CENTRE DEFLECTION (2) Ins Hr-min		MEAN CABLE TEMPERATURE °C Hr-min	Bottom	Top	REMARKS
706	B/½/0	13	6432 (5d) 11450 (13m)	7480 (28d) 10500 (12½m)	11.2	1 - 20	Shear failure of concrete near one end.	0 - 10 1 - 00 1 - 20	0.47 1.06 2.83	0 - 30 1 - 00 1 - 15	150 293 388	105 162 256	
709	B/½/1	13	2239 (5d) 10650 (13m)	7100 (28d) 9900 (12½m)	11.2	6 - 00	NO FAILURE	1 - 00 4 - 00 6 - 00	0.15 1.33 1.63	2 - 00 4 - 00 6 - 00	172 478 829	139 266 456	Beam loaded to destruction immediately after fire test, failed at a load of 14.05 tons, final deflection 2.42 in.
716	B/¼/0	14	5470 (5d) 10500 (14m)	6660 (28d) 9400 (13m)	2.87	0 - 35	Fracture of cable and concrete	0 - 10 0 - 30	0.64 1.13	0 - 15 0 - 35	309 612	65 302	
724	B/¼/1	14½	5090 (6d) 10300 (14½m)	6790 (28d) -	2.87	3 - 25	NO FAILURE	0 - 30 2 - 00 3 - 10	- 0.17 0.15 0.41	1 - 00 2 - 00 3 - 20	187 470 760	125 282 530	Test stopped.
726	B/¼/2	14½	5605 (5d) 9950 (14½m)	6790 (28d) 10050 (14m)	2.87	5 - 40	NO FAILURE	1 - 00 3 - 00 5 - 30	- 0.22 - 0.36 0.00	1 - 00 3 - 00 5 - 34	88 285 720	77 293 505	Immediately after test the beam withstood a load of 4 tons, permanent deflection after release of load 1-in.
719	C/½/0	14	5300 (6d) 9600 (14m)	6260 (28d) 8400 (13½m)	9.9	1 - 16	Shear failure of concrete near one end.	0 - 10 0 - 30 1 - 15	0.42 0.66 2.85	0 - 30 1 - 15	125 615	159 525	
721	C/⅜/0	14	6710 (5d) 10650 (14m)	6900 (28d) 8710 (13½m)	5.9	0 - 50	Fracture of concrete and yield of cable.	0 - 10 0 - 47	0.35 2.56	0 - 15 0 - 45	132 494	152 527	
720	C/⅜/0	13½	6160 (6d) 6850 (13½m)	6750 (28d) 9650 (13m)	5.9	0 - 30	Test stopped for residual deflection readings.	0 - 10 0 - 30	0.37 0.55	0 - 15 0 - 30	114 204	146 341	Time after test. Load. Deflection 0-30 on 1.38 1-00 " 1.51 5-00 " 1.31 5-00 off 0.60
730	C/½/0	15	4100 (5d) 8150 (15m)	6880 (28d) -	2.6	0 - 24	Fracture of cable and concrete at centre	0 - 10 0 - 20	0.23 0.58	0 - 10 0 - 20	185 418	144 272	
708	B/½/½	15	6545 (5d)	6950 (28d)	11.2	2 - 41	Shear failure of concrete near one end.	1 - 00 2 - 00 2 - 30	0.78 2.55 4.25	1 - 00 2 - 00 2 - 30	210 515 613	112 330 429	Beam protection ½-in cement/lime/sand plaster.
714	B/¼/½	14	6080 (5d)	6660 (28d)	2.87	1 - 08	Fracture of cable and concrete at centre.	0 - 30 1 - 00	0.00 0.68	0 - 30 1 - 00	160 558	98 218	Beam protection ½-in gypsum plaster.

Notes:- (1) Figures in bracket after the cube strengths denote the age of the cubes at test. 'd' stands for days and 'm' for months.

(2) - ve sign before deflection readings indicates upward buckling.

