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FIRE RESISTANCE OF STRUCTURAL CONCRETE BEAMS

by

H. L. MALHOTRA

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Fire Research Station,
Borehamwood,
Herts.
Tel. 01-953-6177

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INTRODUCTION

Fire protection requirements for buildings demand that all structural elements should be capable of resisting without collapse exposure to the heating conditions which may be experienced in the course of a fire. In practice the suitability of a construction is judged by exposing a representative specimen to a specified heating programme in a laboratory and observing its behaviour. On the basis of the performance the construction is assigned a certain degree of fire resistance, expressed at the time for which the specimen was able to satisfy the performance criteria.

In the case of elements such as beams which perform the sole function in a building of supporting either the direct loads or loads transferred through other elements such as floors, walls and columns, the only relevant criterion is that of structural stability. The specimens are required to resist exposure to the heating conditions specified in B.S. 476[1] : Part 1 without suffering collapse either during the heating cycle or the cooling period. It has been found experimentally that certain types of constructions with simply supported end conditions may, without suffering any actual collapse, undergo pronounced deflection which could lead to instability of the other elements which are being supported. This consideration has led to proposals being made in the course of the recent revision of B.S. 476 for a limiting value for the central deflection to be specified. The limit which it is being proposed should be placed on the maximum deflection is related to the clear span of the specimen and in future tests beams and floors would be required not only to retain their stability but also not undergo deflections in excess of $l/30$ where l is the clear span.

Fire tests on various types and kinds of beams have been carried out in the past and on the basis of this work structural codes of practice include clauses which give data for assessing fire resistance of constructions. A report[2] published in 1953 gave the limited data then available on beams of reinforced concrete and this was followed in 1960 by a report[3] of a more comprehensive series of tests on prestressed concrete beams. No tests on steel beams with concrete encasement have been conducted but computations have been made from the test data on concrete encased columns and the appropriate information included in the building byelaws and regulations.

Tests have also been carried out in other countries notably U.S.A. and Germany with emphasis in the more recent investigations on exploring the influence of the structural end conditions for prestressed concrete beams.

It was obvious from the published data that information on the performance of reinforced concrete and encased steel beams was very limited and that these constructions had not been subjected to the same close scrutiny as the prestressed concrete. Discussions with the Building Research Station led to the formulation of a co-operative research investigation on this subject with the Building Research Station agreeing to undertake the design and the manufacture of beams and the Fire Research Station obtaining data on their performance. It was considered desirable to compare perhaps for the first time the performance of the three types of beams designed for an identical purpose and to explore the influence of a number of factors including the conditions at the supports. Inevitably the number of factors had to be limited to prevent the programme becoming too large and as a consequence there would seem to be a need for some subsequent investigation for further information on the aspects which have not been fully explored in the present series.

DESIGN OF THE PROGRAMME

The programme as originally designed consisted of 13 beams of 7.6 m (25 ft) length to be tested as simply supported specimens over 7.3 m (24 ft) span. With three exceptions all the beams were designed for a fire resistance of 4 hours following the recommendations in the Codes of Practice and Byelaws. Five additional specimens of 11.3 m (37 ft) length were made which were of an identical design to the equivalent 7.6 m (25 ft) beams and were intended to be supported over the same span but with cantilever ends loaded to produce conditions of negative bending moment over the supports. During the course of experiments it became apparent that the performance of the reinforced concrete beams made with gravel aggregate was lower than anticipated owing to the spalling of concrete cover to the reinforcement. This led to the manufacture of another six specimens of reinforced concrete with and without supplementary reinforcement and having concrete covers in the range of 25-63 mm (1-2.5 in). In all fire tests have been performed on 24 beams in this programme including one repeat test.

The following main factors were included for examination:

1. Type of beam (a) reinforced concrete (b) prestressed concrete (c) encased steel.

2. Type of concrete (a) dense concrete (gravel aggregate) (b) lightweight concrete (expanded clay and foam slag aggregates).

3. Type of steel (a) mild steel (b) cold worked steel (c) hot rolled alloy steel.

4. Thickness of concrete cover. This was varied in the case of reinforced concrete beams from 25 to 63 mm (1 to 2.5 in).

5. Supplementary reinforcement to minimize the effect of spalling.

6. End conditions (a) simply supported (b) simply supported with continuity.

A brief specification of the different test beams is given in Tables 1, 2 and 3.

TABLE 1

Prestressed concrete beams

No.	Type of concrete	Type of beams	Shape of cross section	Thickness of concr. cover	Supplementary reinforcement
		a) 7.6 m (25 ft) long specimens			
1	Dense (gravel aggregate)	Post-tensioned with tendons	Rectangular	100 mm(4 in)	Yes
2	"	" "	"	" "	"
3	"	Pre-tensioned with tendons	I-section	50 mm(2 in)	No
4	"	Pre-tensioned with strands	"	" "	Yes
5	"	" "	"	" "	No (Encasement of 13 mm($\frac{1}{2}$ in) vermiculite/ gypsum plaster)
		b) 11.3 m (37 ft) long specimens			
6	Dense (gravel aggregate)	Post-tensioned strands	Rectangular	100 mm(4 in)	Yes
7	"	" "	"	" "	"

TABLE 2

Reinforced concrete beams

No.	Type of concrete	Type of reinforcement	Thickness of concrete cover	Supplementary reinforcement
		a) 7.6 m (25 ft) long specimens		
8	Dense (gravel aggregate)	Mild steel	63 mm ($2\frac{1}{2}$ in)	None
9	"	Cold worked deformed	" "	"
10	"	Cold worked twisted	" "	"
11	"	Hot rolled alloy steel	" "	"
12	Light weight (expanded clay)	Mild steel	" "	"
13	Light weight (foamed slag)	"	" "	"
14	Dense (gravel aggregate)	"	" "	Yes
15	"	Hot rolled alloy steel	" "	Yes
16	"	Cold worked twisted	" "	Yes
17	"	Hot rolled	38 mm ($1\frac{1}{2}$ in)	Yes
18	"	"	" "	None
19	"	"	25 mm (1 in)	"
		b) 11.3 m (37 ft) long specimens		
20	Dense (gravel aggregate	Mild steel	63 mm ($2\frac{1}{2}$ in)	None
21	"	Cold worked deformed	" "	"

TABLE 3

Encased steel beams

No.	Type of concrete	Type of steel	Shape of steel section	Thickness of concrete cover	Supplementary reinforcement
		a) 7.6 m (25 ft) long specimens			
22	Dense (gravel aggregate)	Mild steel	I-section	63 mm ($2\frac{1}{2}$ in)	Yes
23	"	Hot rolled alloy steel	"	" "	Yes
		b) 11.3 m (37 ft) long specimen			
24	Dense (gravel aggregate	Mild steel	I-section	63 mm ($2\frac{1}{2}$ in)	Yes

The beams were of a rectangular section, with the exception of specimen Nos 3, 4 and 5, and were provided with a cast slab at the top to give a T-beam profile. The top slab was provided primarily to simulate the exposure of the beams to the heating conditions as would be experienced in practice when forming part of a floor construction. This arrangement also facilitated application of load on the upper surface of the T-slabs.

One of the factors which can influence the behaviour of a beam under fire conditions is the area of the exposed surface through which heat can be transferred to the inside section of the beam. To eliminate any variations in performance due to this factor most of the beams were designed to have the same cross sectional area and the same width. This also produced specimens of similar heat capacity. The exception to this general arrangement were designed to a notional 2 hour fire resistance and 11.3 m (37 ft) long prestressed beam No.7.

The beams were made at the Building Research Station under controlled laboratory conditions. Concrete was designed for a cube strength of 5000 lb/in^2 at 28 days and test cubes were cast for control purposes. During the manufacture of the beams thermocouples were attached at selected points to the reinforcement and imbedded in the concrete section in some selected specimens to obtain a record of the temperature conditions within the section.

Beams were made in two lengths of 7.6 m (25 ft) and 11.3 m (37 ft). The 7.6 m (25 ft) specimens were intended to have a notional span of 7.3 m (24 ft) achieved by providing 300 mm (12 in) wide bearing plates at each end. The 11.3 m (37 ft) long specimens were supported at identical points along the span thereby giving 2 m ($6\frac{1}{2}$ ft) long cantilever ends which projected outside the furnace.

(a) Prestressed concrete beams

Prestressed concrete beams of 7.6 m (25 ft) length and rectangular cross section had the same overall dimensions as the reinforced concrete beams. These beams were of the post-tensioned type using 5 mm (0.2 in) diameter tendons. The beams were provided with a 50 mm (2 in) dia. duct with flexible metal liners and a cable consisting of 40 tendons was inserted from one end to the other. After tensioning the tendons were anchored at the end with special grips and the ducts filled with grout.

The position of the duct was such that a concrete cover of 100 mm (4 in) was provided at the soffit and 115 mm (4½ in) from the two vertical sides. At a distance of 25 mm (1 in) from the exposed face stirrups were provided to act as a supplementary reinforcement against spalling. Stirrups of 5 mm (3/16 in) dia. mild steel bars were welded to the 11 mm (7/16 in) longitudinal bars at four corners to form a cage.

Two beams of 11.3 m (37 ft) length were also of the post-tensioned type one having seven 5 mm (0.2 in) dia. tendons in a single duct, and the other was provided with a second cable of 5 mm (0.2 in) tendons at higher level. The former (Specimen No.20) had a width of 280 mm (11 in) and a depth of 380 mm (15 in) in the central 3.3 m (11 ft) of the span, the depth of the beam was increased to 610 mm (24 in) at the supports and again decreased to 380 mm (15 in) at the extremities of the cantilevered section. The cable duct was straight in the section between the supports and had a lift of about 63 mm (2½ in) in the cantilevered parts. The other long beam (Specimen No.7) was of a uniform 710 mm (28 in) depth with the top cable running straight and the lower cable raised at the supports by a distance of 150 mm (5½ in). No other reinforcement was provided in either of these two beams over the supports to counteract negative bending moments.

The I-section beams of 7.6 m (25 ft) length were only 180 mm (7 in) wide at the base and had a total depth of 355 mm (14 in). The web in the intermediate part had a width of only 76 mm (3 in). All I-section specimens were of the pre-tensioned type, one being provided with twenty 5 mm (0.2 in) dia. wires and the other two with four 13 mm (½ in) dia. strands. Specimen No.3 with 5 mm (0.2 in) tendons had a 51 mm (2 in) concrete cover to the tendons nearest to the soffit and the sides. Specimen Nos 4 and 5 with the strands had a minimum cover of 63 mm (2½ in) to any one of the strands. One of the beams with strands was provided with additional supplementary reinforcement consisting of 5 mm (3/16 in) dia. links passing through the web and around the strands. The top slabs for these beams had a depth of only 127 mm (5 in). Specimen No.5 was provided with an insulating encasement consisting of 13 mm (½ in) thick vermiculite/gypsum plaster on the three exposed faces.

(b) Reinforced concrete beams

The reinforced concrete beams were provided with tensile reinforcement consisting of six bars arranged in two layers of three each. An arrangement of double links of 11 mm (7/16 in) dia. mild steel bars was used and these were spaced at 305 mm (12 in) centres except for 915 mm (3 ft) length at each end when the spacing was halved. The slab at the top of the 280 mm (11 in) wide x 380 mm (15 in) deep beam measured 150 mm (6 in) deep x 810 mm (32 in) wide and was reinforced with 11 mm (7/16 in) dia. hooked bars at 150 mm (6 in) centres. The position of the bars was such that for specimens Nos 8 to 16 and 20 and 21 the concrete cover to the two outermost bars was 63 mm (2½ in) from the soffit as well as the sides. For specimen Nos 17 and 18 the bars were re-positioned so that the concrete cover was reduced to 38 mm ($1\frac{1}{2}$ in) whilst in the case of specimen No.19 it was only 25 mm (1 in).

Specimen Nos 14, 15, 16 and 17 were also provided with supplementary reinforcement consisting in the case of specimen No.14 of expanded metal lath of 125 mm x 75 mm (5 in x 3 in) mesh and in all other cases of hard drawn steel wire fabric having a 150 mm x 100 mm (6 in x 4 in) mesh and wires of 3 mm (12 B.G.) diameter. The supplementary reinforcement was positioned half way between the exposed faces and the outermost main bars.

In the case of 11.3 m (37 ft) long beams a similar arrangement of reinforcement was used over the central 4.3 m (14 ft) length beyond which the upper three bars were bent towards the top and three additional bars of short length were provided over the supports to resist the negative bending moments.

(c) Concrete encased steel beams

The steel section used in these beams was a 152 mm x 405 mm x 74.5 kg (6 in x 16 in x 50 lb) British Standard beam of mild steel for specimen Nos 22 and 24 and of hot rolled alloy steel for specimen No.23.

The minimum thickness of concrete cover to a bottom flange was 63 mm ($2\frac{1}{2}$ in) at the soffit and the sides. Around the beams and at a distance of 25 mm (1 in) from the exposed faces a cage of 5 mm (3/16 in) dia. links at 150 mm (6 in) centres was provided to act as supplementary reinforcement. The links were provided with two carrier bars at the bottom and were spot welded to the edges of the upper flange of the beams at the top. No special reinforcement was provided over the supports for the 11.3 m (37 ft) long beam. Cross-section of the encased steel beams is shown in Fig.4.

TEST PROCEDURE

After manufacture the beams were stored under cover in the laboratory for a period of up to 3 years before being subjected to the fire tests. This was considered desirable to ensure that the concrete would have reached stable moisture conditions throughout the section with the laboratory atmosphere.

The fire tests were carried out using the horizontal beam and floor furnace which has an internal opening of 6.9 m x 3 m (22 ft 9 in x 10 ft). The ends of the beams were positioned over the specially strengthened recesses in the end walls and were supported over steel bearing plates. The positioning of the beams was such that the top slab projected about 150 mm (6 in) above the top of the furnace. The two openings on the sides of the slabs were closed by means of refractory concrete slabs, the butting edges being sealed with asbestos rope to permit free deflection of the specimen during a test.

Loads were applied to the slab at 4 points, $\frac{1}{8}$ and $\frac{3}{8}$ of the notional span from each support, by means of two hydraulic jacks each provided with a load spreader as shown in Fig.6. The jacks were attached to portal frames which were designed to be quickly connected and disconnected to permit easy removel of the beams after a fire test.

The beams with cantilevered ends had loads applied to the projecting parts by means of iron weights carried in specially made baskets suspended below the beams with the loading point 1.8 m (6 ft) from the centre of the support. This arrangement of end loading applied no restraint to the longitudinal or angular movement of the beam ends and provided conditions of 'pure continuity'. The projecting ends of the beams were not subjected to any heating.

Thermocouples were inserted into the furnace between the beam and the furnace cover slabs on both sides and were used to control the fuel input into the furnace such that the heating conditions followed the standard time/temperature relationship of B.S. 476 : 1953. Thermocouples cast in the beams were connected to temperature recorders which made a continuous record of their readings. The central deflection of the beams was measured using a gauge mounted over the slab. Observations of the general behaviour of the beams were made visually through port holes in the walls of the furnace and in addition a cine camera took time lapse shots of the specimens in the furnace through one of the specially prepared observation holes.

The beams were loaded just prior to the test and the test load was kept constant for the whole duration of the heating in conformity with the requirements of the British Standard for fire resistance tests. The heating of the beams was usually terminated before collapse took place by keeping a close watch on the deflection readings which indicated the imminence of collapse by a rapid increase in the rate of deflection. In three cases actual collapse of the beams took place before the heating was turned off. This occurred with the two 11.3 m (37 ft) long and one I-section prestressed concrete specimen. A brick pier was built below the soffit of the beams to prevent their falling to the bottom of the furnace in such a case.

TEST RESULTS

GENERAL

The complete log for each test is given in the Appendix and the temperature and deflection curves for various beams are shown in Fig.7 to 16. The appearance of some of the beams before, during and after the test are shown in Plates 1 to 20. The temperature records for a number of beams are not complete owing to the damage suffered by the thermocouples during storage or during the progress of a test. Brief summaries of the test results are given in Tables 4 to 7. To provide a common basis for the comparison of performance of different beams which did not actually collapse computed times for the critical deflection of 1/30 have been shown in the last column of the tables. These times may be taken as the effective fire resistance of various specimens.

The first two tests were carried out on identical specimens (Nos 1 and 2) to investigate the effect of the method of support at the ends. With the first specimen a roller bearing was interposed between two steel bearing plates at one end and a half roller bearing at the other, whereas in the case of specimen No.2 the bearing was provided by the flat surfaces of steel plates. There was very little difference in the overall performance of the two beams. However, the specimen with roller supports (No.1) had slightly greater initial deflection 56 mm (2.2 in) instead of 36 mm (1.4 in) at 1 hour; the difference in the deflection was maintained until $3\frac{1}{2}$ hours and as heating proceeded the deflections became the same at 4 hours. The mean steel temperature at the end of the test in two cases were within 15 degC of each other. As the method of support had only a little effect it was decided to test the remainder of the specimens with flat steel bearing plates without the use of rollers.

The tests were generally terminated when deflection became excessive and the beams were almost resting on the central pier. Only in three cases did the beams actually collapse (specimen Nos 3, 6 and 7) before the test could be terminated.

In cases where the beams had not actually failed the collapse of the specimen was imminent and failure would have taken place had the heat exposure been continued for a few extra minutes.

PRESTRESSED CONCRETE (TABLE 4)

Specimen Nos 1 and 2 were designed following the specification in CP.115 and were provided with supplementary reinforcement (Fig.1). The thickness of concrete cover at the soffit and the sides was 100 mm (4 in) and 115 mm (4.5 in) respectively giving a mean cover thickness of 110 mm (4.3 in). At the end of the heating the mean steel temperatures were such that failure was imminent.

Specimen Nos 3, 4 and 5 were of 'I' shape with a web thickness of 75 mm (3 in) and a flange width of 178 mm (7 in). The reduction in the section at the web was considered to be a point of weakness liable to damage by spalling and therefore specimen No.4 was made with an arrangement of links in the web and around the prestressing steel.

Specimen No.3 with an arrangement of twenty prestressing wires failed by the facture of the web suddenly at 32 min whereas with the introduction of the supplementary reinforcement in the web the performance of specimen No.4 was increased by about 1 h to 1 h 38 min.