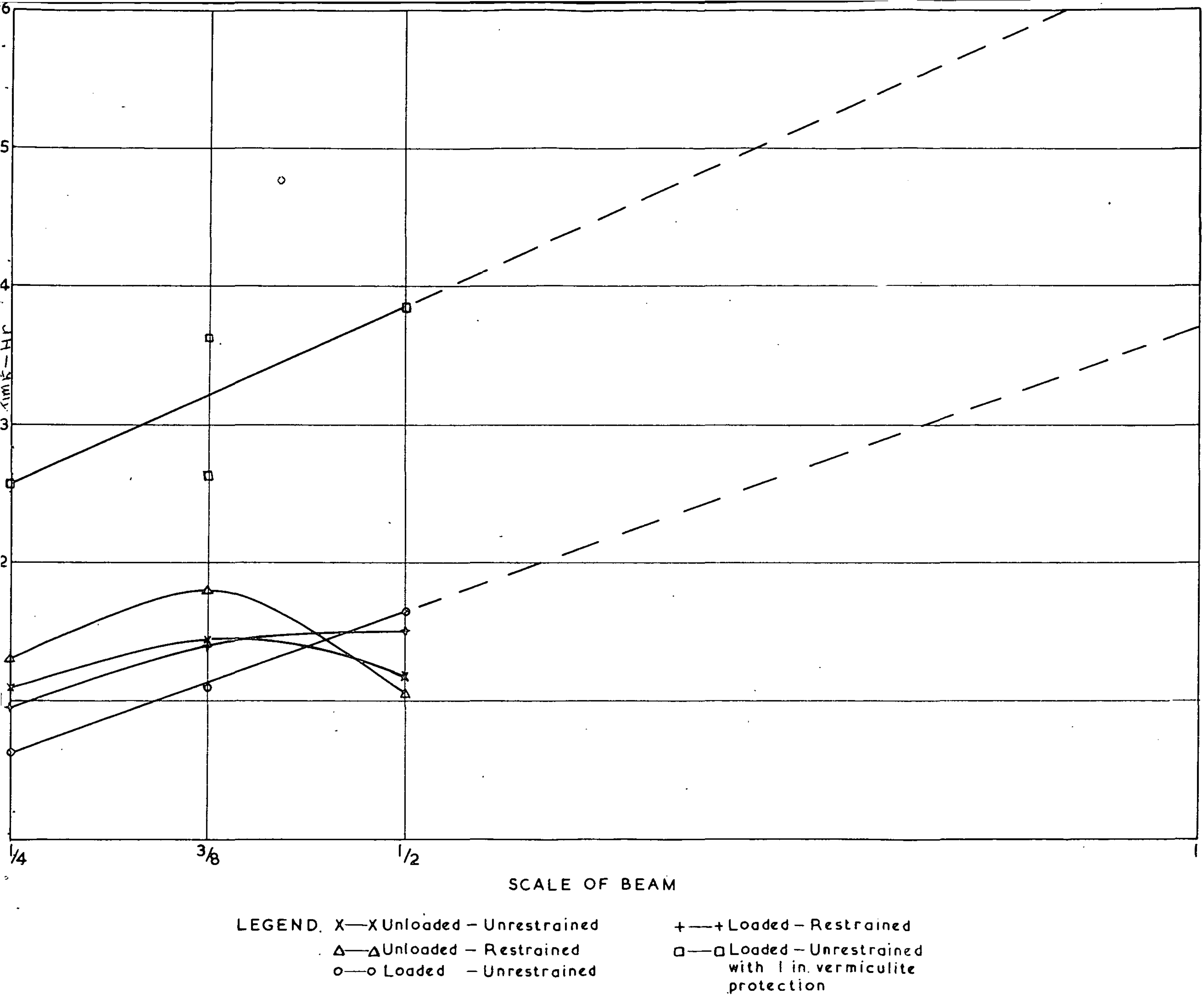


FIG. 7. FIRE RESISTANCE OF TYPE 'A' BEAMS

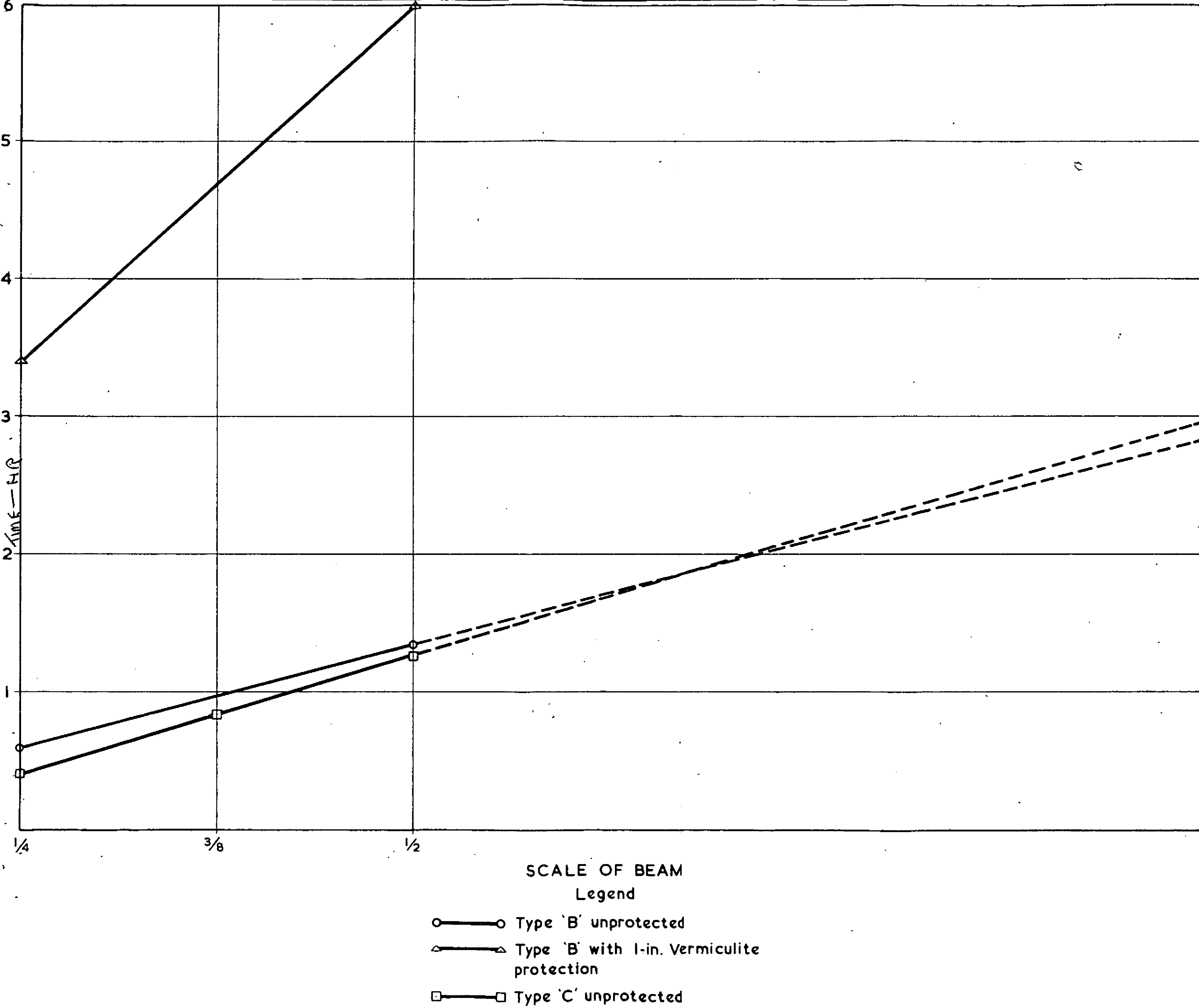
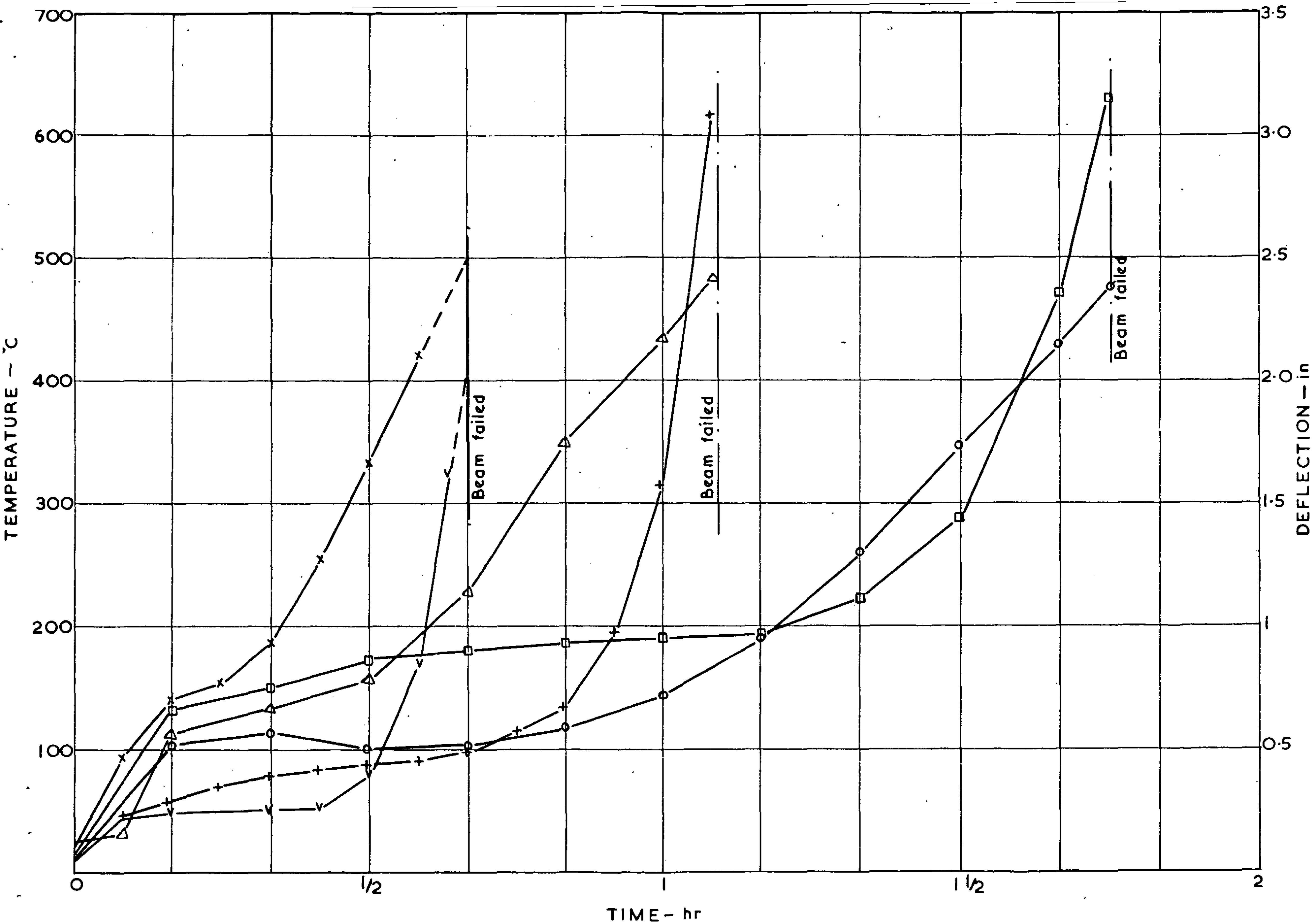


FIG. 8. FIRE RESISTANCE OF TYPE 'B' AND 'C' BEAMS



LEGEND O—O Mean cable temperature - 1/2 scale □—□ Beam centre deflection 1/2 scale
△—△ " " " " 3/8 " +—+ " " " 3/8 "
X—X " " " " 1/4 " V—V " " " 1/4 "