Specimen (No.5) which was otherwise identical to No.4 was given a coating of 13 mm ($\frac{1}{2}$ in) vermiculite/gypsum plaster. This specimen, by virtue of the protection provided by the plaster encasement, survived for 3 h 15 min.

REINFORCED CONCRETE (TABLE 5)

The reinforced concrete specimens Nos 8-13 were designed following the specification in CP.114 to have a fire resistance of 4 h. They were not provided with supplementary reinforcement as the Code does not require it to be included. The test results on the first four specimens, all made with gravel aggregate concrete, were worse than expected with failure becoming imminent after $1\frac{1}{2}$ hours. This adverse performance was caused by the premature spalling of concrete from the soffit and the sides exposing the reinforcement and leading it to its rapid rise of temperature. As a result of these tests, specimen numbers 14-19 were made, additional to the original programme, to investigate the performance of beams with supplementary reinforcement and to determine its need when cover thicknesses were small.

Specimen Nos 12 and 13 made with lightweight aggregate concrete which did not suffer spalling, withstood heating for 6 h without suffering collapse, with mean reinforcement temperatures below 550°C.

Specimen No.14 with 63 mm ($2\frac{1}{2}$ in) concrete cover to the reinforcement and with supplementary reinforcement consisting of expanded metal lath failed at 2 h 51 min, owing to the inability of the lath to retain the concrete cover in position successfully. Examination of the beam after the test showed that the lath was bent to shape with the larger diagonal of the mesh running parallel to the length of the beam. As development of cracks in the concrete cover took place, the lath in the cracked areas became hot and was unable to support the weight of the partially detached concrete pieces. When the expanded metal lath was replaced by a steel wire fabric (specimen Nos 15 and 16), the concrete cover was retained in position more successfully, resulting in an improvement in performance to at least $4\frac{1}{2}$ h. Beams with 38 mm ($1\frac{1}{2}$ in) cover and with and without supplementary reinforcement (specimen Nos 17 and 18 respectively) gave effective fire resistances of 4 h 16 min and 2 h 40 min respectively, showing clearly the significant contribution supplementary reinforcement is capable of making. Without supplementary reinforcement, the performance of beams with 25 mm (1 in) and 38 mm ($1\frac{1}{2}$ in) covers (specimen Nos 19 and 18) were almost identical.

CONCRETE ENCASED STEEL (TABLE 6)

Comparison was made between two specimens (Nos 22 and 23) of identical size and shape using mild steel and hot rolled alloy steel beams respectively. The latter showed slightly greater deflection at corresponding times - at 5 h its deflection was 214 mm (8.4 in) against 168 mm (6.6 in) for the mild steel section, although the temperature of the lower flange was similar, around 700°C. When the tests were terminated neither of the beams had collapsed and their respective computed fire resistances were 5 h 40 min and 5 h 12 min.

BEAMS WITH CANTILEVERED ENDS (TABLE 7)

Five beams of 11.3 m (37 ft) length were tested with simulated conditions of continuity over the supports. The two reinforced specimens (Nos 20 and 21) owing to the absence of supplementary reinforcement, behaved in a similar manner to their corresponding simply supported versions. Loss of the gravel aggregate concrete cover by spalling led to the imminence of collapse after $1\frac{1}{2}$ hours.

The prestressed concrete specimens were provided with means of countering reverse bending moments by increasing the depth of concrete at supports in the beam with one cable, that is specimen No.6, and by raising the cable centre over the supports, in the case of specimen No.7, which was provided with two cables. Both specimens were provided with supplementary reinforcement. The performance of the two specimens were similar, both suffering collapse in the test by the development of three structural hinges, one in the middle of the span and the other two close to each support. The hinges were formed within a very short time of each other. The extensive damage to the concrete in the compression zone close to the supports is shown in Plate 19. The fire resistance of the continuous specimens was only marginally better than the simply supported beams of an identical design (Nos 1 and 2).

The encased steel beam (specimen No.24) gave results which were not much different from the corresponding simply supported specimen (No.22), the deflections in the latter occurring at a slighly slower rate. The cantilevered ends of the beams did not suffer any downward movement indicating the adequacy of the steel section to resist the negative moments over the supports.

DISCUSSION OF RESULTS

1. Spalling of concrete

Of the 24 specimens tested, two were made with lightweight aggregate concrete and the remainder with dense concrete using river gravel for coarse aggregate and pit sand for the fines. There was a pronounced difference in the behaviour of the two concretes under fire conditions; whereas the lightweight concrete withstood heating for 6 h without showing any signs of spalling or fall of concrete cover, the gravel aggregate concretes invariably showed signs of damage by spalling within the first $\frac{1}{2}$ h of the test. The extent of damage varied in severity from the fall of small amounts from the arrisses to large-scale spalling from the soffit and the sides, exposing the reinforcement.

It would seem that upon subjection to the heating conditions, cracks develop in the concrete cover along the arrisses and slightly higher up following the line of the reinforcement. As heating proceeds the cracks widen and spread possibly assisted by the differential expansion at the interface between steel and concrete and by the evolution of water vapour. The exact mechanism of spalling is complex and is not fully understood. It is, however, known from experience that it occurs generally with gravel aggregate concretes and its magnitude is influenced by the shape of the section and the distance from the exposed face to the reinforcement. When the distance from the face of the beam to the steel - be it the main bars, wires or cables, or supplementary reinforcement - is less than about 40 mm (1.6 in) the fall of concrete is generally confined to the arrisses resulting in a rounding off of the corners without any other more serious damage. However, when this distance is increased, the mass of concrete is unable to retain itself in position and extensive spalling can occur leading to an earlier collapse of the beam than expected on the basis of the original concrete cover.

The premature failure of the reinforced concrete beams made with gravel aggregate concrete and having a cover of 63 mm ($2\frac{1}{2}$ in) was entirely due to spalling and its occurrence in four cases leaves no doubt about the validity of these results. The specimens which incorporated supplementary reinforcement 25 mm (1 in) below the exposed surface suffered only slight damage at arrisses and gave at least the expected performance and in some cases better than expected. It would therefore seem imperative that when dealing with concretes which have a tendency to spall, such as those made with silicious aggregates, provision should be made for the mitigation of the effects of spalling. The preventive measures consist of the introduction of supplementary reinforcement of a suitable design or the use of an insulating encasement (specimen No.5) which reduces the thermal gradient across the concrete section.

2. Supplementary reinforcement

To be effective it is essential that supplementary reinforcement should be of adequate strength to retain the concrete mass around the main reinforcement in position after cracking has taken place. The expanded metal lath of the type used in specimen No.7 was unable to keep the concrete cover of 63 mm ($2\frac{1}{2}$ in) thickness in place for very long, whereas the steel wire fabric, having a 125 mm x 75 mm (4.9 in x 3 in) mesh and wires of 3 mm (12 B.G.) dia. gave a satisfactory performance. It may be that for smaller cover thicknesses, that is 40 mm (1.6 in) or less, expanded metal lath may prove satisfactory. The supplementary reinforcement consisting of a cage of 5 mm (3/16 in) steel stirrups at 150 mm (6 in) centres was used for the rectangular prestressed concrete beams (specimen Nos 1, 2, 6 and 7) and for the concrete encased beams (specimen Nos 22, 23 and 24) and it gave a satisfactory performance. Another use made of the supplementary reinforcement was to prevent collapse of the thin section joining thicker parts as the webs in the 'I' section beams. In the absence of such reinforcement specimen No.3 failed in just over 30 min whereas, by its presence, the failure of specimen No.4 was delayed by about 60 min.

3. Lightweight aggregate concretes

In comparison with dense silicious aggregate concretes the two types of lightweight concretes used in the investigation showed themselves to be free from the phenomenon of spalling.

An examination of the temperature curves in Fig.9 shows that the reinforcement temperature rose at a slower rate after the evaporation of the moisture shown by the flat part of the curves at just above 100°C. If specimens 12 and 15 are taken as representative of the two concrete types, the reinforcement attained temperatures of 550°C at 360 and 260 min respectively, the lighweight concrete taking 38 per cent longer owing to lower thermal diffusivity. The thickness of concrete cover necessary to limit the temperature rise to a specified limit for a given size of beam is inversely proportional to the square root of thermal diffusivity. Using a tentative method for estimating the thickness of cover for concrete beams on the basis of thermal diffusivity, it would seem that with lightweight concrete a reduction of cover of about 20 per cent is possible, for similar performance in fire.

4. Type of steel

Owing to the premature failure of the reinforced concrete beams (specimen Nos 8, 9, 10 and 11) it was not possible to make a direct comparison between different types of reinforcing steels as had been originally planned. Even specimen Nos 14 and 15 do not permit a comparison to be made between mild steel and hot rolled alloy steel owing to the supplementary reinforcement in the former not being entirely satisfactory. Data from specimen Nos 15 and 16 show that limiting deflection of $^{1}/30$ was reached at 4 h 48 min and 4 h 30 min by the beams reinforced with hot rolled alloy steel and cold worked twisted steel, respectively. Deflection of the beams is influenced by the temperature of the reinforcement. The mean steel temperatures 600 and 550°C at the time of critical deflection indicating that the alloy steel gave about 10 per cent better performance.

Comparing the results of the two concrete encased steel beams it would seem that on the same basis the mild steel beam gave a somewhat better performance than the hot rolled alloy steel beam. However, these results can be regarded only as indicative of the differences between different steels and some further comparative tests are necessary before definite relationships can be established.

5. Type of beam

The results of these and an earlier series of tests showed that the existing data on the fire resistance of prestressed concrete beams of rectangular section were adequate and that beams could be designed from the current recommendations to provide a specific degree of protection. The existing data did not adequately deal with the design of 'I' section beams which require supplementary reinforcement in the web to protect the beams against failure due to the possible early fracture of the web. In addition, the shape of the lower flange is such that greater transfer of heat is likely to the tendons than would be the case with a rectangular beam of the same width. This could necessitate an increase in the thickness of the concrete cover for beams of 'I' section.

As previously described, reinforced concrete beams when made with gravel aggregate concrete must be so designed that spalling of concrete from the soffit and sides is prevented. It would seem that when concrete cover exceeds 40 mm (1.6 in) there is a need for supplementary reinforcement which should possess substantial strength to retain loosened pieces of concrete in place. Even with a concrete cover of only 40 mm (1.6 in), reducing the severity of spalling can give a marked increase in fire resistance as was the case with specimen No.17; the actual fire resistance was about double that for which it had been designed.

The superior performance of the reinforced concrete beams with gravel aggregate concrete when measures were taken to retain the covering position may have also been due to the fact that only the corner bars of the reinforcement had the minimum thickness of cover, the remainder being better protected owing to their position nearer the centre of the section. This indicates that the nett effective cover was in fact, greater than the values shown in the tables and used as a basis for the design.

The concrete encased steel beams with supplementary reinforcement gave at least 26 per cent superior performance than they were designed for and it would seem reasonable to assume that a reduction in cover of 20 per cent could be made, that is for a fire resistance of 4 h a cover of 50 mm (2 in) instead of the 63 mm ($2\frac{1}{2}$ in) would have been adequate.

6. Effect of continuity

The type of end condition investigated for continuity represented structurally a very simple situation where a beam spans more than one bay in a building but has no longitudinal or angular restraint to movement. In practice, this type of situation is unlikely to be present very often as some end restraint by the surrounding elements will usually be available. However, for the purposes of this investigation it was considered useful to establish first of all, whether the existence of pure moments over the supports would have any influence on the behaviour of beams under fire conditions.

The tests results have shown that the method of continuity employed had only a marginal effect on the overall performance of beams although the mode of failure was different.

With the two prestressed concrete beams there was a clear indication of the formation of three hinges, one at the centre of the span and one each close to the point of contraflexure. The design and the size of these beams was such that the hinges were formed more or less simultaneously. The failure of the reinforcement occurred in tension at the centre of the specimen and that of concrete in compression close to the supports.

The results on the reinforced concrete beams gave no information on this aspect of testing owing to the spalling of the concrete cover.

The encased steel beam specimens gave almost the same performance in both cases and owing to the substantial contribution of the steel section the beam was able to resist compressive stresses at the points of contraflexure, therefore only one hinge was formed at the centre of the span.

The results have shown that when the use of the beams is such that conditions of 'pure' continuity or cantilevered ends occurs without any restraint to thermal movement the nett performance is unlikely to be much different from that of a simply supported element.

CONCLUSIONS

The results of the investigation on the performance of structural concrete beams described in this report permit the following conclusions to be drawn:

1. Spalling of gravel aggregate concrete is likely to have serious effects on the performance of beams and when the thickness of concrete cover exceeds 40 mm (1.6 in) a supplementary reinforcement should be included in the concrete cover. This reinforcement may consist of a system of links at 150 mm (6 in) centres or a steel wire fabric of a mesh not greater than 150 mm (6 in). Even with concrete covers of 40 mm (1.6 in) or less, supplementary reinforcement can show gains in the performance of beams.

2. Lightweight aggregate concrete is able to retain its integrity under fire conditions and does not require to be provided with supplementary reinforcement, even when the cover is 63 mm ($2\frac{1}{2}$ in) thick.

3. Lightweight aggregate concrete provides better insulation to the reinforcement and its use can result in at least 40 per cent improvement in fire resistance in comparison with dense aggregate concretes.

4. Cold worked twisted steel reinforcement gave a slightly lower performance than hot rolled alloy steel which, in its turn, showed slightly increased deflections in comparison with mild steel. However, for reasons mentioned in the report a more precise relationship could not be established between various steels. The indications are that the differences may not be greater than about 10 per cent.

5. Provision of 'pure' continuity without any restraint has only a marginal effect on the overall performance of beams, although with certain types it may alter the mode of failure.

TABLE 4

Summary of test results on 7.6 m prestressed concrete beams

Ref.No.	Beam type	Age (months)	Nominal concrete cover (mm)	Test load (tons)	Designed[@] fire resistance (h)	Duration of test (h - min)	Steel temp. in °C: mean	Steel temp. in °C: max	Maximum deflection (mm)	Time to reach critical deflection 1/30 (h - min)
1[+]	R/PO/W/SR	28	100	21.0	4	4 - 15	395	-	230	4 - 16
2	R/PO/W/SR	29	100	21.0	4	4 - 10	400	-	230	4 - 11
3	I/PR/W/X	35	50	10.9	1.5	0 - 32	80	112	Collapse	
4	I/PR/S/SR	26	50	10.9	1.5	1 - 35	410	445	210	1 - 38*
5	I/PR/S/X(VG)	33	50	10.9	2.5	3 - 16	350	420	230	3 - 15

R - rectangular
I - I-section
S - Strands
SR - Supplementary reinforcement
X - No " "
VG - Vermiculite/gypsum plaster

PO - Post tensioned
PR - Pretensioned
W - Wire tendons

+Roller supports
@In accordance with Codes of Practice
*Estimated

TABLE 5

Summary of test results on 7.6 m reinforced concrete beams

Ref.No.	Beam type	Age	Nominal concrete cover	Test load	Designed[e] fire resistance	Duration of test		Steel temp.		Maximum Deflection	Time to reach critical deflection l/30
		(months)	(mm)	(tons)	h	h	min	mean	max	(mm)	h - min
8	DG/MS/X	34	63	18.5	4	1	38	492	863	230	+
9	DG/CD/X	36	63	17.0	4	1	35	455	575	220	+
10	DG/CT/X	26	63	17.0	4	1	37	520	850	235	+
11	DG/HR/X	26	63	17.0	4	1	39	500*	800*	230	+
12	LC/MS/X	34	63	18.5	4	6	00	545	625	165	6 - 40
13	LS/MS/X	32	63	18.5	4	6	00	490	630	140	6 - 56
14	DG/MS/SR	18	63	18.5	4	2	51	495	800*	290	2 - 42
15	DG/HR/SR	18	63	17.0	4	4	55	635	700	290	4 - 48
16	DG/CT/SR	18	63	17.0	4	4	35	560	645	290	4 - 30
17	DG/HR/SR	18	38	18.4	1.5	4	24	640	985	270	4 - 16
18	DG/HR/X	18	38	18.4	1.5	2	41	600*	780	250	2 - 40
19	DG/HR/X	18	25	19.0	1	2	44	600	750	255	2 - 38

DG - Dense gravel concrete
LC - Lightweight expanded clay concrete
LS - Lightweight foamed slag concrete
MS - Mild steel
CD - Cold worked deformed steel
CT - Cold worked twisted steel

HR - Hot rolled alloy steel
SR - Supplementary reinforcement
X - No " "

*Estimated
+Not calculated owing to failure of beams
[e]In accordance with Codes of Practice

TABLE 7

Summary of test results on 11.3 m beams with cantilevered ends

Ref. No.	Beam type	Age (months)	Nominal concrete cover (mm)	Test load (tons) Centre span	Test load (tons) Ends	Designed[@] fire resistance h	Duration of test h	Duration of test min	Steel temp. mean	Steel temp. max	Maximum deflection (mm)	Time to reach critical deflection 1/30 h	-	min
6	Prestressed concrete	37	100	23.6	5.9	4	4	26	645	675	Collapse	4	-	24
7	"	36	100	20.8	5.2	4	4	43	440	465	"			
20	Reinforced concrete	40	63	37.4	9.35	4	1	36	495	760	265		+	
21	"	38	63	35.8	8.95	4	1	37	500	710	275		+	
24	Encased steel	42	63	38.6	9.65	4	5	15	705 (lower flange)		230	5	-	16

+Not calculated owing to premature failure of specimen before reaching critical deflection

@In accordance with Codes of Practice.

TABLE 6

Summary of test results on 7.6 m encased steel beams

Ref.No.	Beam type	Age (mnths)	Nominal concrete cover (mm)	Test load (tons)	Designed[e] fire resistance h	Duration of test h	min	Steel temp. lower flange (°C)	Maximum Deflection (mm)	Time to reach critical deflection 1/30 h - min
22	DG/MS/SR	18	63	18.5	4	5	39	740(at 5h)	230	5 - 40
23	DG/MR/SR	26	63	26.9	4	5	00	700(at 5h)	210	5 - 12

DG - Dense gravel concrete
MS - Mild steel
HR - Hot rolled alloy steel
SR - Supplementary reinforcement

[e]In accordance with Codes of Practice

APPENDIX

Log of tests

Specimen No.1.