Fig. 9

CABLE TEMPERATURE AND BEAM DEFLECTION CURVES FOR 'A' TYPE UNPROTECTED BEAMS

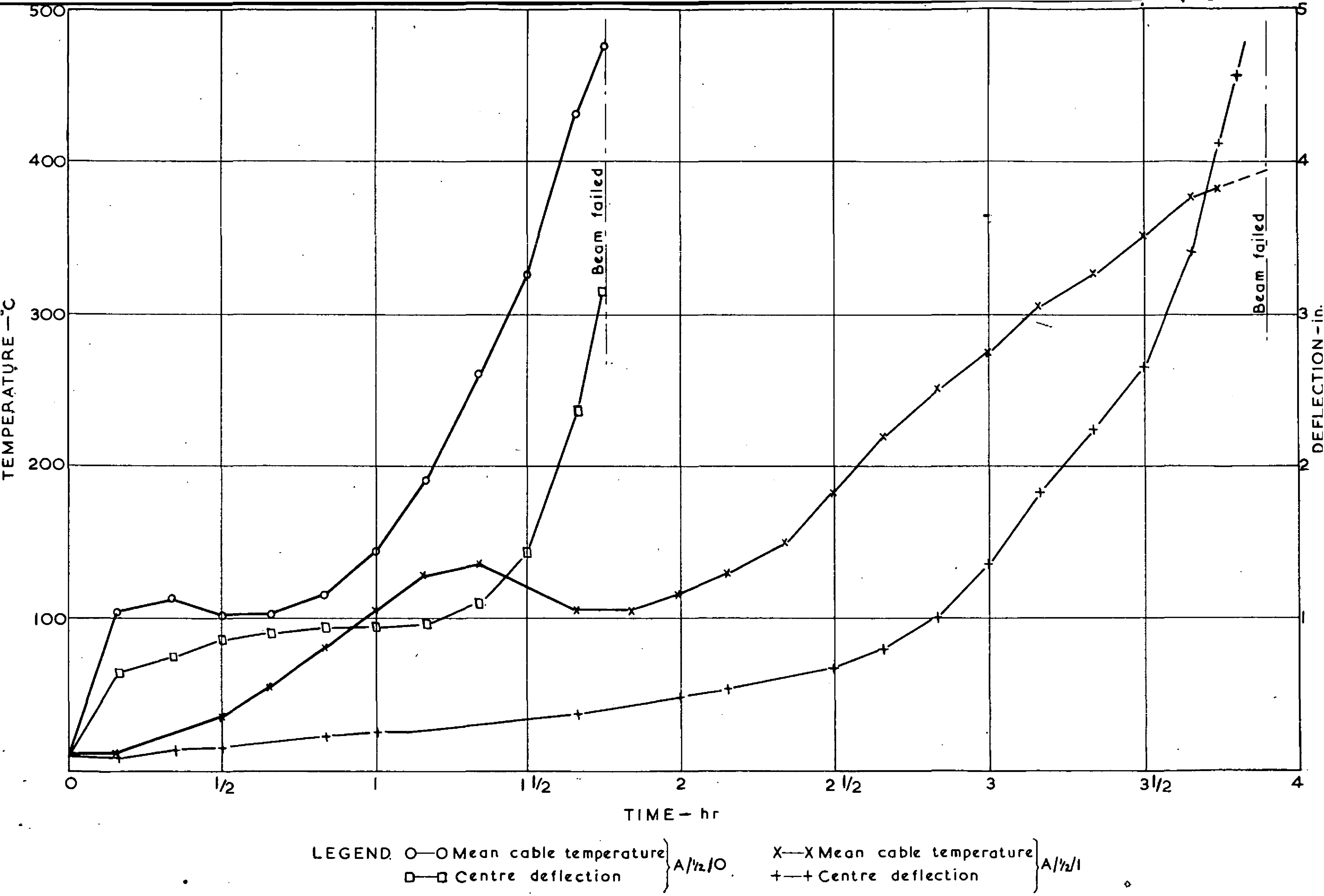


FIG. 10. CABLE TEMPERATURE AND BEAM DEFLECTION CURVES FOR 'A' TYPE 1/2 SCALE BEAMS

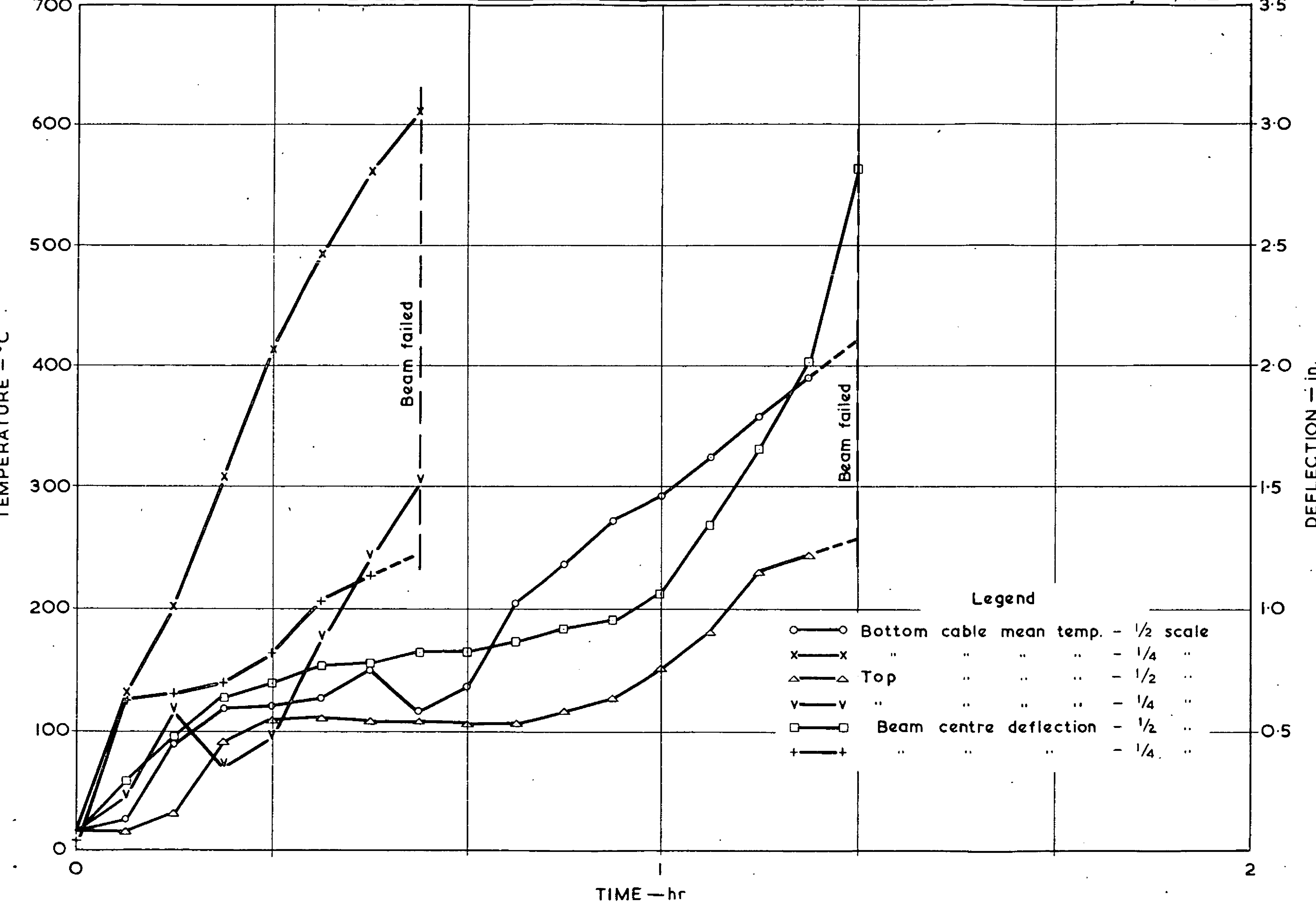
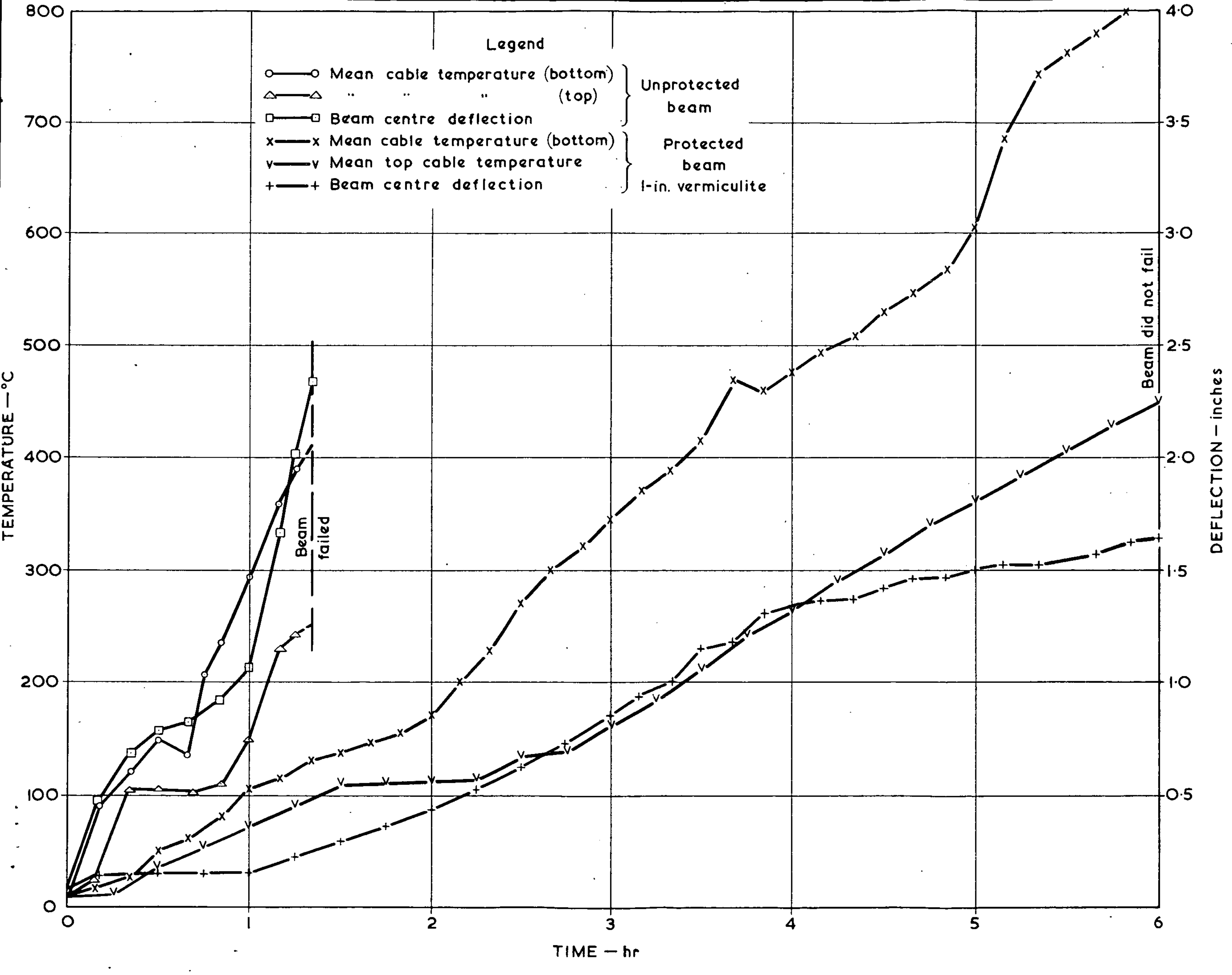


FIG. 11. CABLE TEMPERATURE AND BEAM DEFLECTION CURVES FOR TYPE 'B' BEAMS



CABLE TEMPERATURE AND BEAM DEFLECTION CURVES FOR TWO ½ SCALE 'B' TYPE BEAMS.

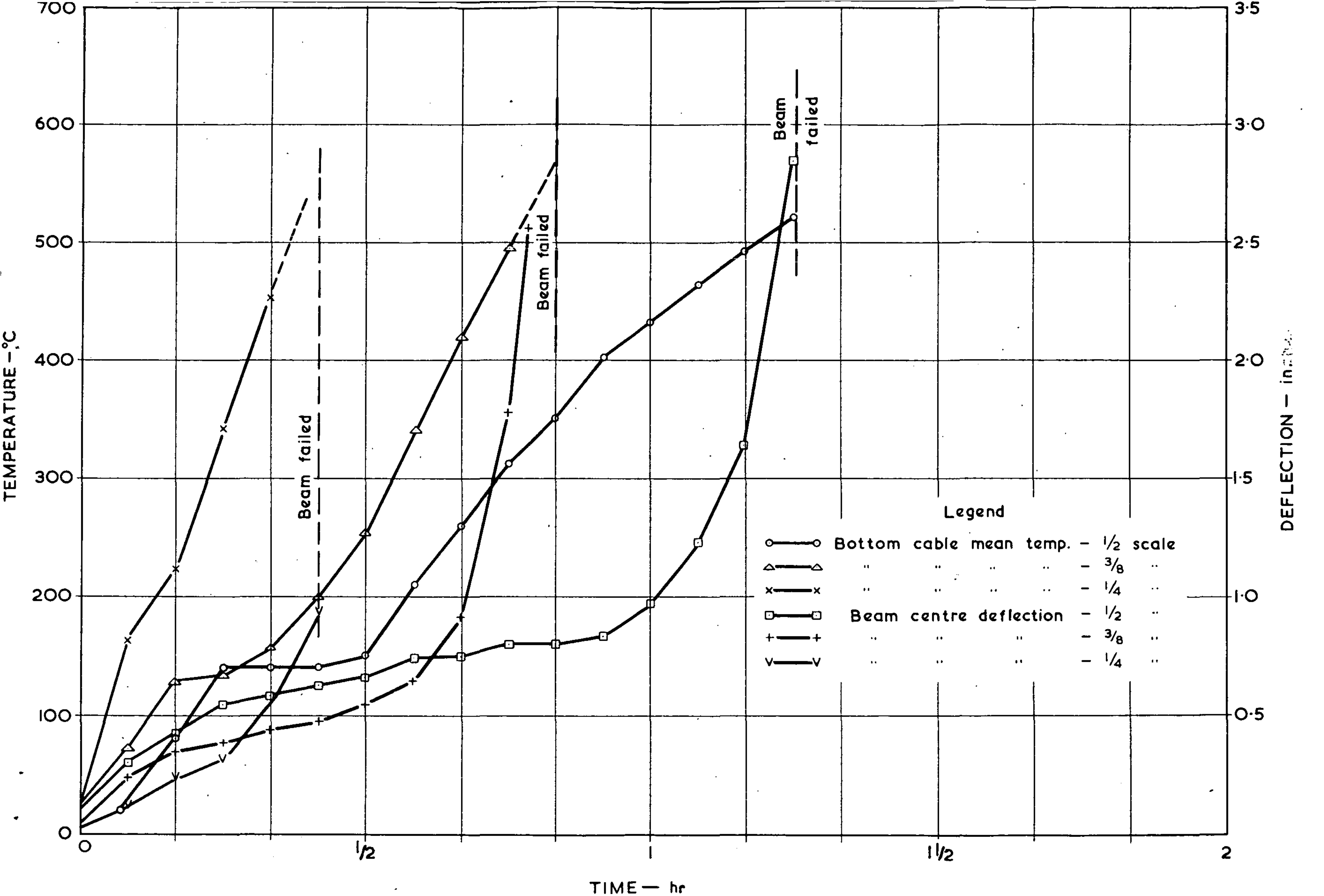


FIG. 13. CABLE TEMPERATURE AND BEAM DEFLECTION CURVES FOR TYPE 'C' BEAMS.

restrained and loaded/simply supported conditions showed that more confidence could be placed in extrapolating from the second than the first. This fact, together with the less likely occurrence of the fully restrained condition in actual buildings, lead to the decision to adopt freely supported ends for the beams in the later phases of the programme. Repeat tests made under the selected conditions on beams of all scales (Nos. 678, 695 and 693) gave results which did not differ from those first obtained by more than 10 per cent.

(2) Phase 2. It was apparent that the fire-resistance of the beams had been underestimated originally when planning the programme for encased beams, and the tests in this phase showed also that the protection behaved better than anticipated. A protection of vermiculite concrete 1 in. thick gave an addition of about 2 hours to the fire resistance in each scale and only these results have been plotted in Figure 7. The behaviour of the 3/8-scale beam (No.692) first tested was remarkable for the spalling which occurred close to one haunch after about $1\frac{1}{4}$ hours of test. The spalling, which was of sufficient violence to remove the cover to the cable and the protection for a distance of about 12 in. probably caused an early failure, since a repeat test, in which spalling did not occur, was of about 1 hour greater duration. Spalling of concrete which is covered by even thin protective coatings such as plaster is unusual and has not occurred previously in a floor test. Since no failure had occurred after 6 hours heating of the A / $\frac{1}{2}$ / 2 and A / $\frac{1}{2}$ / $2\frac{1}{2}$ beams application of water from a fire hose was made followed by loading tests. Beams in the $\frac{1}{4}$ and $\frac{3}{8}$-scales with the $2\frac{1}{2}$ in. thick encasement were not tested.

(3) Phase 3. The Type B beams dealt with in this part of the programme had half the concrete cover to the cable of the corresponding beams of Type A and were characterised by the split cable. Both unprotected and protected beams were included but the $\frac{3}{8}$-scale specimens were omitted from this phase. The fire-resistance of the unprotected beams was a little less than that of Type A beams for $\frac{1}{4}$-scale and about 20 per cent less for the $\frac{1}{2}$-scale. Protected Type B beams, however, were superior to protected Type A beams with the same encasement. Thus beam No.724 (B / $\frac{1}{4}$ / 1) was still supporting its load with a relatively small deflection after 3 hours 25 minutes of test although the cable temperature was higher than that at which failure was considered inevitable. The test was stopped at this point to examine the beam and loading equipment in case the beam was partially supported by the furnace covers or the load shown by the dynamometer was not in fact being applied. Everything appeared to be in order, and confirmation of the high resistance of beams so protected was obtained on the B / $\frac{1}{4}$ / 2 and B / $\frac{1}{2}$ / 1 specimens. In the first of these (beam No.726) the test was stopped after 5 hours 40 minutes without failure. The beam after removal from the furnace and still hot supported without failure a load of 4 tons (1•06 tons more than the test load) with a permanent deflection after removal of the load of 1 in. The B / $\frac{1}{2}$ / 1 specimen (No.709) was heated for 6 hours and immediately after removal from the furnace was subjected to a 6 minute water test then loaded to failure at 14•05 tons, an increase of 2•85 tons over the test load. It was considered that no useful purpose would be served by carrying out tests on the Type B beams with thicker encasements and the remainder of the specimens were reserved for the investigation of certain other factors not specifically included in the programme.