Beam:	Prestressed concrete, post-tensioned, gravel aggregate concrete.
Reinforcement:	5 mm (0.2 in) dia. high tensile tendons.
Concrete cover:	Soffit 100 mm (4 in), sides 115 mm (4½ in)
Supplementary reinforcement:	5 mm (3/16 in) dia. links at 152 mm (6 in) centres.
Length:	7.6 m (25 ft), span 7.3 m (24 ft), simply supported, roller end supports.
Test load:	21.0 tons

Time h	Time min	Observation
0	30	Fall of some concrete from the arrisses
1	00	Concrete spalled off from both arrisses for nearly whole length, deflection 57 mm (2.2 in).
2	00	Deflection 50 mm (2.3 in).
3	00	Deflection 75 mm (3 in).
4	00	Deflection 165 mm (6.5 in) rapidly increasing.
4	15	Deflection 230 mm (9.0 in), test stopped, beam has not collapsed.

At 4 h 15 min mean reinforcement temperature 395°C

Specimen No.2.

Beam:	Prestressed concrete, post-tensioned, gravel aggregate concrete.
Reinforcement:	5 mm (0.2 in) dia. high tensile tendons.
Concrete cover:	Soffit 100 mm (4 in), sides 115 mm (4½ in).
Supplementary reinforcement:	5 mm (3/16 in) dia. links at 152 mm (6 in) centres.
Length:	7.6 m (25 ft), span 7.3 m (24 ft), simply supported, plate end supports.
Test load:	21.0 tons

Time h	min	Observation
0	30	Fall of a concrete piece from arris
1	00	Most of the arrises have suffered fall of concrete, deflection 35 mm (1.5 in).
2	00	Deflection 32 mm (1.6 in).
3	00	Some cracks on beam faces, deflection 50 mm (1.9 in).
4	00	Deflection 170 mm (6.7 in) rapidly increasing.
4	10	Deflection 230 mm (9.0 in), test stopped, beam not collapsed.

At 4 h 10 min mean reinforcement temperature 400°C

Specimen No.3.

Beam:	Prestressed concrete, pre-tensioned, I-section.
Reinforcement:	Twenty 5 mm (0.2 in) dia. tendons.
Concrete cover:	Soffit 50 mm (2 in), sides 50 mm (2 in).
Supplementary reinforcement:	None.
Length:	7.6 m (25 ft), span 7.3 m (24 ft) simply supported.
Test load:	10.9 tons.

Time h	min	Observation
0	25	Appearance of cracks in the beam, deflection 110 mm (4.3 in).
0	30	An almost continuous longitudinal crack in the web, deflection 175 mm (6.8 in).
0	32	Collapse of beams, diagonal shear cracks near the supports, fall of concrete from soffit, deflection 205 mm (8.0 in). Test stopped.

At 32 min maximum reinforcement temperature 112°C
mean " " 80°C

Specimen No.4.

Beam:	Prestressed concrete, pre-tensioned, I-section.
Reinforcement:	Four 50 mm ($\frac{1}{2}$ in) dia. strands.
Concrete cover:	Soffit 70 mm ($2\frac{3}{4}$ in), sides 70 mm ($2\frac{3}{4}$ in).
Supplementary reinforcement:	5 mm (3/16 in) dia. links.
Length:	7.6 m (25 ft), span 7.3 m (24 ft), simply supported.
Test load:	10.9 tons.

Time h	min	Observation
0	30	No spalling of concrete, deflection 40 mm (1.6 in).
1	00	Appearance of cracks in the web, deflection 65 mm (2.6 in).
1	20	Cracks along the web, deflection 130 mm (5.1 in).
1	35	Deflection, rapidly increased to 210 mm (8.3 in), spalling of concrete from arris has taken place. Test stopped.

At 1 h 35 min maximum reinforcement temperature 445°C
mean " " 410°C

Specimen No.5.

Beam: Prestressed concrete, pre-tensioned, I-section. 13 mm ($\frac{1}{2}$ in) vermiculite/gypsum plaster coating on exposed faces.

Reinforcement: Four 50 mm ($\frac{1}{2}$ in) strands.

Concrete cover: Soffit 70 mm ($2\frac{3}{4}$ in), sides 70 mm ($2\frac{3}{4}$ in).

Supplementary reinforcement: None.

Length: 7.6 m (25 ft), span 7.3 (24 ft), simply supported.

Test load: 10.9 tons.

Time h	min	Observation
0	25	Some finish coat of plaster from soffit detached.
0	45	Some fall of plaster from sides.
1	00	Part of arris exposed, deflection 20 mm (0.8 in).
2	00	Cracks in arrisses, deflection 48 mm (1.8 in).
2	45	Fall of a piece of concrete from arris, deflection 95 mm (3.7 in).
3	00	Deflection 140 mm (5.5 in).
3	15	Deflection 230 mm (9.0 in) and rapidly increasing. Test stopped. Beam not collapsed.

At 3 h 15 min maximum reinforcement temperature 420°C
mean " " 350°C

Specimen No.6.

Beam: Prestressed concrete, post-tensioned, gravel aggregate concrete.

Reinforcement: Seven 50 mm ($\frac{1}{2}$ in) dia. strands in single duct.

Concrete cover: Soffit 100 mm (4 in), sides 115 mm ($4\frac{1}{2}$ in) (central 3.3 m (11 ft) of span)

Supplementary reinforcement: 5 mm ($^{3}/16$ in) links at 152 mm (6 in) centres.

Length: 11.3 m (37 ft), cantilevered ends, providing continuity.

Test load: Central span 23.6 tons, cantilevered ends 5.9 tons.

Time (h)	Time (min)	Observation
0	50	Spalling of concrete from the arrisses.
1	30	Further spalling from arrisses, deflection 15 mm (0.6 in).
2	00	Cracks in slab over supports, deflection 15 mm (0.6 in)
3	00	Deflection 15 mm (0.6 in).
3	40	Diagonal cracks on beam faces close to supports.
4	00	Horizontal cracks along duct or beam faces, deflection 35 mm (1.4 in).
4	25	Widening of cracks or beam faces, deflecting rapidly increased to 230 mm (9.0 in).
4	26	Collapse of beam ends to floor followed by break-up in middle. Deflection 255 mm (10 in) before collapse.

Reinforcement temperature (break-up of some thermocouples at 1-45) Maximum 675°C
Mean 445°C

Specimen No. 7.

Beam:	Prestressed concrete, post-tensioned, gravel aggregate concrete.
Reinforcement:	2 cables.
Concrete cover:	Soffit 100 mm (4 in), sides 110 mm (4 5/16 in) (central 1.8 m (6 ft) of span).
Supplementary reinforcement:	5 mm (3/16 in) links at 152 mm (6 in) centres.
Length:	11.3 m (37 ft), span 7.3 m (24 ft) cantilevered ends providing continuity.
Test load:	Central span 20.8 tons, cantilevered ends 5.2 tons.

Time h	Time min	Observation
1	00	Spalling of concrete from arrisses, beam deflection at centre decreasing.
2	00	Beam central deflection reduced to zero.
3	00	Beam commences to deflect again.
4	00	Central deflection 12 mm (0.5 in) horizontal cracks or beam faces.
4	40	Collapse of one cantilevered end, central deflection 68 mm (2.7 in).
4	43	Collapse of the other cantilevered end, collapse in the middle.

Reinforcement temperature (lower cable) 4 h 40 min
Maximum 465°C
Mean 440°C

Specimen No.8

Beam:	Reinforced concrete, gravel aggregate.
Reinforcement:	Mild steel bars 6 off, 28 mm ($1\frac{1}{8}$ in) diameter.
Concrete cover:	Soffit 63 mm ($2\frac{1}{2}$ in), sides 63 mm ($2\frac{1}{2}$ in).
Length:	7.6 m (25 ft), span 7.3 m (24 ft) simply supported.
Test load:	18.5 tons.

Time h	Time min	Observation
0	15	Slight spalling of concrete from arrisses.
0	30	Further spalling of concrete.
0	40	Concrete from whole of soffit spalled.
0	45	Lower reinforcement exposed.
0	55	Spalling of concrete from the sides
1	00	Central deflection 100 mm (4.0 in).
1	15	Further spalling from sides, deflection 140 mm (5.5 in).
1	30	Crack on the side of beam, rapid increase of deflection to 200 mm (8.0 in).
1	38	Test stopped. Maximum deflection 230 mm (9.2 in) and increasing rapidly. Beam had not completely collapsed at end of test.

At 1 h 38 min maximum reinforcement temperature 863°C
mean " " 492°C

Specimen No.9

Beam:	Reinforced concrete, gravel aggregate.
Reinforcement:	Cold worked steel deformed (ribbed) bars 6 off, 22 mm ($\frac{7}{8}$ in) dia.
Concrete cover:	Soffit 63 mm ($2\frac{1}{2}$ in), sides 63 mm ($2\frac{1}{2}$ in).
Supplementary reinforcement:	None.
Length:	7.6 m (25 ft), span 7.3 m (24 ft) simply supported.
Test load:	17.0 tons.

Time h	Time min	Observations
0	20	Slight spalling of concrete from one arris.
0	30	Extensive spalling of concrete from soffit and arrisses.
0	50	Spalling continues, steel reinforcement exposed.
1	00	Further spalling, central deflection 63 mm (2.5 in)
1	20	Deflection 130 mm (5.2 in).
1	35	Test stopped. Deflection 220 mm (8.8 in) and rapdily increasing. Beam had not collapsed.

At 1 h 35 min maximum reinforcement temperature 575°C
mean " " 455°C

Note: Some of the thermocouples were destroyed at 65 minutes when the maximum temperature was 680°C, therefore the final readings may not be correct.

Specimen No.10.

Beam:	Reinforced concrete, gravel aggregate.
Reinforcement:	Cold worked steel, square twisted bars, 6 off, 22 mm ($\frac{7}{8}$ in).
Concrete cover:	Soffit 63 mm ($2\frac{1}{2}$ in), sides 63 mm ($2\frac{1}{2}$ in).
Supplementary reinforcement:	None.
Length:	7.6 m (25 ft), span 7.3 m (24 ft) simply supported.
Test load:	17.0 tons

Time h	min	Observation
0	34	Spalling of concrete from arrisses.
0	50	Most of concrete from soffit spalled away.
1	00	Deflection 70 mm (2.8 in) reinforcement exposed.
1	10	Cracks in beam faces.
1	30	Deflection 157 mm (6.2 in).
1	37	Deflection 235 mm (9.2 in) and rapidly increasing. Test stopped.

Beam had not collapsed at end of test.

At 1 h 35 min maximum reinforcement temperature 850°C
mean " " 520°C

Specimen No.11.

Beam:	Reinforced concrete, gravel aggregate.
Reinforcement:	Hot rolled alloy steel, 6 off, 22 mm ($\frac{7}{8}$ in) dia.
Concrete cover:	Soffit 63 mm ($2\frac{1}{2}$ in), sides 63 mm ($2\frac{1}{2}$ in).
Supplementary reinforcement:	None.
Length:	7.6 m (25 ft), span 7.3 m (24 ft) simply supported.
Test load:	17.0 tons.

Time h	min	Observation
0	36	Spalling of concrete from arrisses.
0	40	Extensive spalling; lower reinforcement exposed.
1	00	Spalling of concrete at sides, deflection 81 mm (3.2).
1	15	Deflection 120 mm (4.7 in).
1	39	Soffit extensively damaged, deflection 230 mm (9 in) and rapidly increasing. Test stopped.

Beam had not collapsed at end of test.

At 1 h 39 min (estimated) maximum reinforcement temperature 800°C
mean " " 500°C

Specimen No.12.

Beam:	Reinforced concrete, expanded clay aggregate.
Reinforcement:	Mild steel bars, 6 off, 28 mm ($1\frac{1}{8}$ in) dia.
Concrete cover:	Soffit 63 mm ($2\frac{1}{2}$ in), sides 63 mm ($2\frac{1}{2}$ in).
Supplementary reinforcement:	None.
Length:	7.6 m (25 ft), span 7.3 m (24 ft) simply supported.
Test load:	18.5 tons.

Time h	min	Observations
0	30	No change in appearance of beam.
1	00	Deflection 12 mm (0.5 in), no other change.
2	00	Deflection 12 mm (0.5 in).
2	50	Fine vertical cracks on sides of beams.
3	00	Deflection 35 mm (1.4 in).
4	00	Deflection 73 mm (2.9 in).
5	00	Deflection 110 mm (4.3 in).
6	00	Deflection 165 mm (6.5 in). Test terminated.

With the exception of surface cracking no other change in the appearance of the beam.

At 6 h maximum reinforcement temperature 625°C
mean " " 545°C

Specimen No.13.

Beam:	Reinforced concrete, foamed slag aggregate.
Reinforcement:	Mild steel bars, 6 off, 28 mm ($1\frac{1}{8}$ in) dia.
Concrete cover:	Soffit 63 mm ($2\frac{1}{2}$ in), sides 63 mm ($2\frac{1}{2}$ in).
Supplementary reinforcement:	None.
Length:	7.6 m (25 ft), span 7.3 m (24 ft) simply supported.
Test load:	18.5 tons.

Time h	min	Observations
1	00	Deflection 12 mm (0.5 in), no change in appearance.
2	00	Deflection 15 mm (0.6 in).
3	00	Deflection 30 mm (1.2 in).
4	00	Deflection 73 mm (2.9 in).
5	00	Deflection 100 mm (3.9 in).
6	00	Deflection 140 m (5.6 in). Test stopped.

With the exception of surface cracking, no other change in the appearance of the beam.

At 6 h maximum reinforcement temperature 630°C
mean " " 490°C

Specimen No.14.

Beam:	Reinforced concrete, gravel aggregate.
Reinforcement:	Mild steel bars, 6 off, 28 mm ($1\frac{1}{8}$ in) diameter.
Concrete cover:	Soffit 63 mm ($2\frac{1}{2}$ in), sides 63 mm ($2\frac{1}{2}$ in).
Supplementary reinforcement:	Expanded metal lath 125 mm x 75 mm (5 in x 3 in) mesh.
Length:	7.6 m (25 ft), span 7.3 m (24 ft) simply supported.
Test load:	18.5 tons.

Time h	min	Observations
1	00	Appearance of cracks along arrisses.
1	20	Fall of concrete from arrisses, deflection 10 mm (0.4 in).
1	30	Further falls of concrete from soffit, exposing reinforcement.
1	45	Concrete continues to spall, mesh not able to retain it in position.
2	00	Deflection 90 mm (3.6 in).
2	15	75 per cent of lower reinforcement exposed.
2	30	Deflection 170 mm (6.7 in) and rapidly increasing.
2	50	Deflection 290 mm (11.5 in). Test stopped.

Beam badly damaged but did not collapse.

At 2 h 15 min maximum reinforcement temperature 800°C
mean " " 495°C

(Damage sustained by some thermocouples resulted in lower mean temperature at 2 h 50 min).

Specimen No.15.

Beam:	Reinforced concrete, gravel aggregate.
Reinforcement:	Hot rolled alloy steel, 6 off, 22 mm ($\frac{7}{8}$ in) dia.
Concrete cover:	Soffit 63 mm ($2\frac{1}{2}$ in), sides 63 mm ($2\frac{1}{2}$ in).
Supplementary reinforcement:	Wire fabric 150 mm x 100 mm x 3 mm (6 in x 4 in x 12 B.G) dia.
Length:	7.6 m (25 ft), span 7.3 m (24 ft) simply supported.
Test load:	17 tons.

Time h	min	Observation
1	05	Cracking of concrete along arrisses, fall of some concrete.
2	00	Concrete from most of arrisses spalled away, deflection 28 mm (1.1 in).
3	00	Slight further fall from arrisses but soffit intact. Deflection 60 mm (2.4 in).
4	00	Deflection 110 mm (4.3 in).
4	50	Deflection 245 mm (9.7 in) and rapidly increasing.
4	55	Deflection 290 mm (11.5 in). Test stopped, beam resting on pillar.

Maximum reinforcement temperature 700°C (estimated)
mean " " 635°C

Specimen No.16.

Beam:	Reinforced concrete, gravel aggregate.
Reinforcement:	Cold worked steel twisted bars, 6 off, 22 mm ($\frac{7}{8}$ in) dia.
Concrete cover:	Soffit 63 mm ($2\frac{1}{2}$ in), sides 63 mm ($2\frac{1}{2}$ in).
Supplementary reinforcement:	Steel wire fabric 150 mm x 100 mm x 3 mm (6 in x 4 in x 12 B.G) dia
Length:	7.6 m (25 ft), span 7.3 (24 ft) simply supported.
Test load:	17.0 tons.

Time h	min	Observations
0	45	Cracks along arrisses, fall of a small piece of concrete.
1	50	Fall of concrete from arrisses, deflection 25 mm (0.95 in).
3	00	Most of concrete from arrisses fallen way, deflection 53 mm (2.1 in).
4	00	No significant change, deflection 112 mm (4.4 in).
4	30	Severe surface cracking but reinforcement not exposed, deflection 215 mm (8.5 in) and rapidly increasing.
4	35	Deflection 290 mm (11.5 in). Test stopped.

Maximum reinforcement temperature 645°C
mean " " 560°C

Specimen No.17.

Beam:	Reinforced concrete, gravel aggregate.
Reinforcement:	Hot rolled alloy steel, 6 off, 22 mm ($\frac{7}{8}$ in) dia.
Concrete cover:	Soffit 38 mm ($1\frac{1}{2}$ in) sides 38 mm ($1\frac{1}{2}$ in).
Supplementary reinforcement:	Steel wire fabric 150 mm x 100 mm x 3 mm (6 in x 4 in x 12 B.G) dia.
Length:	7.6 m (25 ft), span 7.3 m (24 ft) simply supported.
Test load:	18.4 tons.

Time h	min	Observations
0	30	Cracks along arrisses.
1	00	Spalling of concrete along arrisses.
2	00	Deflection 50 mm (2.0 in).
2	55	Some further spalling from arrisses, deflection 82 mm (3.25 in).
4	00	Some more spalling exposing small length of a bar, deflection 155 mm (6.20 in).
4	20	Deflection 270 mm (10.7 in) and rapidly increasing.
4	24	Test stopped.

Maximum reinforcement temperature 985°C
mean " " 640°C

Specimen No.18.

Beam: Reinforced concrete, gravel aggregate concrete.

Reinforcement: Hot rolled alloy steel, 6 off, 22 mm ($\frac{7}{8}$ in) dia.

Concrete cover: Soffit 38 mm ($1\frac{1}{2}$ in), sides 38 mm ($1\frac{1}{2}$ in).

Supplementary reinforcement: None.

Length: 7.6 m (25 ft), span 7.3 m (24 ft) simply supported.

Test load: 18.4 tons

Time h	min	Observation
0	36	Fall of about 1.8 m (6 ft) length of concrete piece from an arris.
0	42	Fall of concrete from the other arris.
0	58	Fall of concrete from soffit, deflection 50 mm (2.0 in).
2	00	Reinforcement becoming exposed from soffit, deflection 110 mm (4.4 in).
2	25	Cracks in sides, increasing in width.
2	30	Spalling of concrete from whole of soffit, deflection 170 mm (6.7 in) and rapidly increasing.
2	41	Deflection 290 mm (9.8 in). Test stopped.

At 2 h 40 min maximum reinforcement temperature 780°C
mean " " 600°C (estimated)

Specimen No.19.

Beam: Reinforced concrete, gravel aggregate.

Reinforcement: Hot rolled alloy steel, 6 off, 22 mm ($\frac{7}{8}$ in) dia.

Concrete cover: Soffit 25 mm (1 in), sides 25 mm (1 in).

Supplementary reinforcement: None.

Length: 7.6 m (25 ft), span 7.3 m (24 ft) simply supported.

Test load: 19.0 tons

Time h	min	Observation
0	33	Fall of a piece of concrete from an arris.
0	42	Further fall of concrete from arris.
1	00	Both arrisses now damaged, deflection 65 mm (2.6 in).
1	40	Fall of concrete from the soffit, some reinforcement exposed.
2	00	Deflection 115 mm (4.5 in).
2	17	Exposure of further reinforcement.
2	30	Deflection 178 mm (7.0 in) and rapidly increasing.
2	44	Deflection 255 mm (10.0 in). Test stopped.

At 2 h 45 min maximum reinforcement temperature 750°C (estimated)
mean " " 600°C

Specimen No.20.

Beam:	Reinforced concrete, gravel aggregate concrete.
Reinforcement:	Mild steel bars, 6 off, 28 mm ($1\frac{1}{8}$ in) dia.
Concrete cover:	Soffit 63 mm ($2\frac{1}{2}$ in), sides 63 mm ($2\frac{1}{2}$ in).
Supplementary reinforcement:	None.
Length:	11.3 m (37 ft), span 7.3 m (24 ft) cantilevered ends providing continuity.
Test load:	Central span 37.4 tons, cantilevered ends 9.35 tons.

Time h min	Observation
0 42	Fall of a piece of concrete from an arris, horizontal cracks on beam face.
0 45	Spalling of concrete from arrisses and soffit exposing reinforcement.
1 00	Spalling of concrete from beam faces near the supports, deflection 115 mm (4.5 in).
1 07	Severe spalling of concrete near support.
1 20	Central deflection 180 mm (7.1 in) and rapidly increasing.
1 36	Central deflection now 265 mm (10.5 in) cantilevered ends deflected upward about 178 mm (7 in).
	Test stopped, beam had not collapsed.

At 1 h 35 min maximum reinforcement temperature 760°C
mean " " 495°C

Specimen No.21.

Beam:	Reinforced concrete, gravel aggregate concrete.
Reinforcement:	Cold worked steel ribbed bars, 6 off, 22 mm ($\frac{7}{8}$ in) dia.
Concrete cover:	Soffit 63 mm ($2\frac{1}{2}$ in), sides 63 mm ($2\frac{1}{2}$ in).
Supplementary reinforcement:	None.
Length:	11.3 m (37 ft), span 7.3 m (24 ft) cantilevered ends providing continuity.
Test load:	Central span 35.8 tons, cantilevered ends 9.35 tons.

Time h	min	Observation
0	35	Extensive spalling of concrete from the arrisses, exposing steel reinforcement.
0	45	Some spalling from the beam face.
1	00	Deflection 65 mm (2.6 in).
1	10	Spalling of concrete near the supports exposing continuity steel.
1	30	Deflection 190 mm (7.4 in) and rapidly increasing.
1	37	Deflection 275 mm (11.0 in). Test stopped, beam not collapsed.

At 1 h 35 min maximum reinforcement temperature 710°C
mean " " 500°C

Specimen No.22.

Beam:	Concrete encased steel, gravel aggregate concrete.
Steel section:	152 mm x 405 mm x 74.5 kg(6 in x 16 in x 50 lb/ft) mild steel beam.
Concrete cover:	Soffit 63 mm ($2\frac{1}{2}$ in), sides 63 mm ($2\frac{1}{2}$ in).
Supplementary reinforcement:	5 mm (3/16 in) dia. links at 150 mm (6 in) centres.
Length:	7.6 m (25 ft), span 7.3 m (24 ft) simply supported.
Test load:	18.5 tons.

Time h	min	Observation
0	32	Spalling of concrete from an arris.
1	00	Some further spalling restricted to the arrisses, deflection 15 mm (0.6 in).
2	00	Deflection 28 mm (1.1 in).
3	00	Deflection 58 mm (2.3 in).
4	00	Deflection 93 mm (3.7 in).
5	00	Deflection 158 mm (6.6 in).
5	30	Deflection 220 mm (8.6 in).
5	39	Deflection 230 mm (9.1 in) and rapidly increasing. Test stopped, beam had not collapsed.

Steel temperature lower flange, maximum 740°C at 5 h.

Specimen No.23.

Beam:	Concrete encased steel, gravel aggregate concrete.
Steel section:	152 mm x 405 mm x 74.5 kg (6 in x 16 in x 50 lb) hot rolled alloy steel beam.
Concrete cover:	Soffit 63 mm ($2\frac{1}{2}$ in), sides 63 mm ($2\frac{1}{2}$ in).
Supplementary reinforcement:	5 mm (3/16 in) dia. links at 150 mm (6 in) centres.
Length:	7.6 m (25 ft), span 7.3 m (24 ft) simply supported.
Test load:	26.9 tons.

Time h	min	Observation
0	35	Some spalling of concrete from arrisses.
1	00	Most of arrisses suffered damage, supplementary reinforcement exposed, deflection 6 mm (0.5 in).
2	00	Deflection 30 mm (1.2 in).
3	00	Deflection 65 mm (2.6 in).
4	00	Some further spalling of concrete from arrisses, deflection 125 mm (4.9 in).
5	00	Deflection 210 mm (8.3 in) and rapidly increasing. Test stopped.

Steel temperature, lower flange 700°C

Specimen No.24.

Beam:	Concrete encased steel, gravel aggregate concrete.
Steel section:	152 mm x 405 mm x 74.5 kg (6 in x 16 in x 50 lb) mild steel beam.
Concrete cover:	Soffit 63 mm ($2\frac{1}{2}$ in). dia, links at 150 mm (6 in) centres.
Supplementary reinforcement:	5 mm (3/16 in) dia. links at 150 mm (6 in) centres.
Length:	11.3 m (37 ft), span 7.3 m (24 ft), cantilevered ends providing continuity.
Test load:	Central span 38.6 tons, cantilevered ends 9.65 tons.

Time h	min	Observation
0	25	Spalling of concrete from the arrisses.
0	30	All arrisses suffered damage by spalling.
0	40	Spalling on beam faces exposing supplementary reinforcement.
1	00	Deflection 23 mm (0.9 in).
2	00	Deflection 33 mm (1.3 in).
3	00	Deflection 70 mm (2.8 in).
4	00	Deflection 120 mm (4.8 in).
5	00	Deflection 200 mm (7.8 in).
5	15	Deflection 230 mm (9.0 in) rapidly increasing. Test stopped, beam had not failed.

Steel temperature, lower flange 700°C at 5 h.

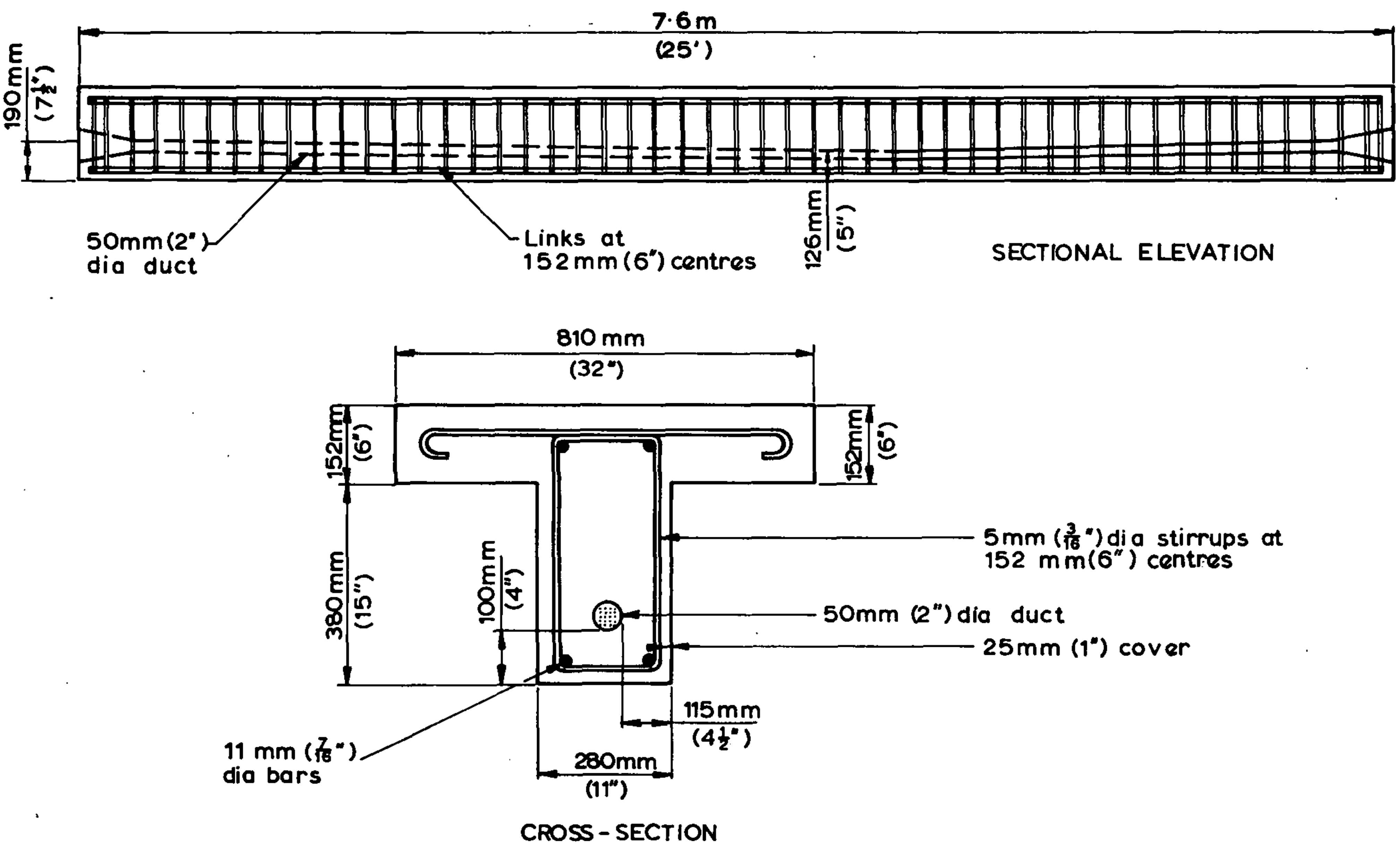
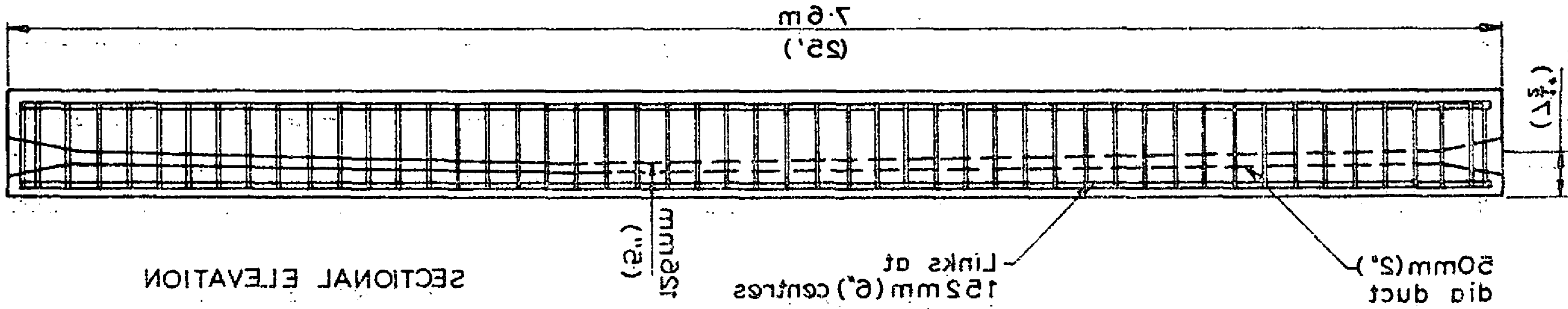
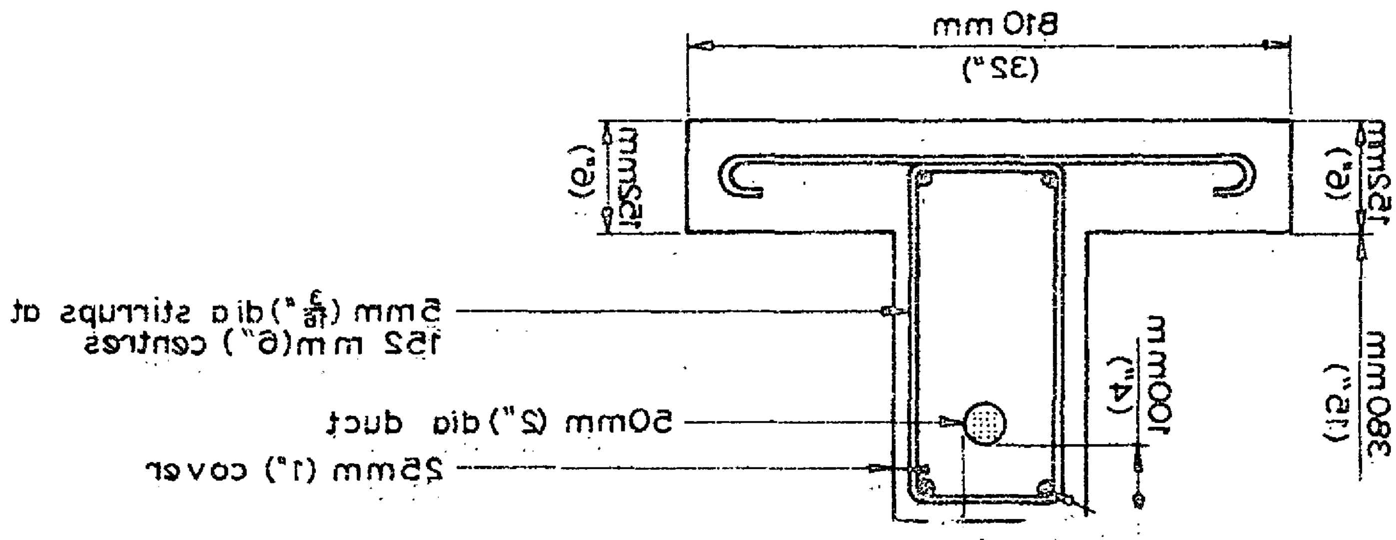