(4) Phase 4. The beams in this part of the programme, designated Type C, were all unprotected, having the split cable and differing from Type B beams principally in the shape of cross-section. Failures were slightly earlier in each scale than for the corresponding beams of Type B, the difference for the $\frac{1}{2}$-size being negligible.

(5) General behaviour of beams in fire tests. Observations were made of the behaviour of all the beams during the fire test, and there was, in general, a marked similarity in the records obtained. It appears that, when the first cracks became visible, the unprotected beams had reached about one half of the heating time which would produce collapse. The damage usually started as short longitudinal cracks on the sides of the beam with sometimes longitudinal cracks on the soffit. Later, cracks formed on the sides of the beam approximately along the line of the cable. Failure in the majority of the specimens was due to failure of the cable. Although actual fracture of the wires occurred only in the smaller beams the typical signs of large strains in the steel were present. Collapse of the beam was preceded by the formation of large cracks in the tension zone accompanied by a marked increase in the deflection. The rapid decrease in the compression area of the concrete caused crushing of the concrete and failure

of the beam to support the imposed load. In one or two instances the formation of inclined cracks showed the presence of excessive diagonal stresses. (See Figures 14, 15 and 16).

The behaviour of the vermiculite encasements of the protected beams was uniformly good, little damage being visible until failure was approaching, when cracks appeared near the centre in the tension zone. Most of the encasement survived the water test in specimens where it was applied.

(6) Other tests. As an alternative beam protection to the encasement of vermiculite slabs, three of the spare unprotected beams were used to obtain results with normal in situ plaster finishes. Beams Nos. 685, 708 and 714 were plastered $\frac{1}{2}$ in. thick, the first two with cement/lime and the last with gypsum plaster, after the application of a brushed-on keying agent. In the tests on all three beams the greater part of the plaster remained in place until the end, giving an increase in fire-resistance of 32 minutes for both the A / $\frac{1}{4}$ and B / $\frac{1}{4}$ beams and 1 hour 20 minutes for the B / $\frac{1}{2}$ beam.

Loading tests were made after cooling on beams which did not fail in the fire tests. An A / $\frac{3}{8}$ / 0 beam, No.677, when loaded cold after a furnace test lasting 45 minutes, failed at 9·9 tons, that is 82 per cent of the original strength. The characteristics of an A / $\frac{1}{2}$ / $2\frac{1}{2}$ beam, No.694, which had been subjected to a 6 hour fire test and a six minutes water test, giving a final mean cable temperature of about 300°C are shown by the load/deflection curve of Figure 17 in comparison with the curve for a beam before heating. The ultimate strength of the heated beam was 16·8 tons equivalent to 78 per cent of the original. The fire test on a C / $\frac{3}{8}$ / 0 beam No.720, was stopped at 30 minutes, a little over half of the time to failure and observations made of the recovery during cooling and the residual deflection as shown in Table 6.

Discussion of results

In considering the results of the fire tests on the types of beam included in the programme it should be borne in mind that certain features, which may not appear representative of prestressed concrete design, were deliberately introduced because they were regarded as likely to be critical under fire conditions. Thus the percentage of steel in the combined section of Type A beams was low, although it gave a factor of over 2·0 for the ratio ultimate load/dead load + design superimposed load. Since the moment of inertia of the precast element was about one quarter of that of the combined section any deficiency in composite action between the element and the floor slab would be more pronounced.

(1) Effect of size of beam. Considering only the beams tested under conditions of full load and simple support for the ends, it is shown in Figures 7 and 8 that there is straight line relationship, over the range tested, between beam size and fire-resistance when all the linear dimensions are scaled. For the Type A and Type C beams, extrapolation on the graphs gives fire-resistances for the full size beams of 3 hours 40 minutes and 2 hours 50 minutes respectively. The only other conditions of practical importance where extrapolation is of interest, full load and ends restrained, are shown by the graph, Figure 7, to indicate a fire-resistance for full size Type A beams substantially lower than that for full load and simple support, assuming that the same degree of longitudinal restraint is obtained in the full scale as in the smaller beams.

(2) Effect of end conditions. Two conditions at the ends of the beams were investigated: simple support with a 3 in. bearing of concrete-steel, and a high degree of longitudinal restraint. As indicated in (1) above, the effect of restraint was to increase the fire-resistance of the two smaller scale beams compared with the simply supported condition, and to decrease it for the $\frac{1}{2}$ scale beam. For the $\frac{1}{4}$ scale beam No.690 the difference, taking the results for simple support as the basis of comparison was + 58 per cent, for the $\frac{3}{8}$ scale No.672 + 26 per cent, and for the $\frac{1}{2}$ scale No.664 - 16 per cent (see Figure 7). It will be seen from the results in Table 4 that restraint reduced the amount of early deflection, which is attributable chiefly to the large difference in temperature between the lower and upper surfaces of the beam, but the additional concrete stresses did not cause the rapid disintegration which had been considered possible. In the smaller size beams the mean temperature of the cable at failure was higher for restraint than for simple support, while in the $\frac{1}{2}$ scale beam it was lower, showing that for the specimens of the $\frac{1}{4}$ and $\frac{3}{8}$ scales the maximum bending moment was reduced by restraint.

(3) Effect of load.

As previously explained the condition of no imposed load was included because it was considered that the resulting stress conditions might lead to spalling of the concrete in the test. When it was found that spalling did not occur in any of the unloaded beams, simply supported or restrained, this condition was ignored as being unrepresentative of practical conditions for the types of beam in the programme. Since a test on an A / $\frac{1}{2}$ / 0 beam No.667, with an imposed load equal to the dead load assumed in design, gave a greater fire-resistance than the test when the applied load was $1\frac{1}{2}$ times the design load, it was justifiable to consider that fire-resistance decreases with increase of load. The behaviour of the beams in the tests without imposed load is interesting. In the $\frac{1}{4}$ and $\frac{3}{8}$-scale beams, Nos. 683, 684, 665 and 676, while upward deflection occurred it was insufficient to cause failure and the tests were stopped when the cover to the cable fell. After exposure to the furnace temperatures for 1 hour the $\frac{1}{2}$-scale beams Nos. 671 and 675 were in the condition where the concrete for at least 1 in. from the exposed surfaces had suffered a considerable reduction in strength while the steel in the cable was still little affected. The effective concrete area was thus diminished and the eccentricity of the cable virtually increased, leading to the type of failure shown in Figure 18 in which the force in the cable was sufficient to cause tensile stresses in the slab and crushing of the concrete in the lower part of the beam. Application of a load equivalent to the assumed dead load, while it did not suppress upward deflection of the beam entirely, prevented this mode of failure.

(4) Effect of concrete cover to the cable.

The importance of the thickness of concrete around the cable lies in the protection it provides against dangerous rise in temperature of the cold-drawn steel wires. Temperature measurements made during the tests on the wires showed that, in general, failure was likely when the mean cable temperature exceeded 400°C. The Type B beams, which were specifically designed to show the influence of thickness of concrete cover to the cable, having half of the cover in each scale of the corresponding Type A beams, behaved much better than was expected. For example the two A / $\frac{1}{4}$ / 0 beams having 1 in. cover to the cable gave a mean time to failure of 38 minutes, whereas the B / $\frac{1}{4}$ / 0 beam with $\frac{1}{2}$ in. cover gave 35 minutes; the two A / $\frac{1}{2}$ / 0 beams having 2 in. cover gave a mean time to failure of 102 minutes and the B / $\frac{1}{2}$ / 0 beam with 1 in. cover gave 80 minutes. If it is desired to estimate the cover required for some specified fire-resistance the results of the tests on Type B beams are of little help. The Type A beams are more useful here, for in these tests on specimens of each scale failure occurred at about the same mean cable temperature (480°C approximately). Thus $\frac{1}{4}$ scale with 1 in. cover gave 36 minutes, $\frac{3}{8}$ scale with $1\frac{1}{2}$ in. cover gave 66 minutes and $\frac{1}{2}$ scale with 2 in. cover gave 98 minutes. This indicates a fire-resistance of a few minutes over 2 hours for $2\frac{1}{2}$ cover and about 4 hours for $4\frac{1}{2}$ in. cover. There is a possibility that the thicker concrete covers would tend to spall away and to guard against this it would be desirable to include light mesh reinforcement in the concrete at the sides and below the cable when the thickness of cover exceeds 3 in.

(5) Effect of insulation.

The insulation used in the tests as beam encasement was very effective, 1 in. thickness providing an increase in fire-resistance in the Type A beams of about 2 hours in all scales, the improvement being somewhat greater with the larger specimens. Extrapolation shows that a fire-resistance of 6 hours can be expected for the full size beam with 1 in. of vermiculite concrete protection. An <u>in situ</u> application of plaster $\frac{1}{2}$ thick on an A / $\frac{1}{4}$ beam gave an increase in fire-resistance of 30 minutes. Assuming similar behaviour in all scales to that obtained with vermiculite concrete $\frac{1}{2}$ in. plaster on the full scale Type A beam would raise the fire-resistance to 4 hours. The mean cable temperature at failure tended to be lower for encased than for unprotected beams. This behaviour agrees with the results of tests on cold-drawn wire, in which it has been found that the strength at a given temperature depends on the rate of heating.

Unaccountably high fire-resistances were obtained for protected Type B beams. For example the B / $\frac{1}{2}$ / 1 beam, No.709, had not failed after a 6 hour test. A water test of 6 minutes duration did not affect the beam which was able to support in addition to the test load of 2.35 tons before collapse. The temperature of the lower cable at the end of the test exceeded 800°C, a temperature at which the strength of the steel wires was negligible. No useful purpose would have been served in carrying out tests on the thicker protections on Type B beams.

(6) Effect of shape of section

Since Type B and Type C beams were closely similar in design except for the form of cross-section, the influence of shape of prestressed concrete beams on their fire-resistance is shown for rectangular and I sections, by a comparison of the performance of these two types. It can be seen from the graphs in Figure 8 that unprotected B and C type beams differed little in the scales tested and extrapolation indicates approximately the same fire-resistance for both types. The important fact brought out by the tests on the Type C beams was that the I section was no more liable to spall than the rectangular.

(7) Effect of temperature of cable

It will be seen from Figures 9, 11 and 13 that the curves for central deflection and cable temperature plotted against time of heating have the same general form, with the first changing slope a little later than the second. For type A unprotected beams collapse is imminent when the mean cable temperature exceeds 400°C. The results of some tests made on samples of the batch of cold drawn steel wire used in the beams to determine its strength-temperature properties are shown in Figure 19. If the wire was tested at constant temperature, that is, if the wire was raised to a particular temperature before the tension was applied, the hot strength of the wire fell rapidly after 200°C, until at 400°C only about 50 per cent of the original strength remained. When tested at constant load, that is with a tension equal to 50 per cent of the ultimate maintained throughout, failure of the wire occurred at 370°C. If relaxation of the wire from the initial stress was allowed to take place another set of results was obtained giving strengths up to temperatures of 350°C of the same order as those for constant temperature but increasingly lower strengths at higher temperatures.

During a fire test the strength-temperature relationship for the wires in a beam might be intermediate between the last two curves. The majority of failures in the fire tests were due to failure of the cables although actual fracture of the wires occurred only in the smaller beams. Under these conditions, the stress (t) in the cable at failure can be determined approximately from the relationship M=tdA, where M is the maximum bending moment, d the lever arm and A the area of steel.

For $A/\frac{1}{2}$ beams $M = {}^{WL}/8 = 11{\cdot}4 \times {}^{120}/8 = 171$ ton. in.

$d = 9''$ nearly and $A = 0{\cdot}273$ in^2.

then $t = {}^{171}/9 \times 0{\cdot}273 = 70$ tons/in^2. or assuming the ultimate cold strength to be 125 tons/in^2, 56% of the original. This figure is the same for $A/\frac{3}{8}$ and $A/\frac{1}{4}$ beams since they are scaled linearly.

It will be seen from Figure 19 that the wire is reduced to 56% of its original strength at about 380°C, which is a much lower temperature than that obtained at failure in the tests on Type A beams. The difference may be due to the fact that the thermocouples for measuring the temperatures of the wires were placed at mid-span where cracks invariably formed shortly before failure leading to high local heating of the cable and the relatively rapid increase in temperature shown on the graphs, Figures 9, 11 and 13. The strength reduction of the wires as a whole would not then be as rapid as the curves of Figure 19 indicate.

The residual strength of wire tested cold after heating to various temperatures is of interest for assessing the loss of prestress in beams heated for shorter durations than those required to cause collapse. The results of two series of tests are plotted in Figure 19. In the first series the wires were heated to pre-determined temperatures without stress, allowed to cool and then loaded to failure. For wires which were heated to less than 300°C, the cold strength is little effected.

The second series of tests was on wires heated to pre-determined temperatures after an initial stress equal to 50 per cent of the ultimate had been applied and relaxation allowed to take place. After cooling the wires were tested to failure and gave strengths which were lower than those obtained in the first series.

Heating cold-worked steel produces other effects which remain on cooling. It has been shown (5) that there is an appreciable residual expansion after cooling in wires heated between 150° and 450°C.

(8) Effect of concrete temperature

Information exists on the cold strength of concrete after heating to various temperatures, but this needs to be supplemented by an investigation of the strength of concrete at these temperatures while under load. These data, together with a knowledge of the change in properties of the cable with increase of temperature, will be valuable in analysing the results of the beam tests. An example of the temperature distribution in a Type A, $\frac{1}{2}$ scale beam after 1 hour heating is shown in Figure 20, and similar diagrams can be plotted for any time in the test. It is known from preliminary tests that concrete of the mix and strength used in the beams has 75 per cent of its original strength at about 340°C and 50 per cent at about 500°C. Such information is required in dealing with "over reinforced" beams where failure is determined by crushing of the concrete. The cold strength of concrete after heating to various temperatures will be required for assessing the extent of repair needed by beams which have suffered some damage from fire and the possibility of restoring prestress.

Conclusions

The following conclusions are put forward tentatively, as the result of tests on scaled-down beams of types which, although they may not be regarded as representative of practice, will nevertheless give valuable indications of the behaviour of post-tensioned beams at high temperatures. An opportunity to check the validity of the extrapolation method for obtaining the fire-resistance of large beams has arisen through the kind co-operation of the National Bureau of Standards, Washington, who have offered to test a number of 4/5 scale beams in their large furnace, together with a $\frac{1}{2}$-scale beam as a control.

(1) Time to collapse is determined largely by the rate of rise of temperature of the cable. A fire-resistance of 2 hours can be obtained with a concrete cover to the cable of about $2\frac{1}{2}$ in. Longer periods are likely if the cover is increased, but it may be desirable, to include a light reinforcement, say steel mesh, in the cover to the cable as a precaution against its spalling away if its thickness is increased beyond about 3 in.

(2) For a fire-resistance of 4 hours or more an insulating encasement is probably required. Normal in situ plastering with gypsum or cement/lime/sand may give up to $\frac{1}{2}$ hour additional resistance if there is an adequate key with the concrete. Protection incorporating vermiculite should increase the fire-resistance by about 2 hours when applied 1 in thick.

(3) Beams may fail a little earlier if longitudinal expansion is prevented than if they are free to expand. The effectiveness of the restraint is a determining factor, but the results so far do not permit a quantitative statement to be made.

(4) There is little difference in performance between a beam of rectangular section and an I-beam having the same load-carrying capacity and the same concrete cover to the cable.

(5) Explosive spalling, which may be a serious hazard with small units of the pre-tensioned type, does not seem likely to occur in post-tensioned beams having no part less than about 2 in. total thickness.

(6) Failure is unlikely to be sudden. There is a progressive sagging, which in beams of large span would be most noticeable. The formation and visible extension of cracks with a marked increase in deflection are signs that collapse is imminent.

(7) Beams which have been exposed to a fire of shorter duration than that which would cause failure, representing say less than half of their fire-resistance are likely to retain a high percentage of their original strength on cooling, but with a marked residual deflection and loss of prestress.

(8) Further work is needed to determine whether beams can be repaired after damage by fire. Repair would involve re-tensioning the cable to restore the prestress lost after even short heating.

References

(1) Ashton, L. A. The Fire Resistance of Prestressed Concrete Floors Civil Engineering and Public Works Review, Nov. and Dec. 1951.

(2) Baar, G. Tests on Prestressed Concrete Beams in Holland. Cement (Amsterdam) 1950 (13 and 14) C.C.A. Library Translation No. 21, 1950.

(3) Post-War Building Studies No. 20. Fire Grading of Buildings Part I. H.M.S.O. 1946.

(4) National Building Studies, Research Paper No. 12. Fire Tests on Structural Elements. H.M.S.O. 1953.

(5) J. H. Andrews and Others. Temperature and Cold-Worked Steel. Journal of the Iron and Steel Institute. June, 1950.

Acknowledgements

It is necessary to acknowledge the co-operation of the Building Research Station which enabled the investigation to be carried out. In particular, the assistance and advice of Dr. Thomas and Mr. Bate were invaluable.

Appendix 1

Requirements for the manufacture of the specimens

1. Concrete

Crushing strength (cube) at 28 days	6000 lb./sq.in.
Crushing strength (cube) at the time of tensioning the cables	4500 lb./sq.in.

2. Steel

Tensile strength of 12 S.W.G. wire	120 to 130 tons/sq.in.

3. Manufacture

Cables to be tensioned at not less than 7 days after the casting of the concrete.

The slabs to be cast 14 days after casting the prestressed concrete element.

The concrete to be cured for 7 days under damp sacking followed by storage in air.

The methods of tensioning the cables to be such that slip of the wires in grips is eliminated.

The cable cavities to be effectively grouted after tensioning the cables.

Maximum working stresses

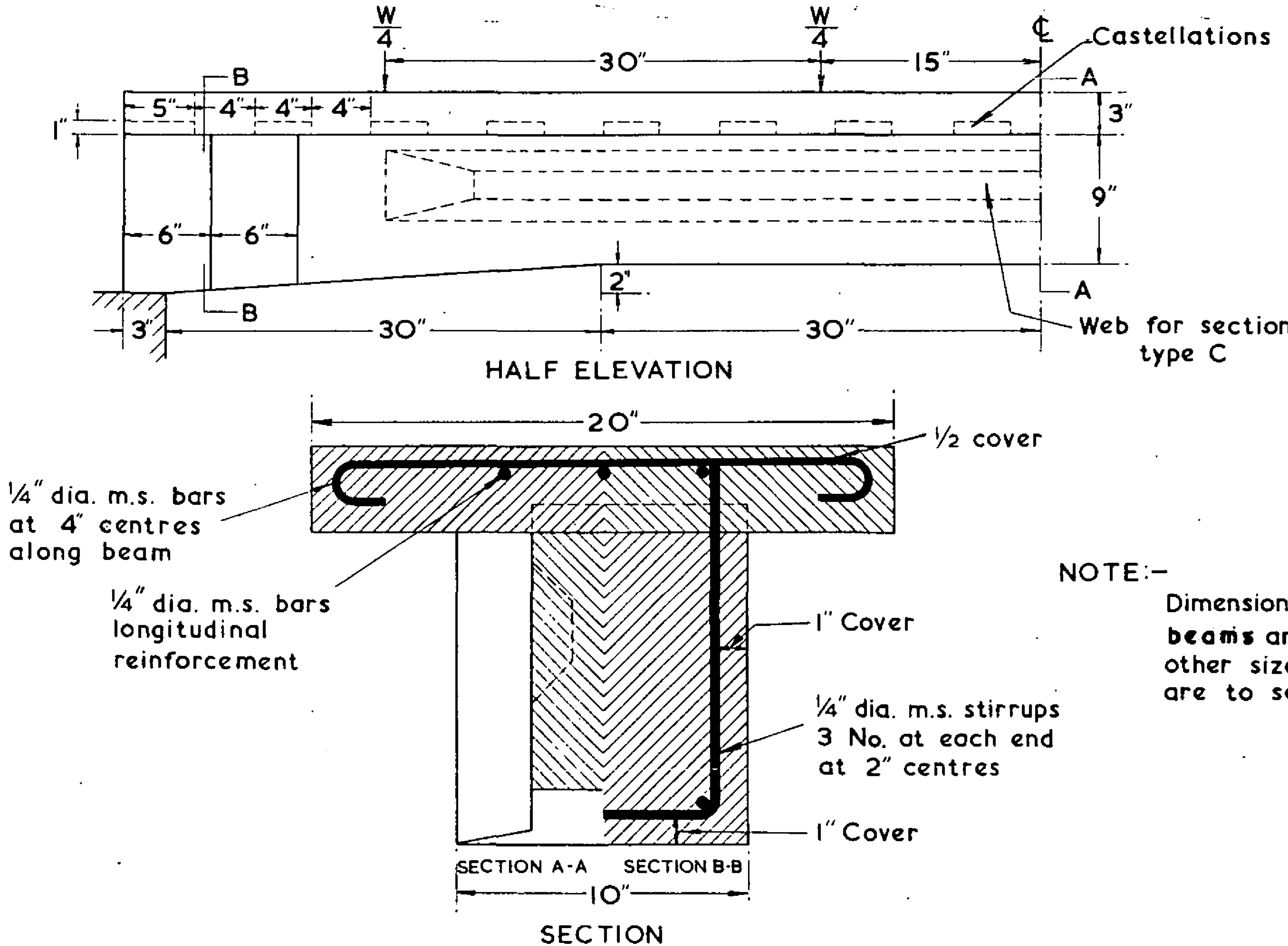
1. Concrete

Compressive stress in the prestressed concrete element under working loads. ($\frac{1}{3}$ cube strength at 28 days)	2000 lb./sq.in.
Compressive stress in unstressed slab	1000 lb./sq.in.
Initial precompression in the prestressed concrete element. ($\frac{1}{2}$ the cube strength at the time of stressing the cables)	2250 lb./sq.in.
Compressive stress in concrete under anchorages	2250 lb./sq.in.
Principal tensile stress for concrete	300 lb./sq.in.
Bending tensile stress for concrete	Nil.