FIG. 1. DETAILS OF 7·6m (25') PRESTRESSED CONCRETE BEAMS



SECTIONAL ELEVATION



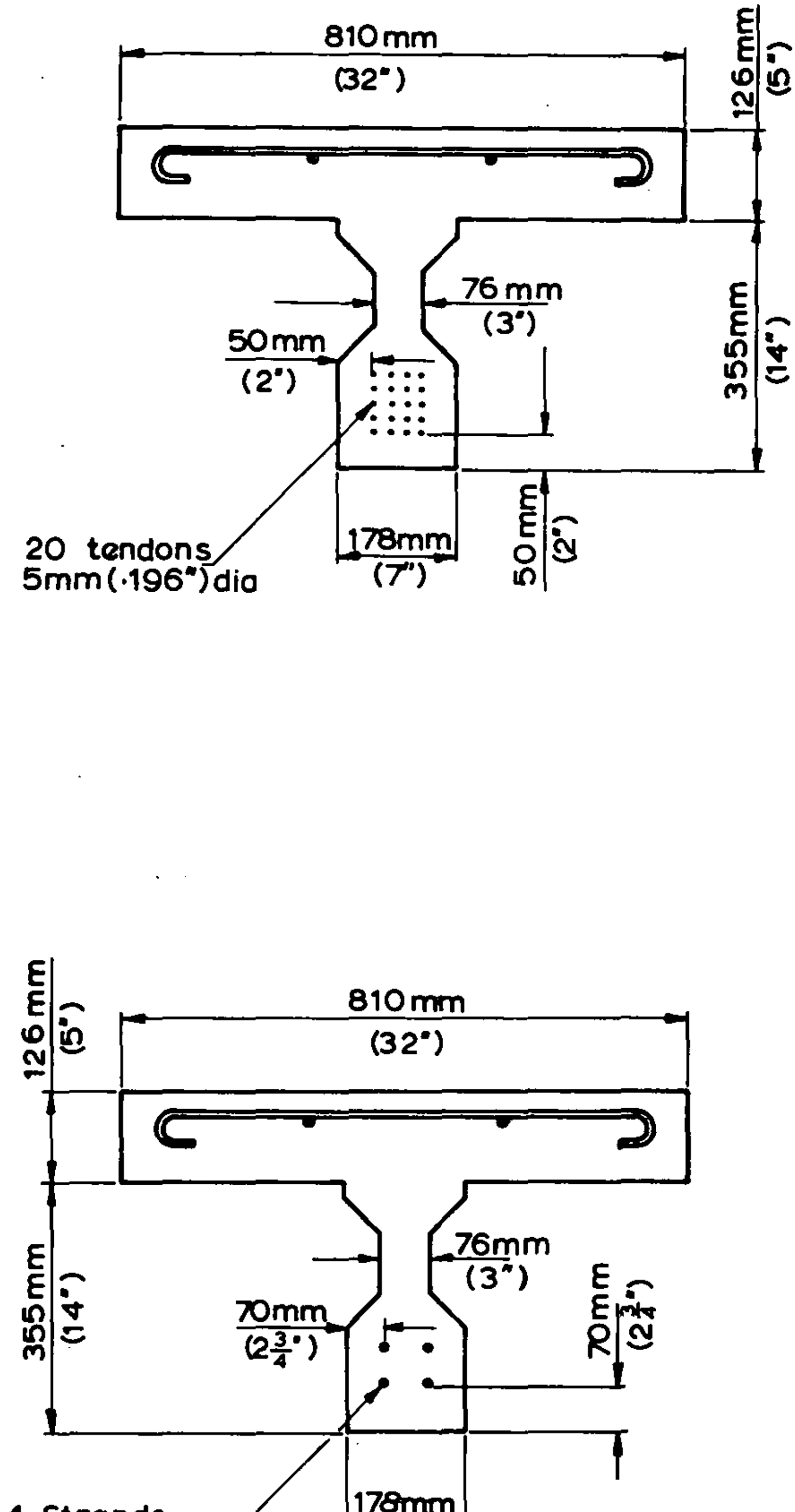


FIG. 2. PRESTRESSED I-SECTION BEAMS

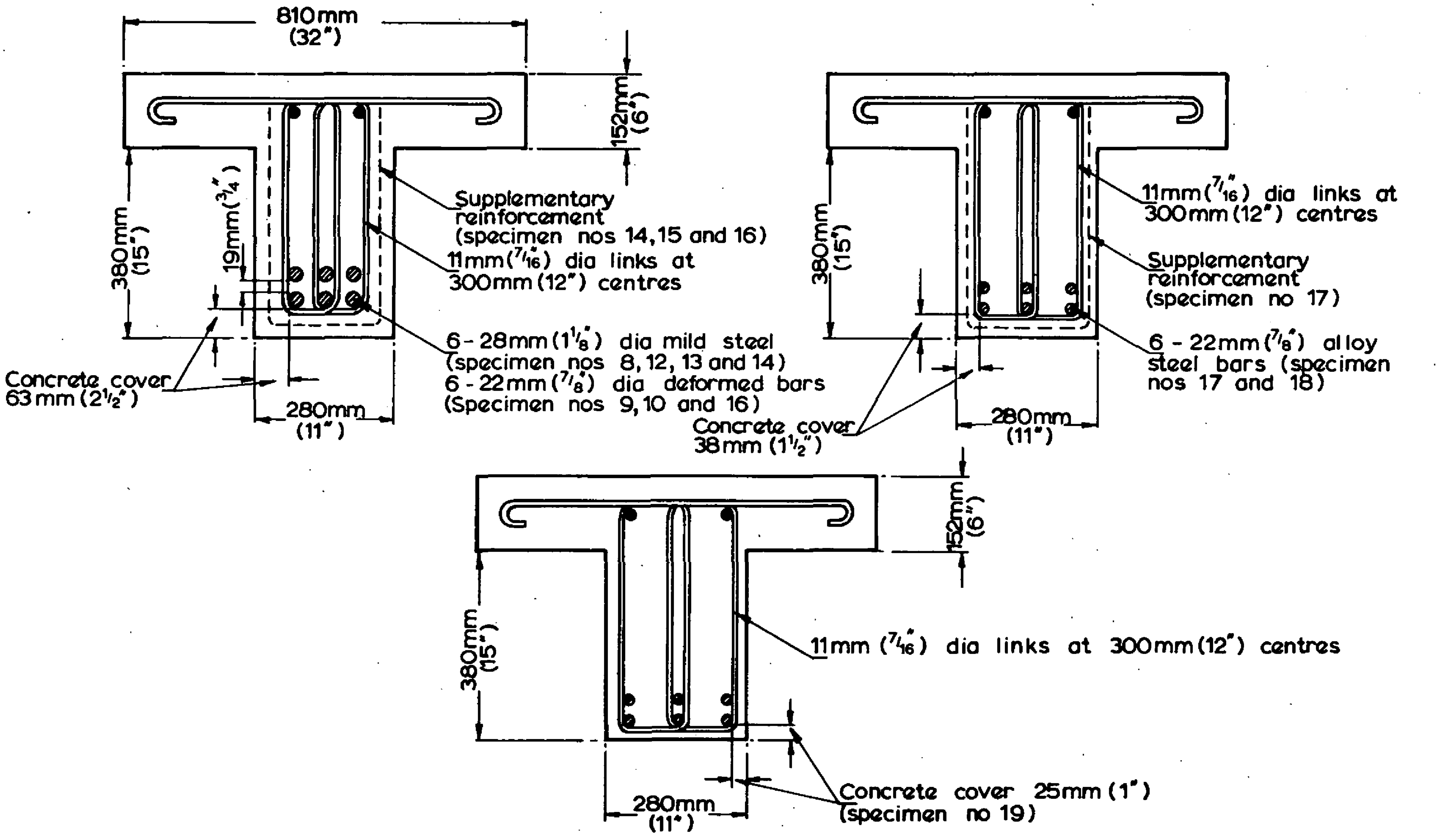


FIG. 3. CROSS-SECTION OF REINFORCED CONCRETE BEAMS

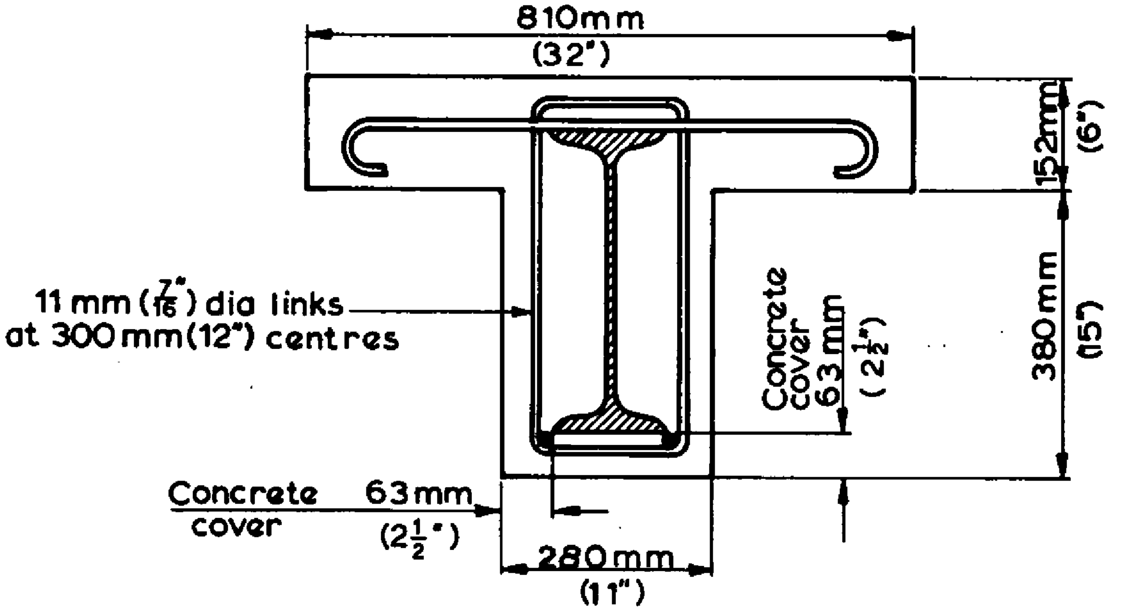


FIG. 4. CROSS - SECTION OF ENCASED STEEL BEAM

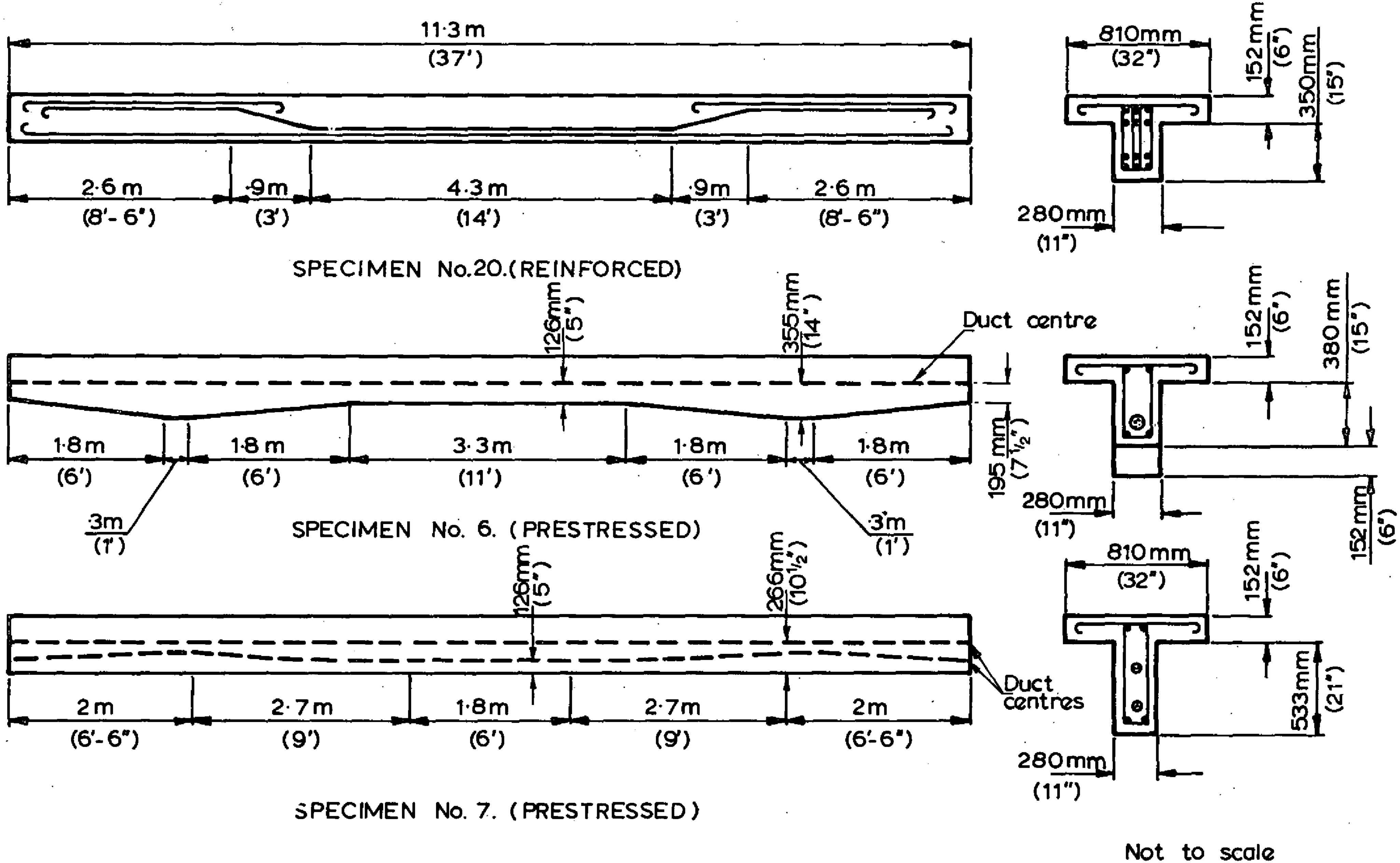


FIG. 5. DETAILS OF 11·3M (37') CONTINUOUS BEAMS

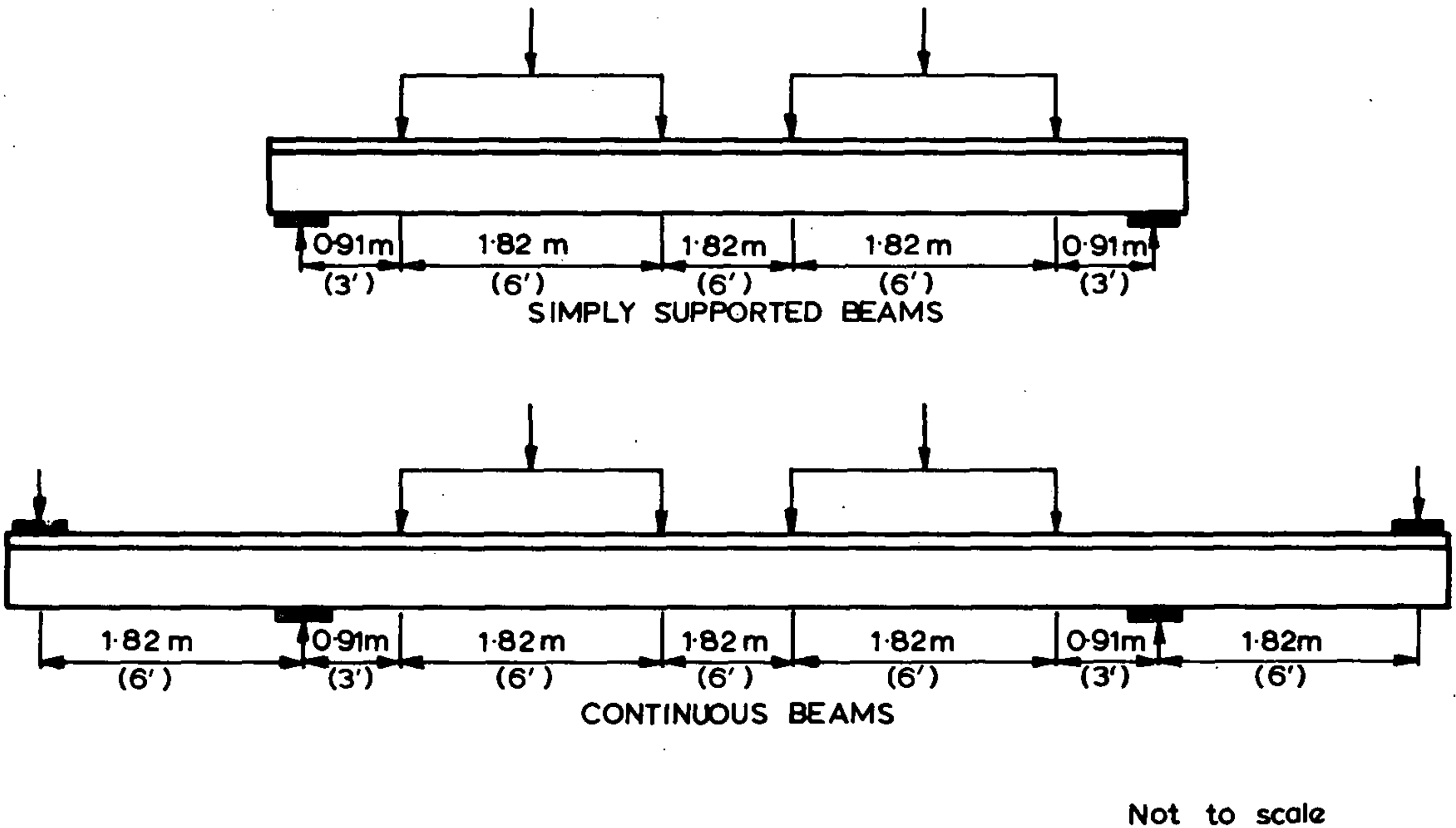


FIG. 6. LOADING OF BEAMS

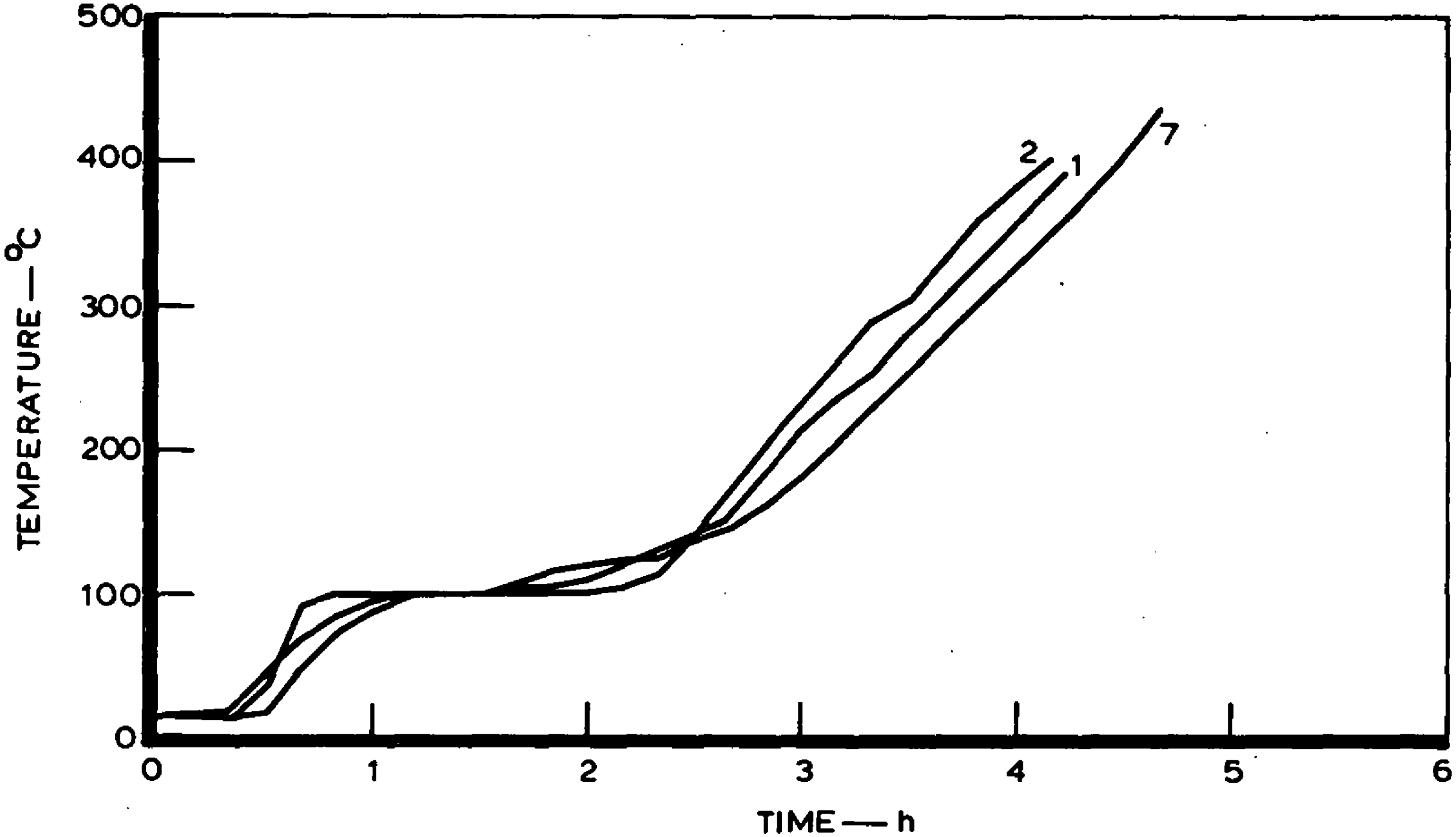


FIG. 7. MEAN REINFORCEMENT TEMPERATURE – PRESTRESSED CONCRETE BEAM Nos 1, 2 & 7

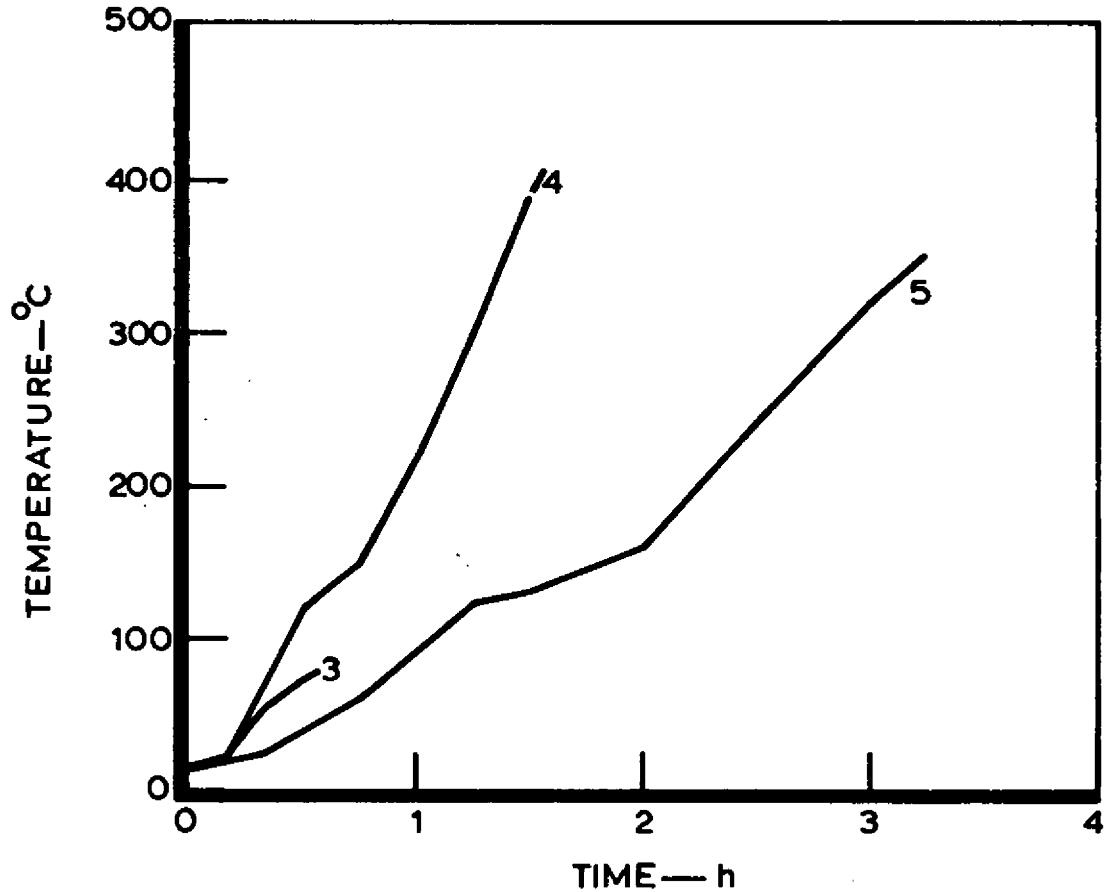


FIG. 8. MEAN REINFORCEMENT TEMPERATURE — PRESTRESSED CONCRETE BEAM Nos 3, 4 & 5

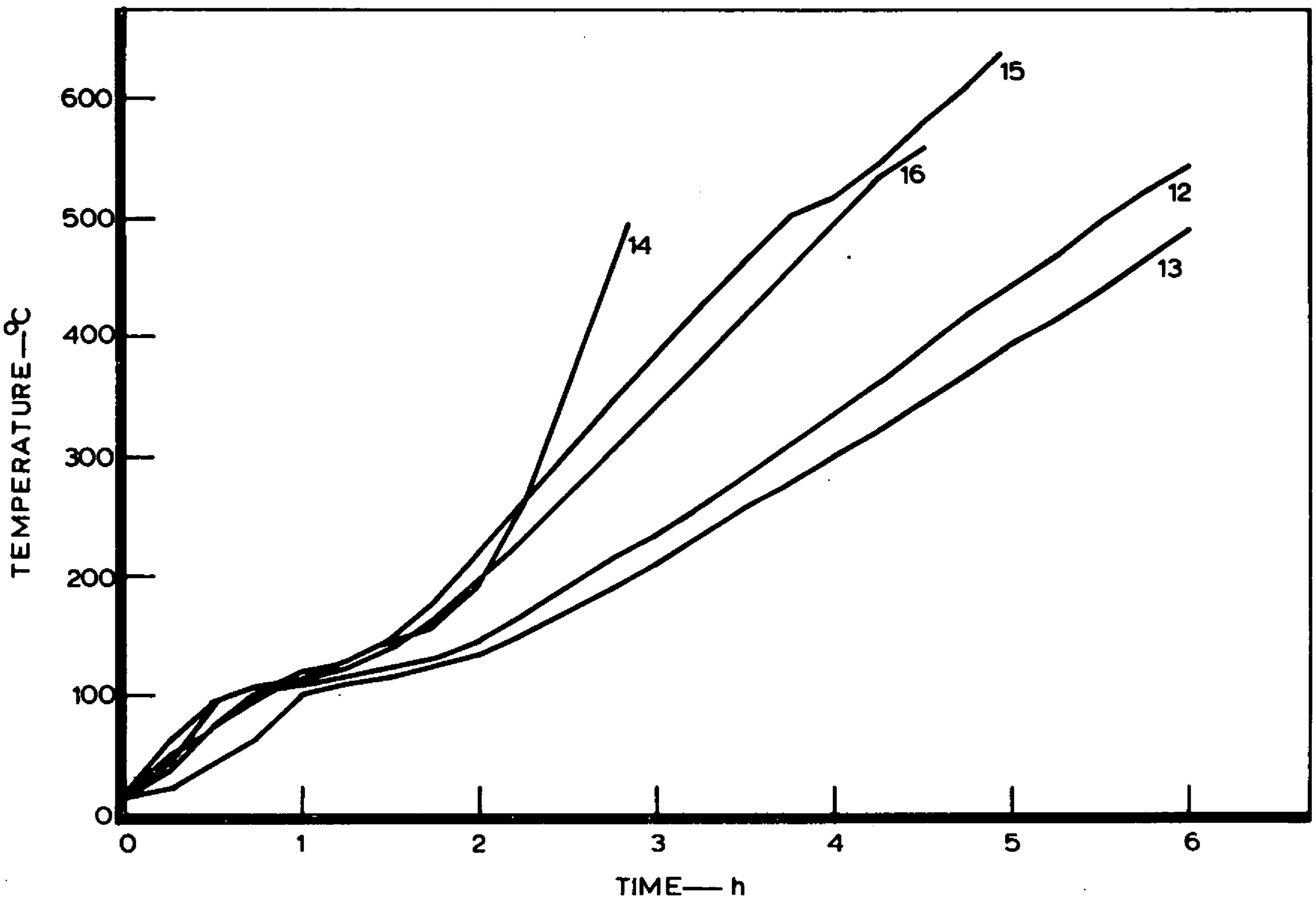


FIG. 9. MEAN REINFORCEMENT TEMPERATURE — REINFORCED CONCRETE BEAMS Nos 12, 13, 14, 15 & 16

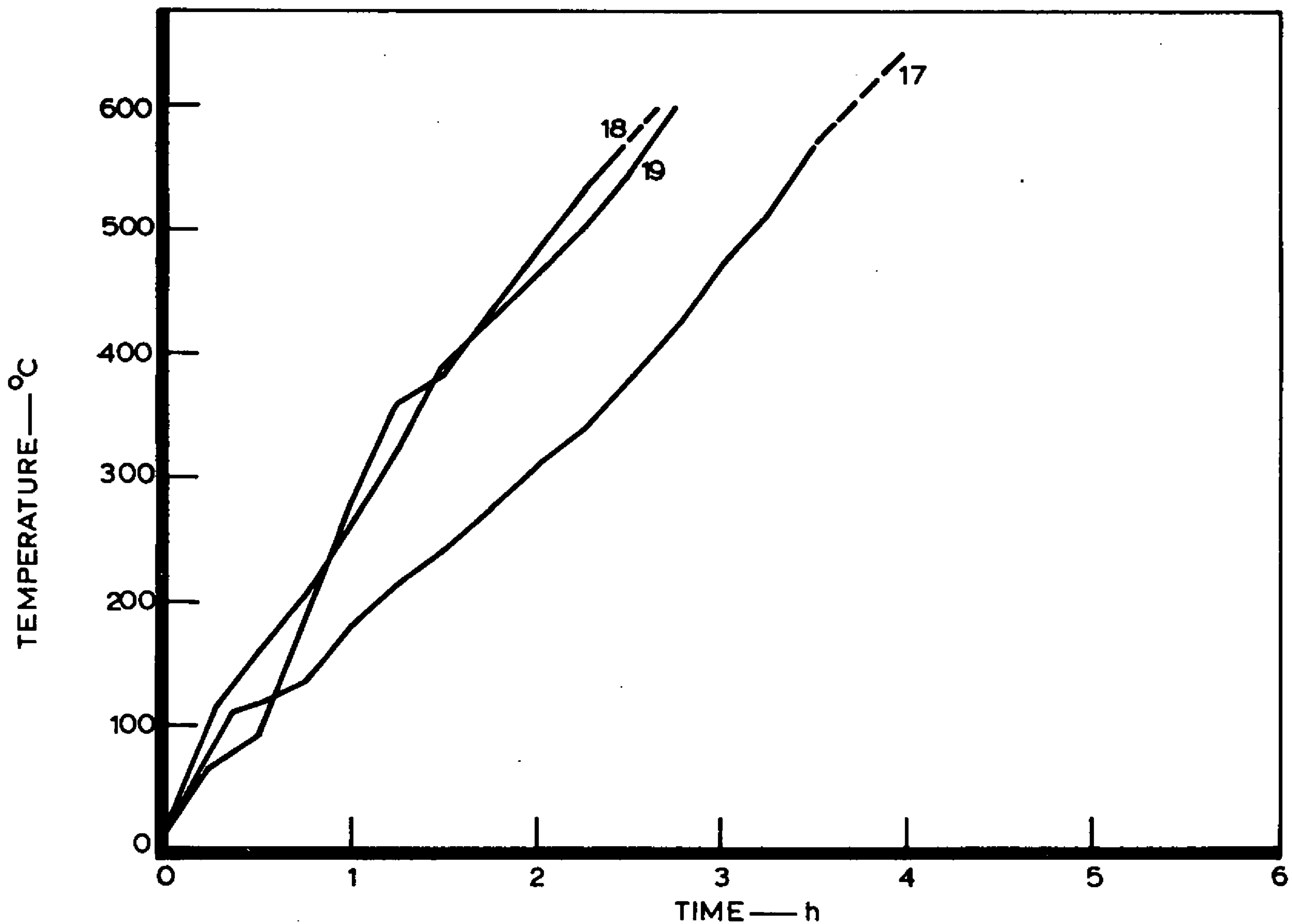


FIG. 10. MEAN REINFORCEMENT TEMPERATURE — REINFORCED CONCRETE BEAM Nos 17, 18, & 19

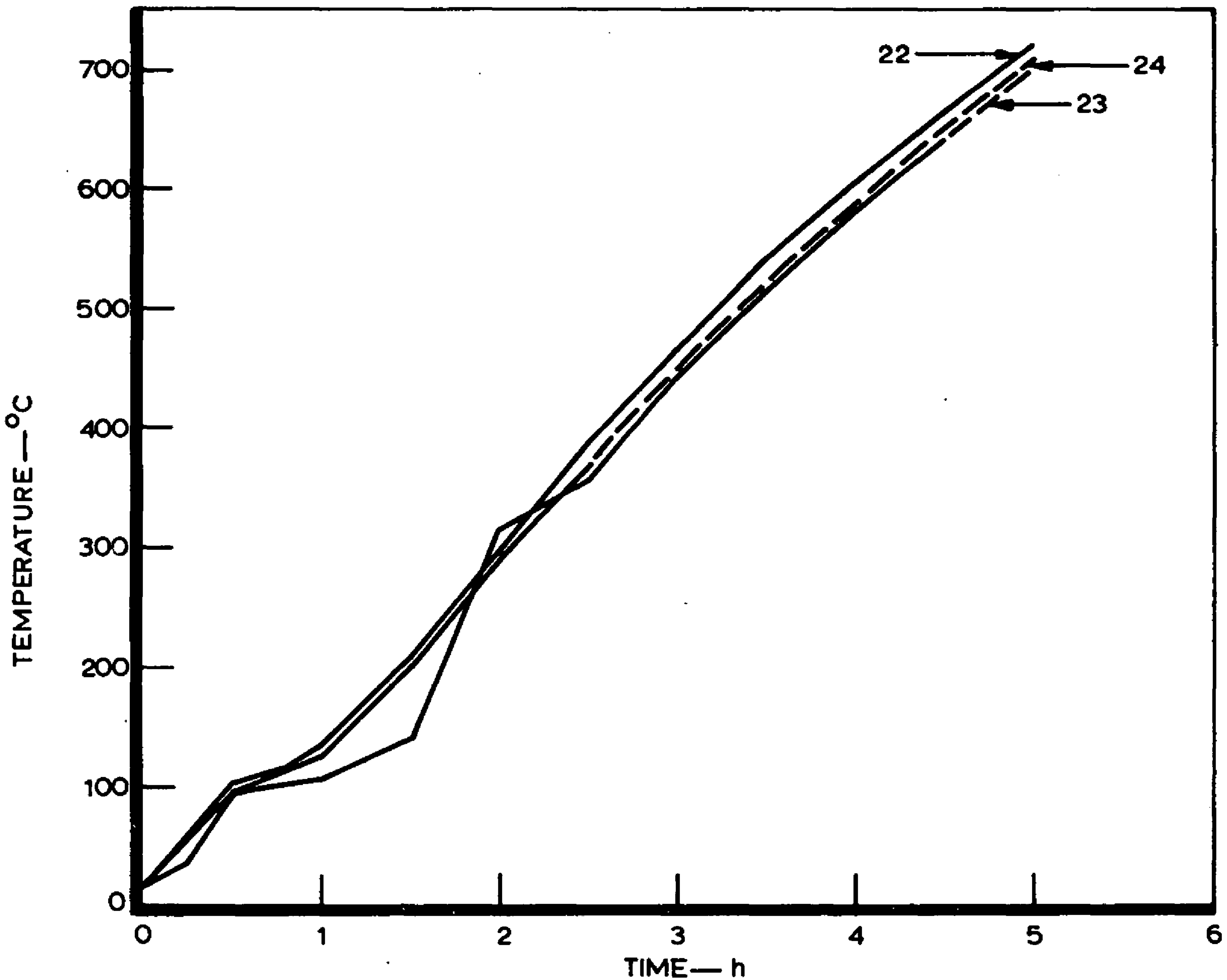


FIG. 11. TEMPERATURE OF LOWER FLANGE—ENCASED STEEL BEAMS Nos 22, 23 & 24

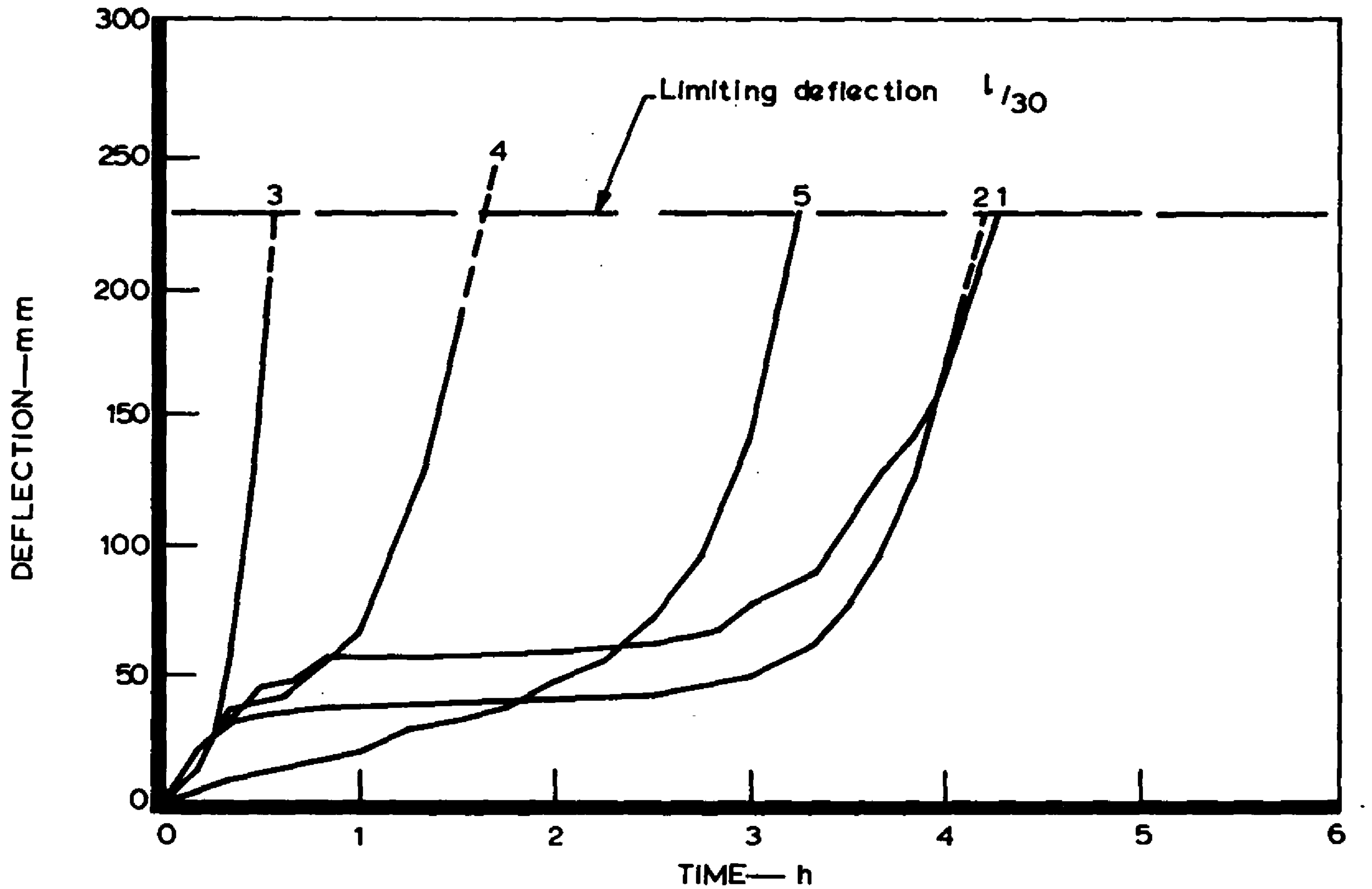


FIG. 12. CENTRAL DEFLECTION PRESTRESSED CONCRETE BEAM Nos 1,2,3,4 & 5

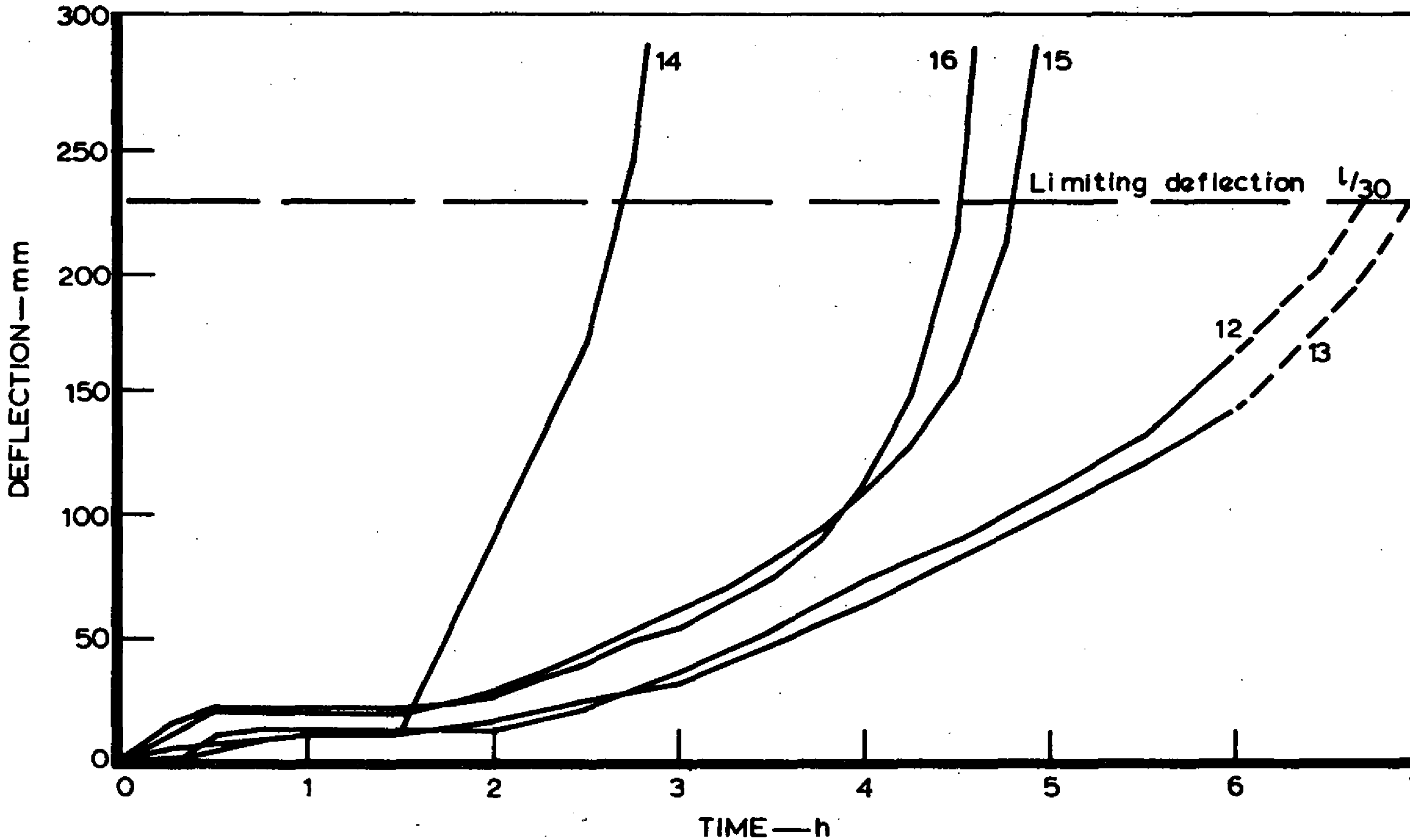


FIG. 13. CENTRAL DEFLECTION — REINFORCED CONCRETE BEAM Nos 12, 13, 14, 15 & 16

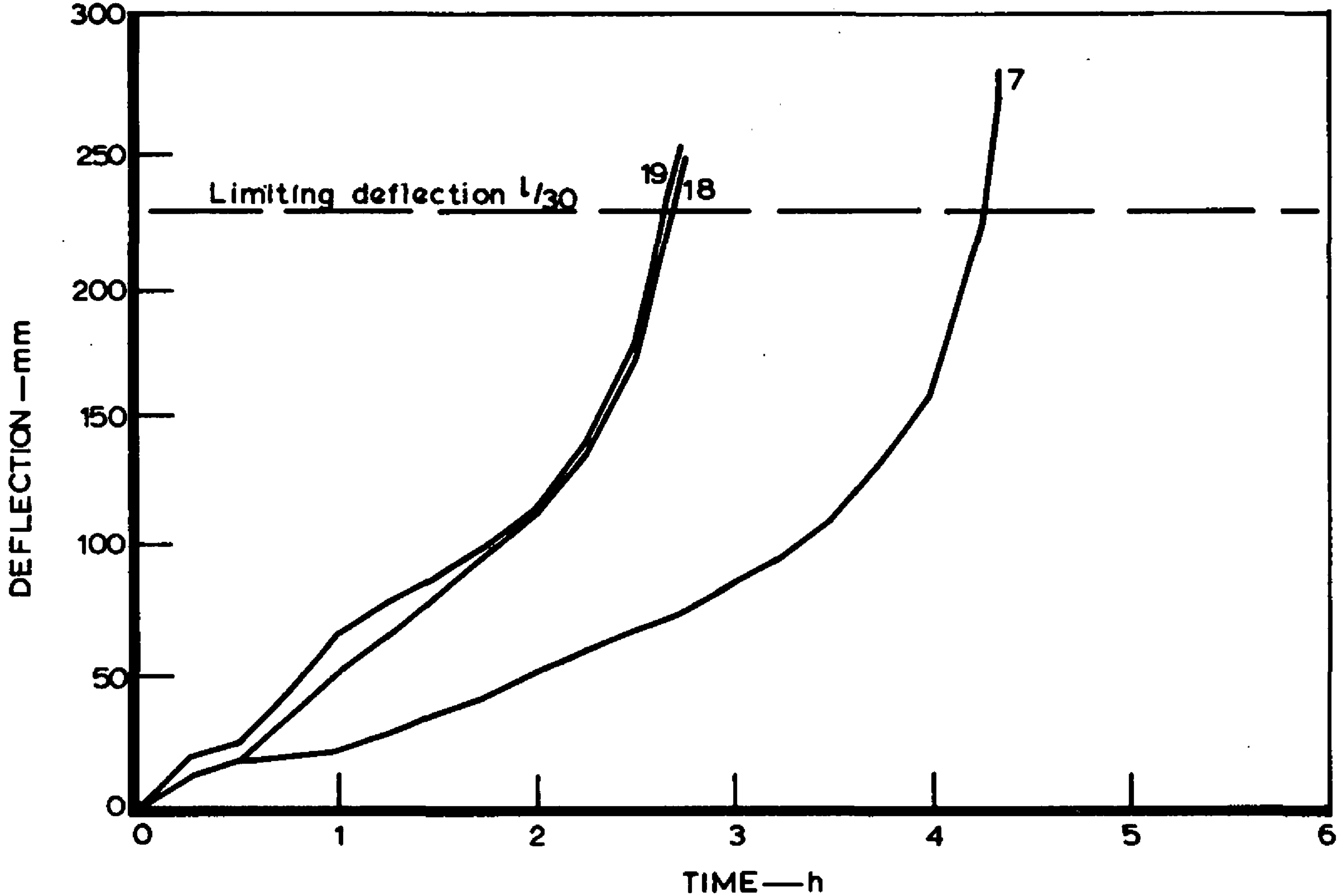