2. Steel

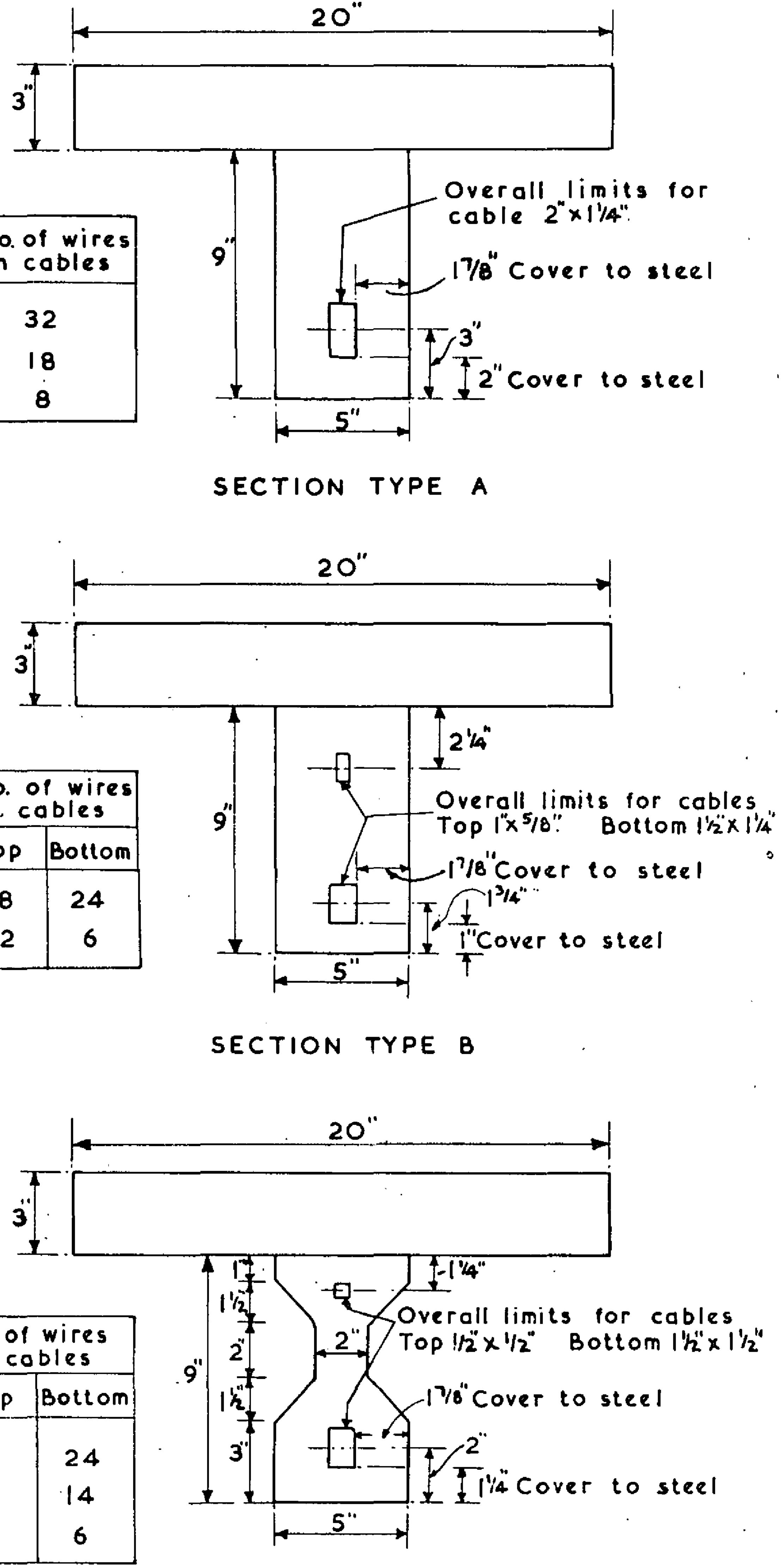
Initial pretension to be established without over-stressing. (0·6 approx. of the tensile strength)	75 tons/sq.in.

List of figures and tables.



NOTE:—
Dimensions for ½ scale beams are given, for other sizes dimensions are to scale.

FIG. 1. DIMENSIONS OF A/½ BEAMS



Scale	No. of wires in cables
1/2	32
3/8	18
1/4	8

SECTION TYPE A

Scale	No. of wires in cables	
	Top	Bottom
1/2	8	24
1/4	2	6

SECTION TYPE B

Scale	No. of wires in cables	
	Top	Bottom
1/2	4	24
3/8	2	14
1/4	1	6

SECTION TYPE C

Note. Dimensions for scale beams are given.
For other sizes dimensions are to scale

FIG. 2. CROSS SECTIONS OF SCALE BEAMS TYPES A, B & C

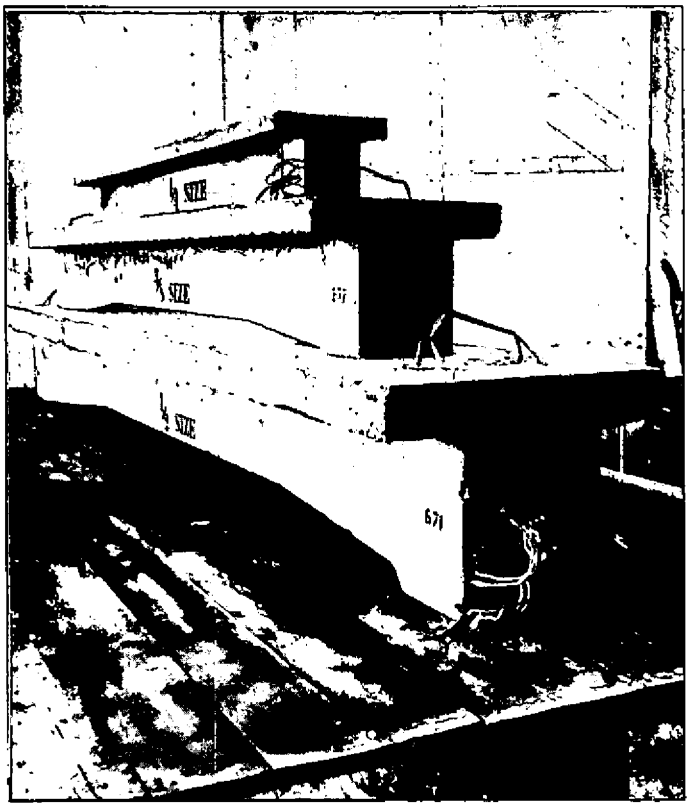


FIG. 3. THE UNPROTECTED TYPE 'A' BEAMS

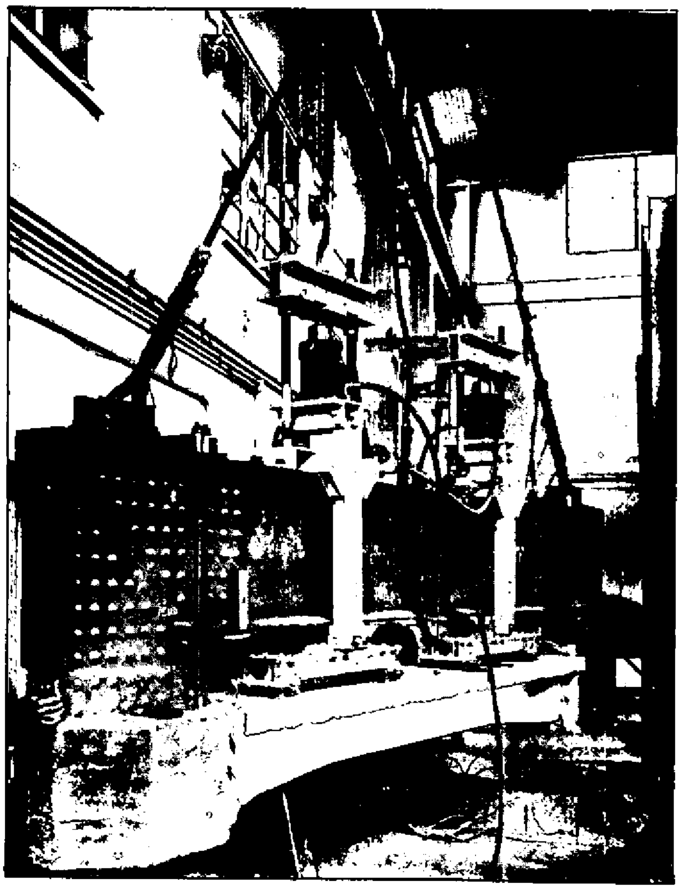


FIG. 5. AN A/1½ BEAM IN LOADING BRIDGE READY FOR TEST

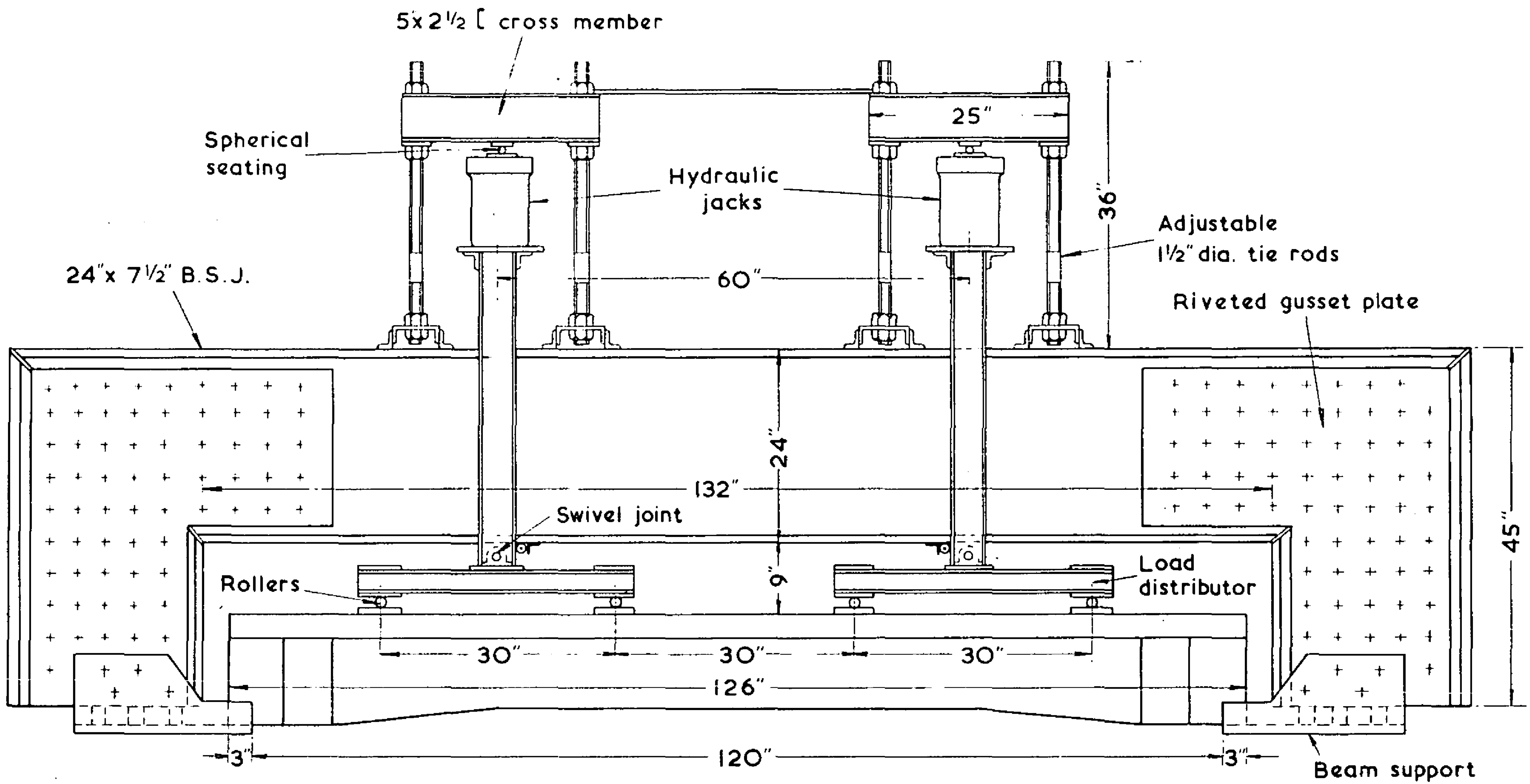


FIG. 4. HYDRAULIC LOADING BRIDGE WITH FOUR POINT LOADING FOR 10'-0" BEAMS.

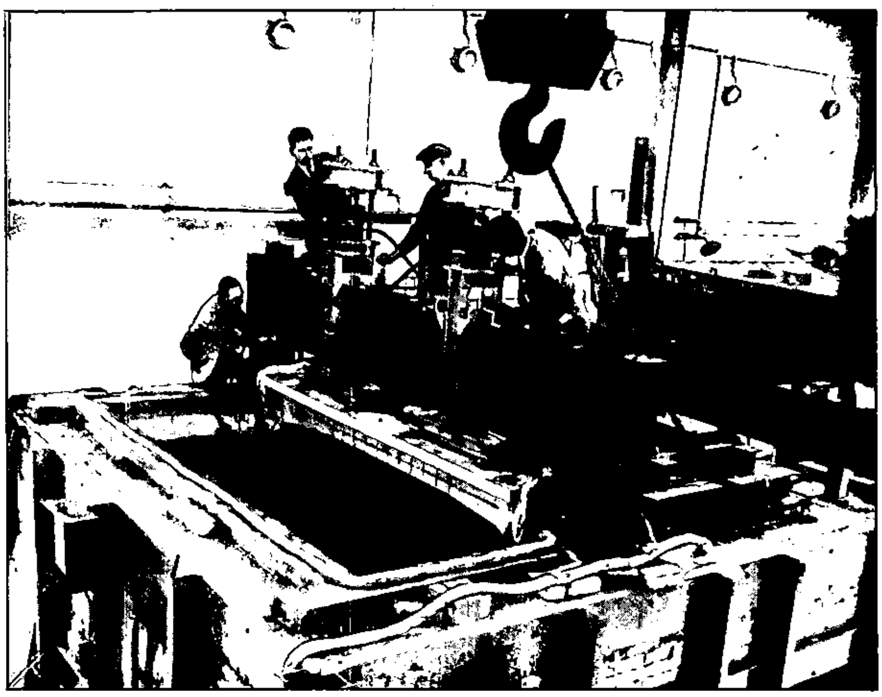


FIG. 6. BEAM IN LOADING BRIDGE BEING MOUNTED ON FURNACE

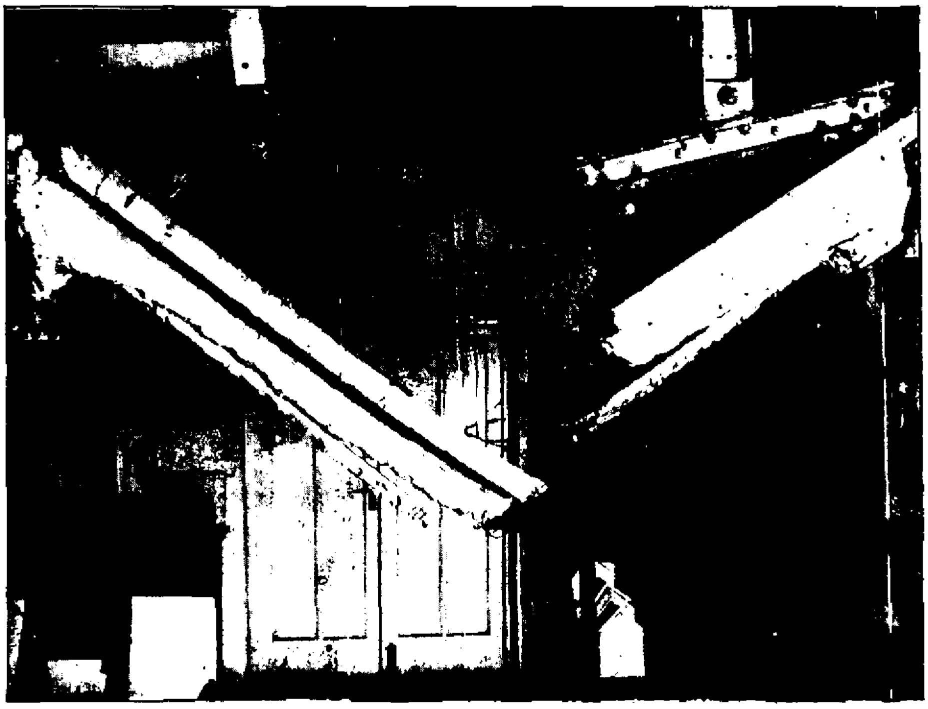


FIG.14. BEAM No.695 (A/3/8/O) AFTER TEST

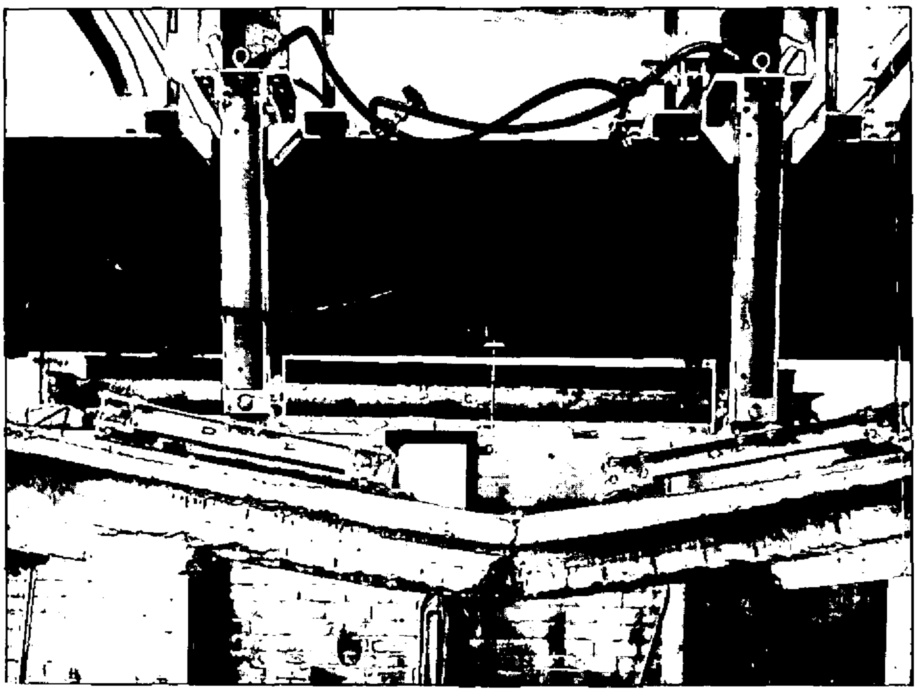


FIG.15. BEAM No.681 (A/1/2/O) AFTER TEST

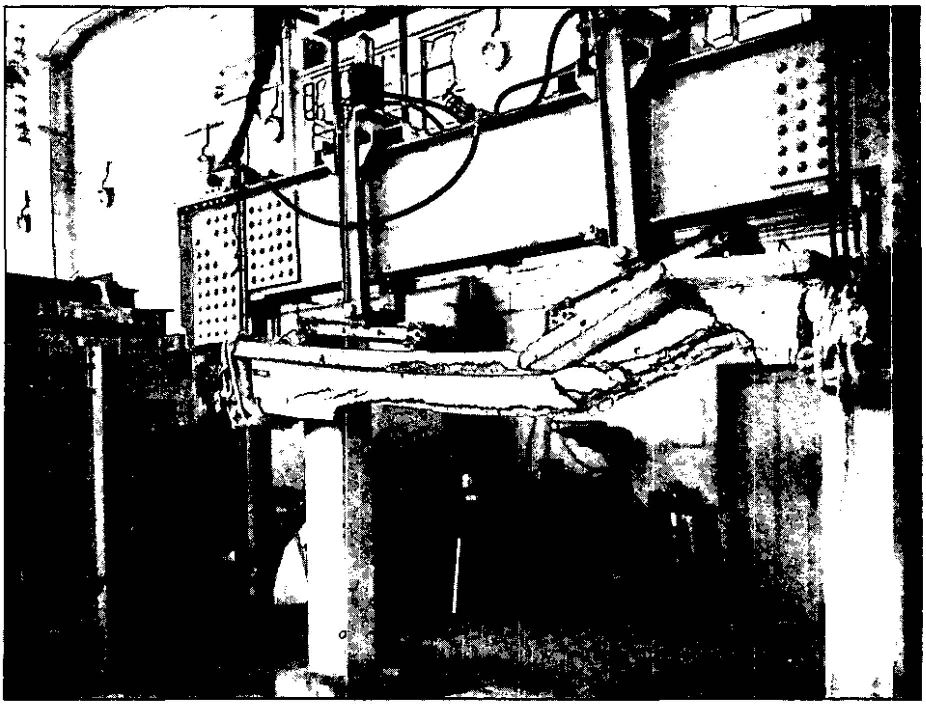


FIG.16. BEAMS No. 678 (A/½/O) AFTER TEST

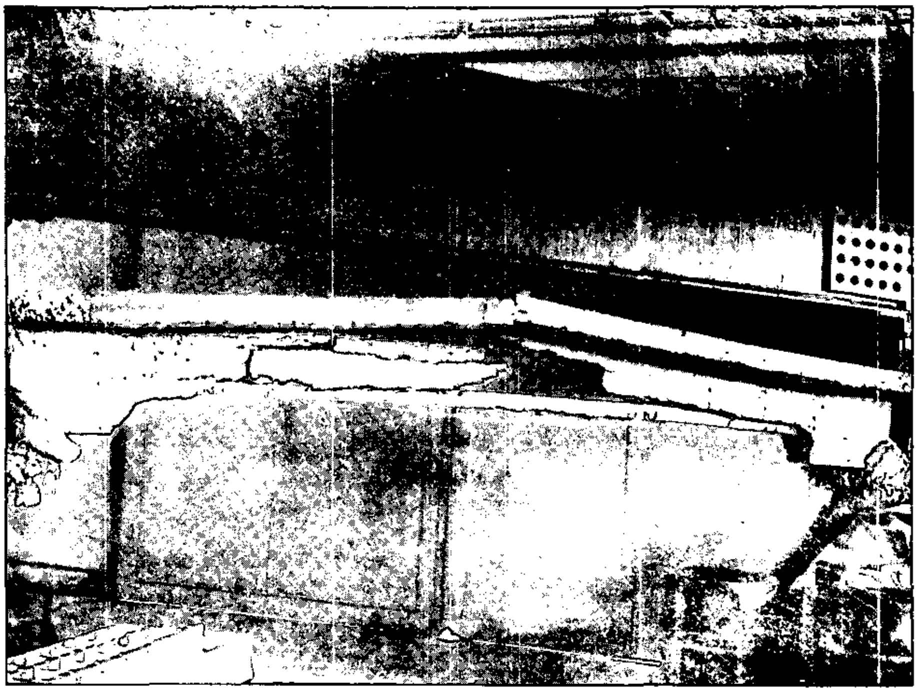