FIG. 14. CENTRAL DEFLECTION REINFORCED CONCRETE BEAMS Nos 17, 18 & 19

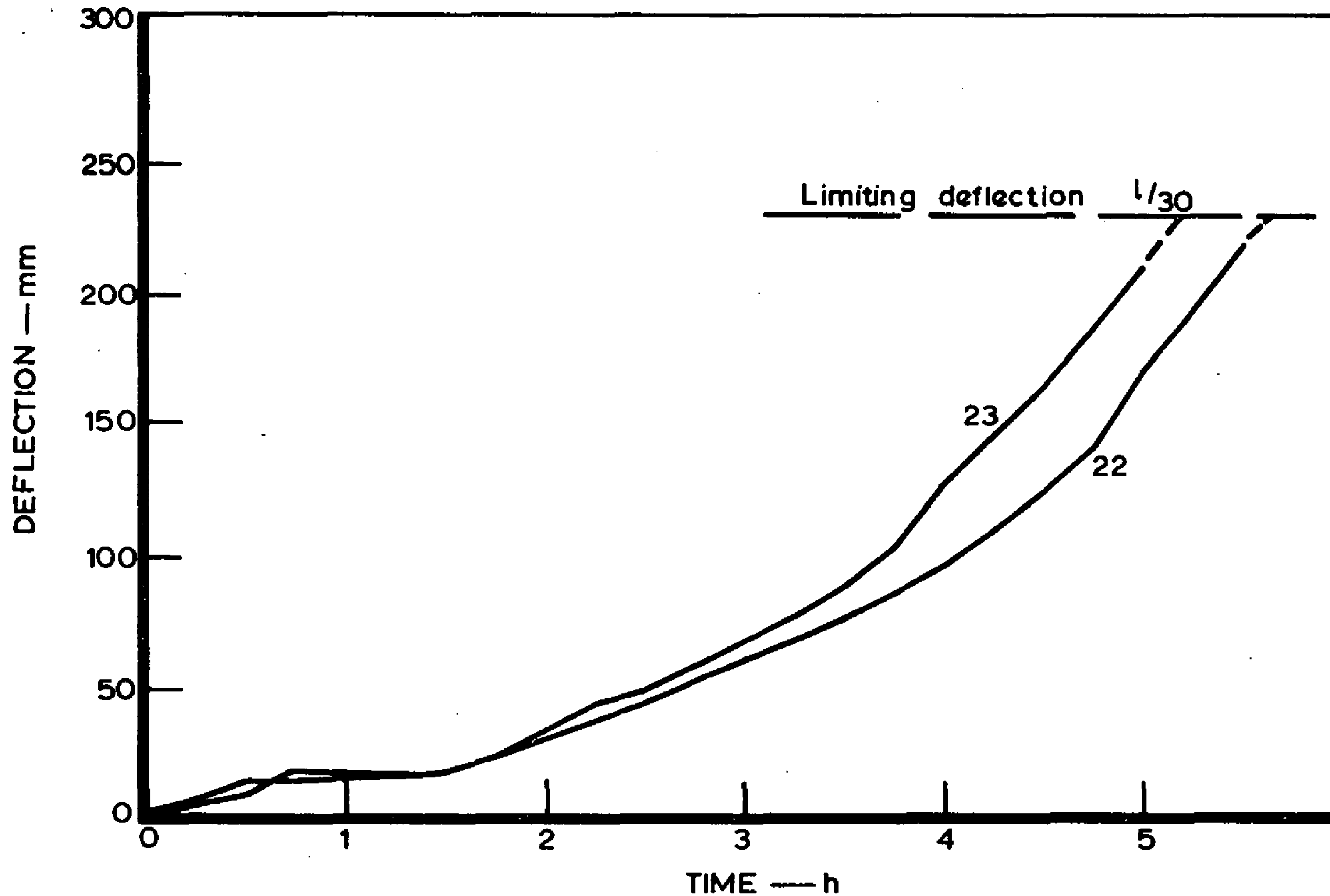


FIG. 15. CENTRAL DEFLECTION — ENCASED STEEL BEAM Nos 22 & 23

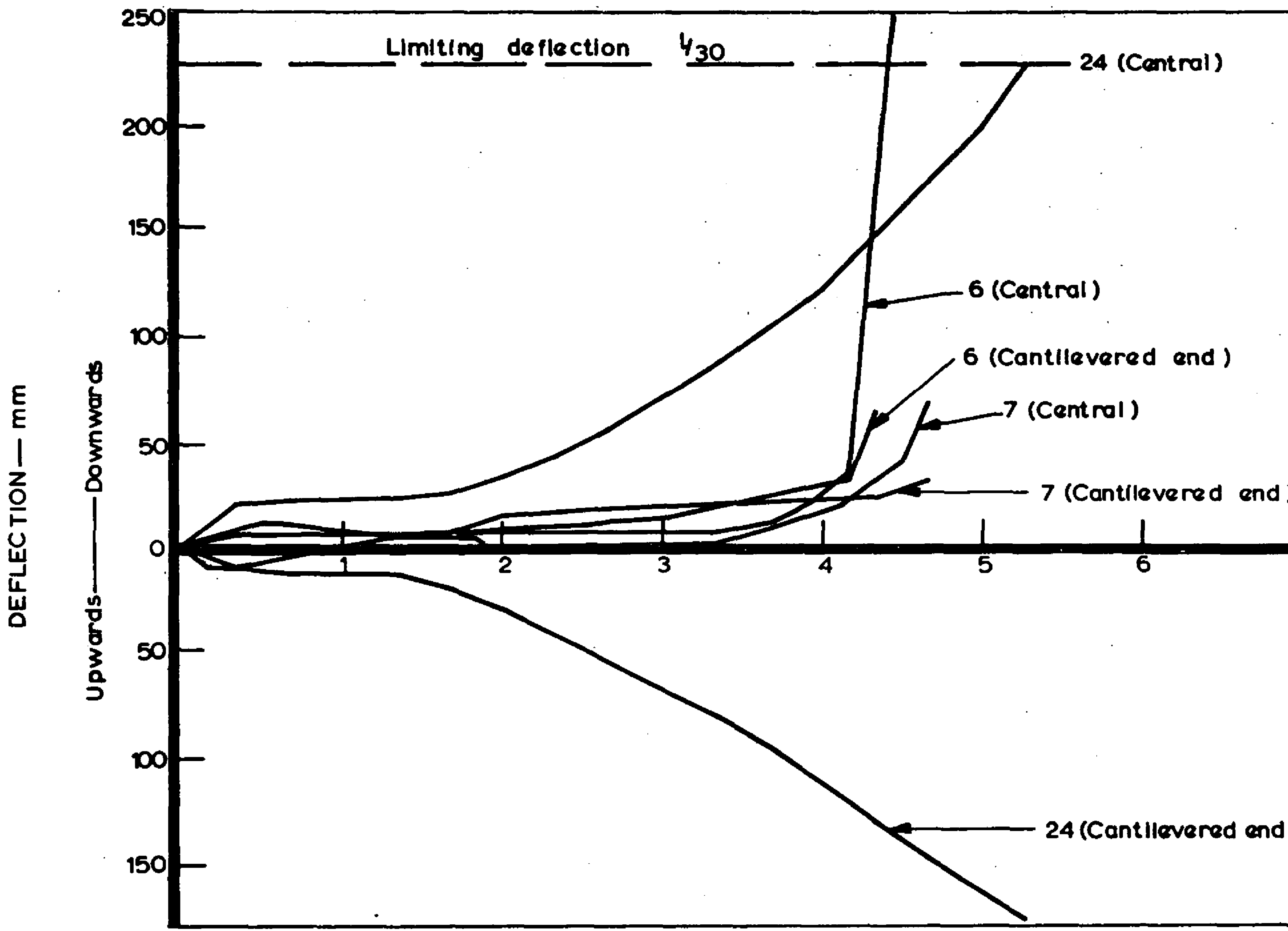
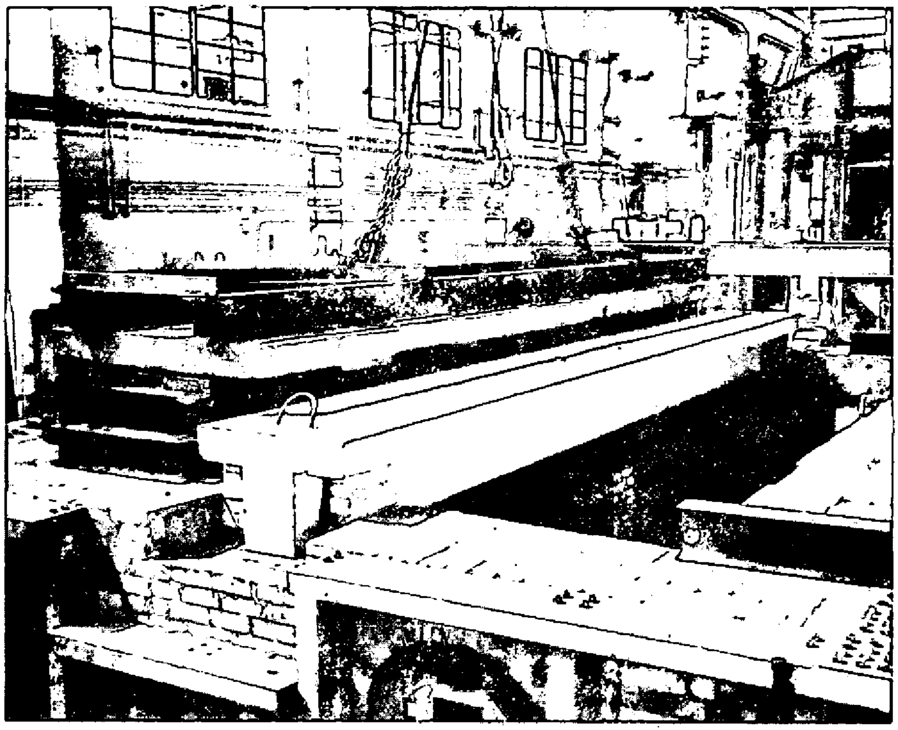
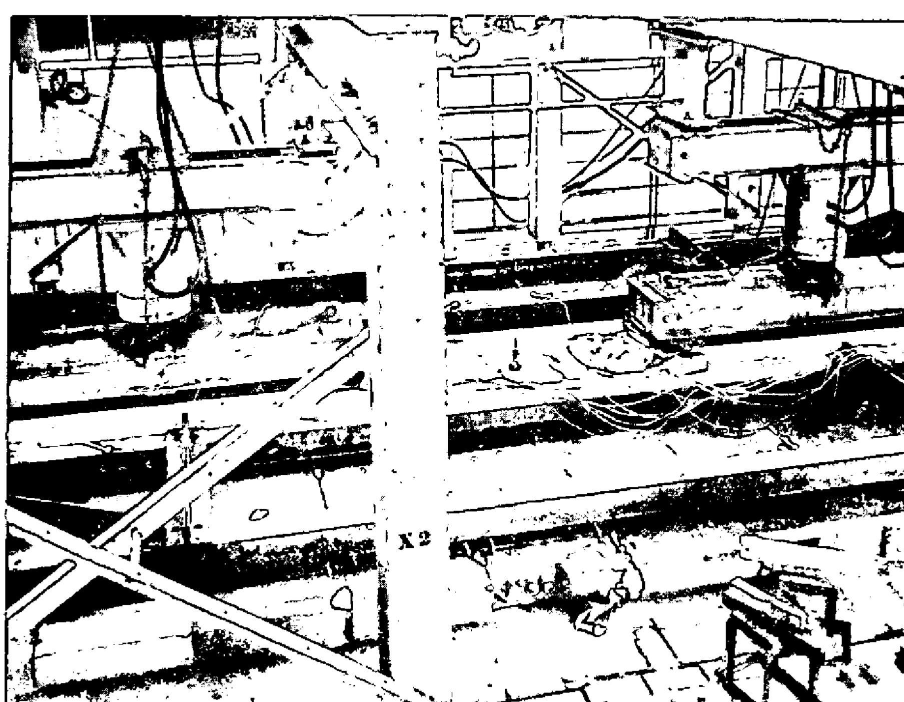


FIG. 16. DEFLECTION OF CONTINUOUS BEAMS



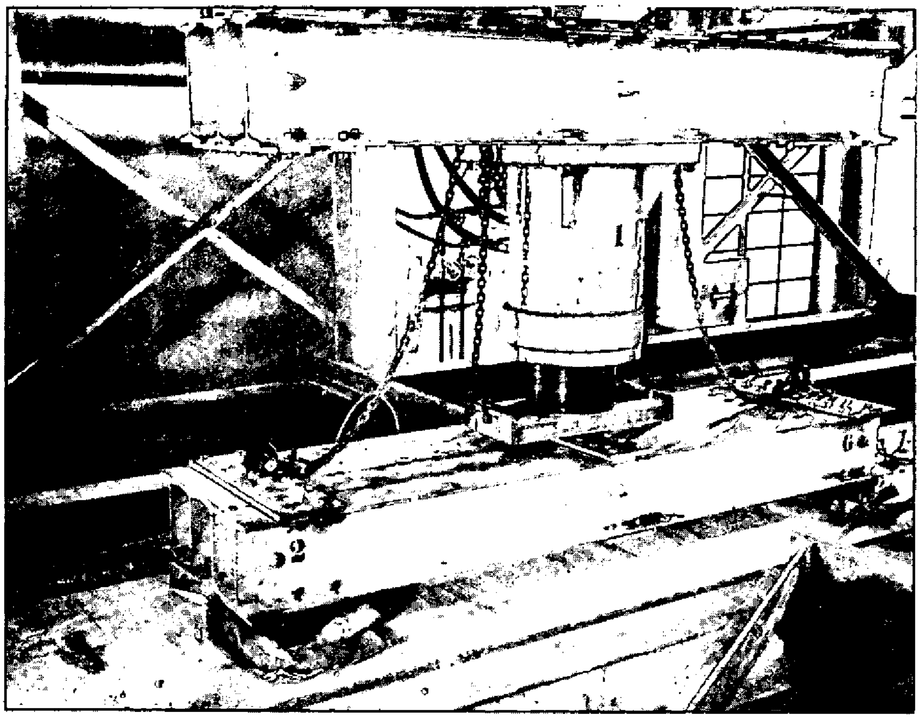
LOCATION OF A BEAM ON FURNACE

PLATE 1



LOADING GEAR IN POSITION

PLATE 2



CLOSE UP OF A HYDRAULIC JACK AND
LOAD DISTRIBUTOR

PLATE 3

WEB FRACTURE OF I-SECTION PRESTRESSED
CONCRETE BEAM

PLATE 4

I-SECTION PRESTRESSED CONCRETE BEAM WITH WEB REINFORCEMENT

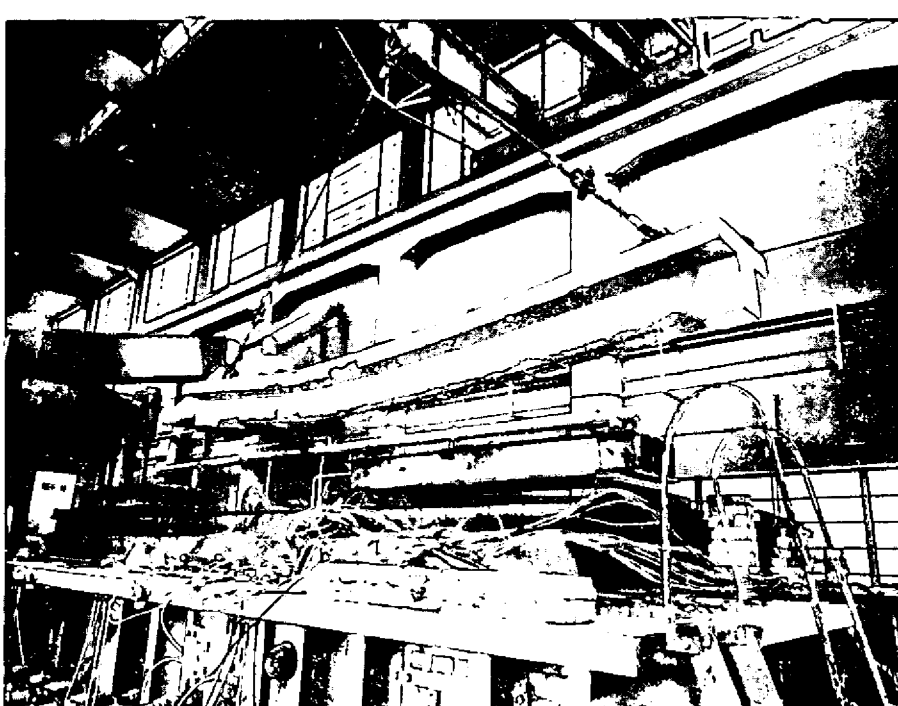