FIG.18. BEAM No. 675 (A/½/O-U/U) AFTER TEST

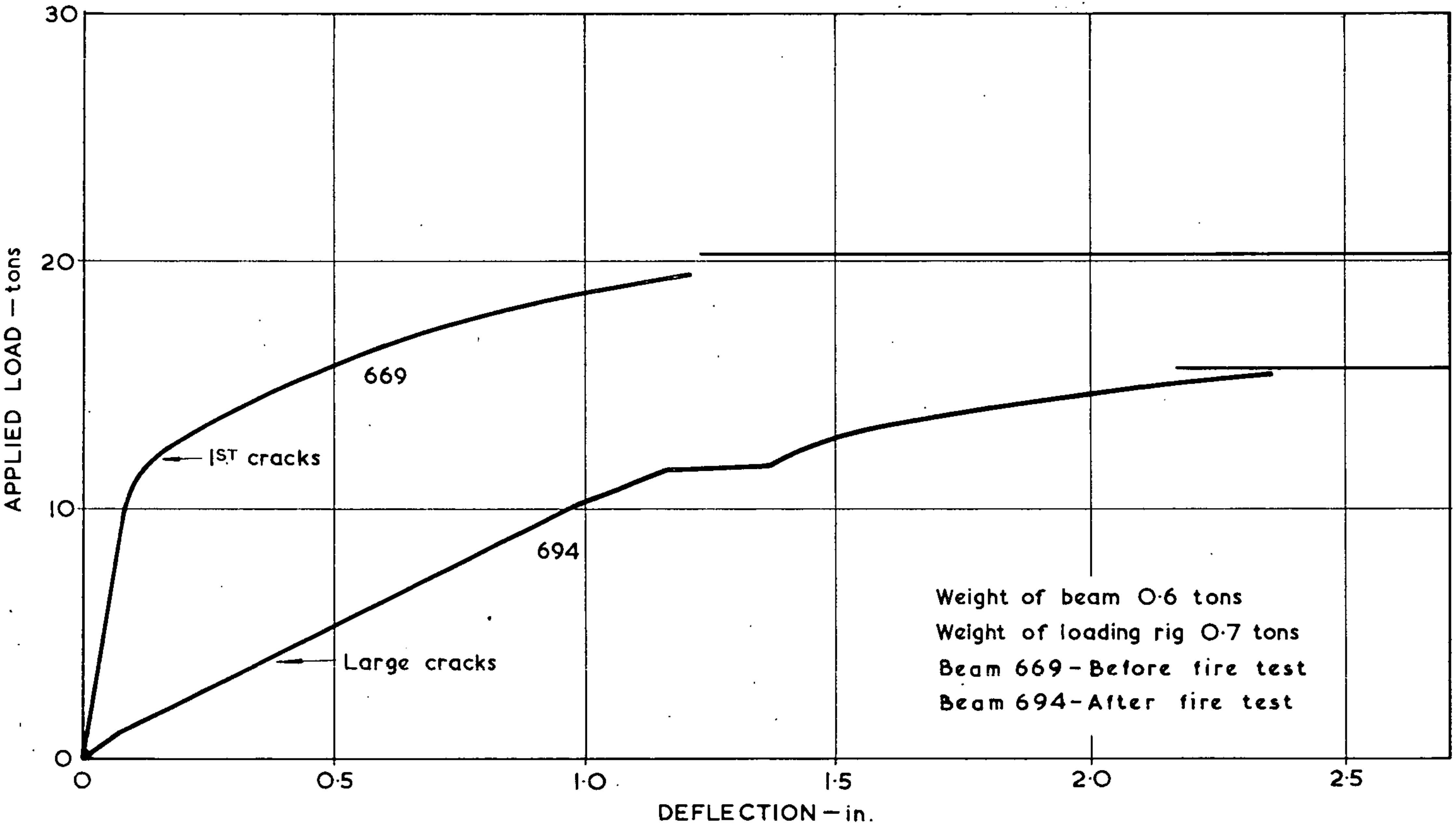
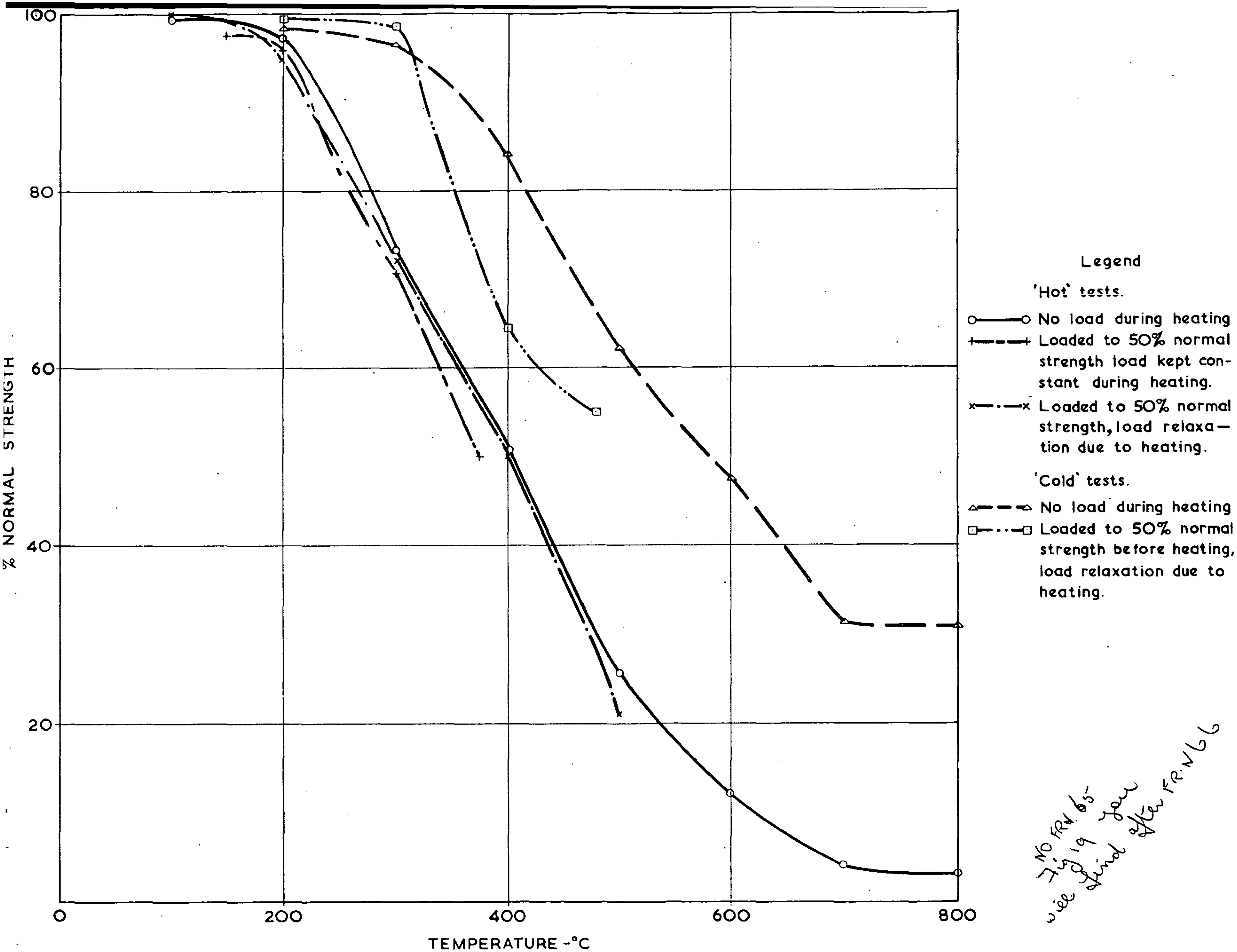


FIG. 17. LOAD/DEFLECTION CURVES FOR BEAMS BEFORE AND AFTER FIRE TESTS.



NO FRN 65
Fig 19 you will find after FR.N 66

FIG. 19. TEMPERATURE/STRENGTH CHARACTERISTICS OF 12G. COLD DRAWN STEEL WIRE.

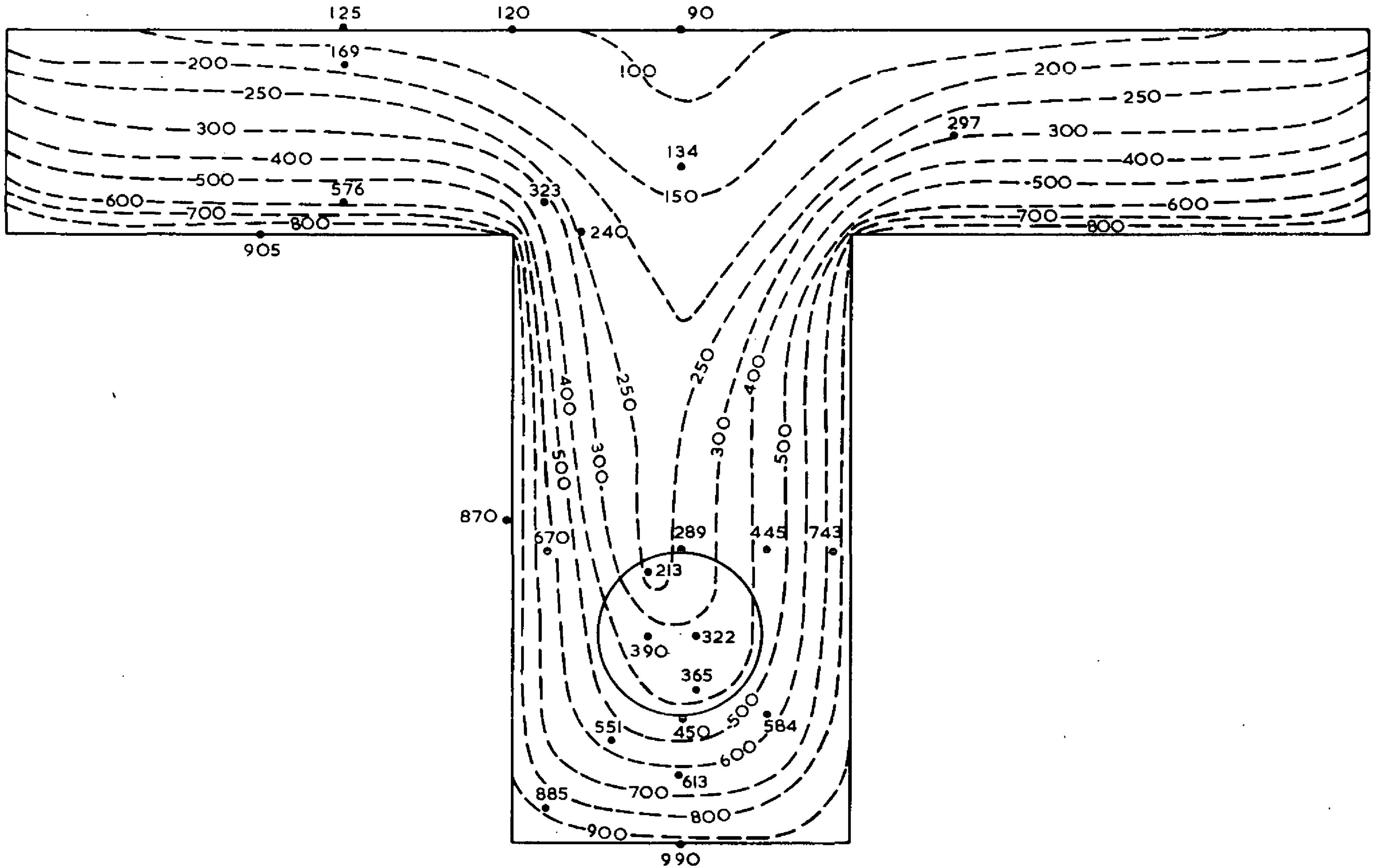


FIG. 20. TEMPERATURES IN CONCRETE OF A/½ BEAM AFTER 1 HOUR OF TEST.