PLATE 5

SPALLING OF CONCRETE EXPOSING REINFORCEMENT OF A REINFORCED CONCRETE BEAM

PLATE 6

LIGHTWEIGHT CONCRETE BEAM AFTER TEST FOR 6 HOURS

PLATE 7

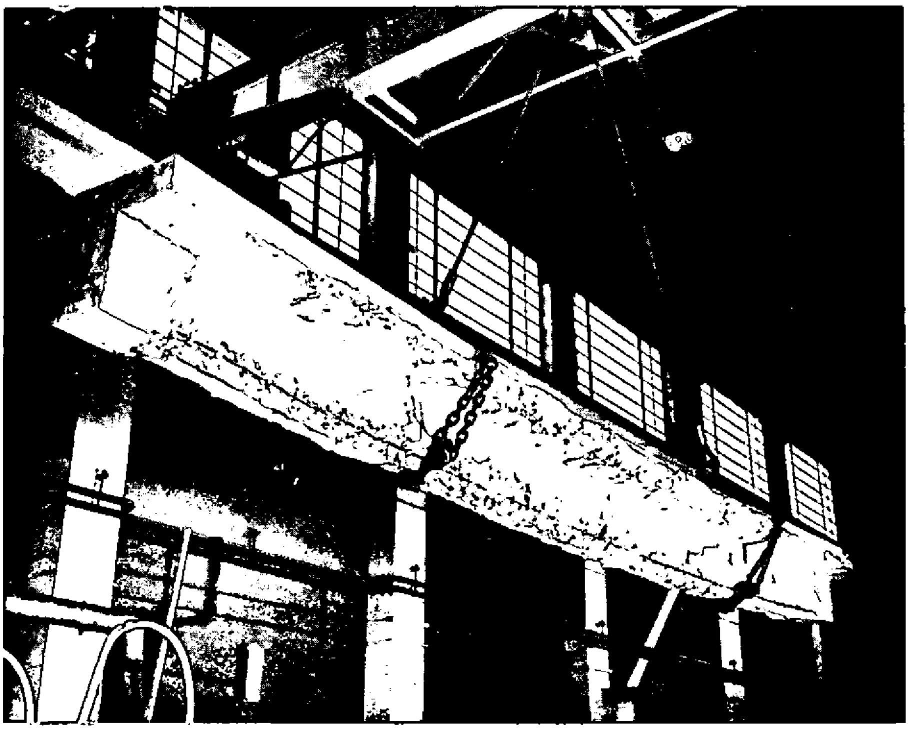


REINFORCED CONCRETE BEAM 38 MM COVER AND
NO SUPPLEMENTARY REINFORCEMENT

PLATE 8

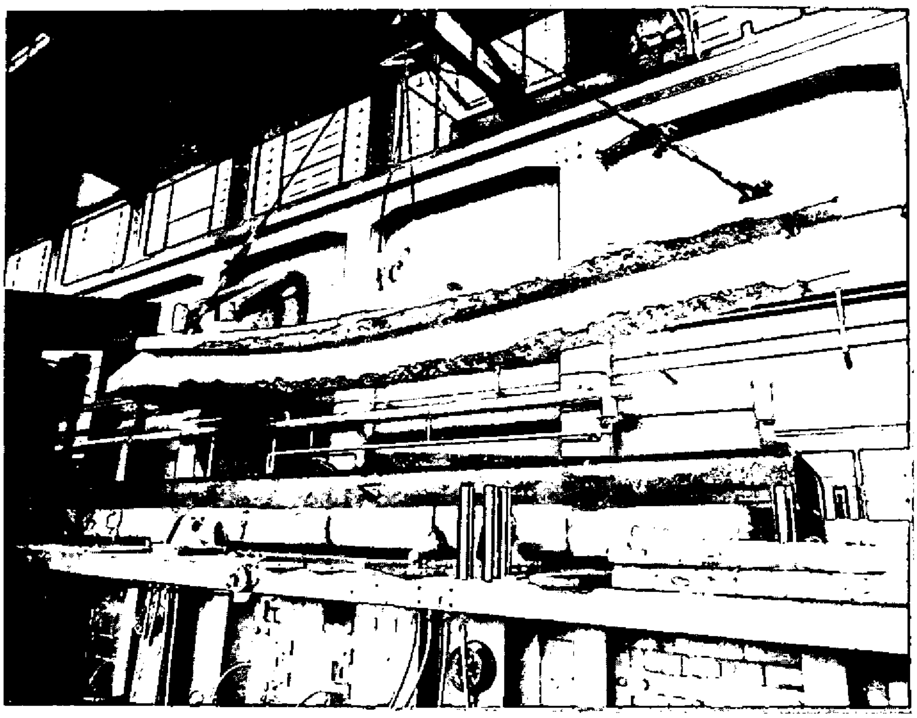
SPALLING OF CONCRETE FROM ARRISES UP TO
SUPPLEMENTARY REINFORCEMENT

PLATE 9



STEEL BEAM WITH CONCRETE ENCASEMENT
SPALLING OF CONCRETE AT ARRISES

PLATE 10

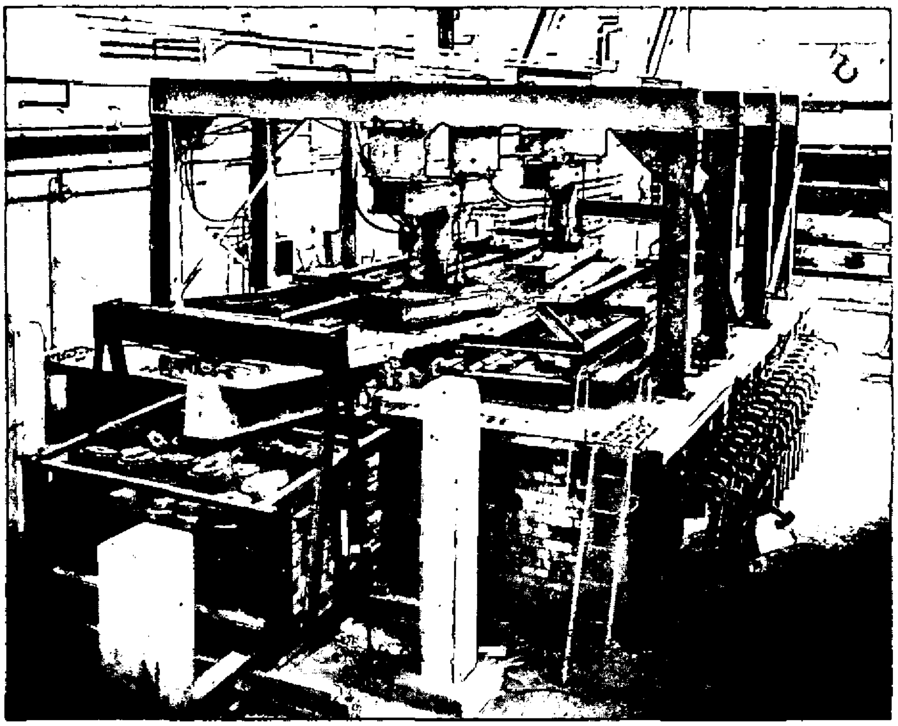


A TESTED BEAM AFTER TEST

PLATE 11

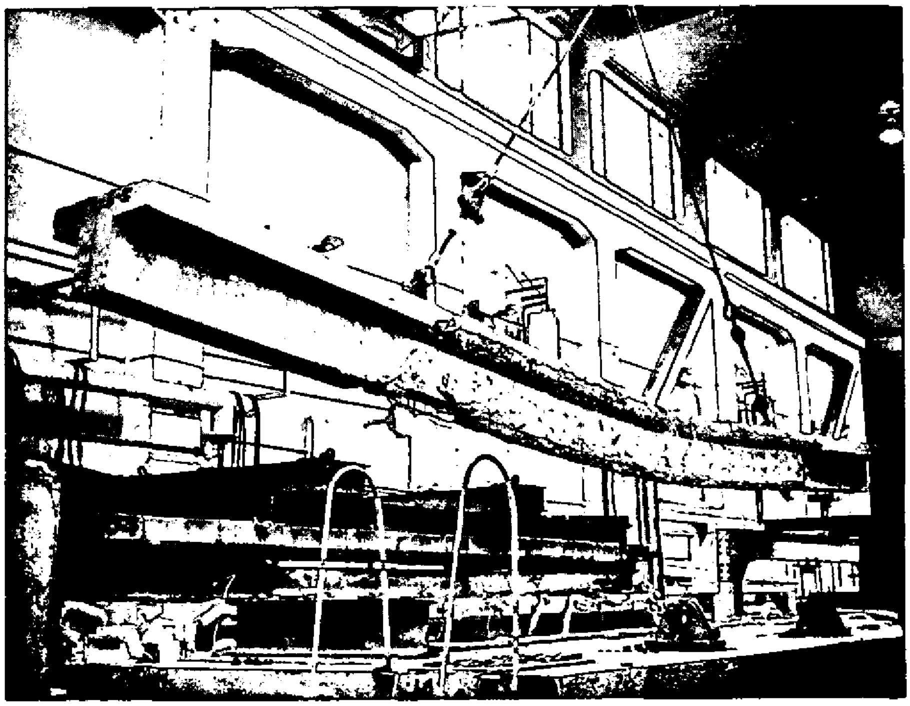
11.3 M BEAM BEFORE TEST

PLATE 12



11.3 M CONCRETE ENCASED STEEL BEAM DURING TEST

PLATE 13

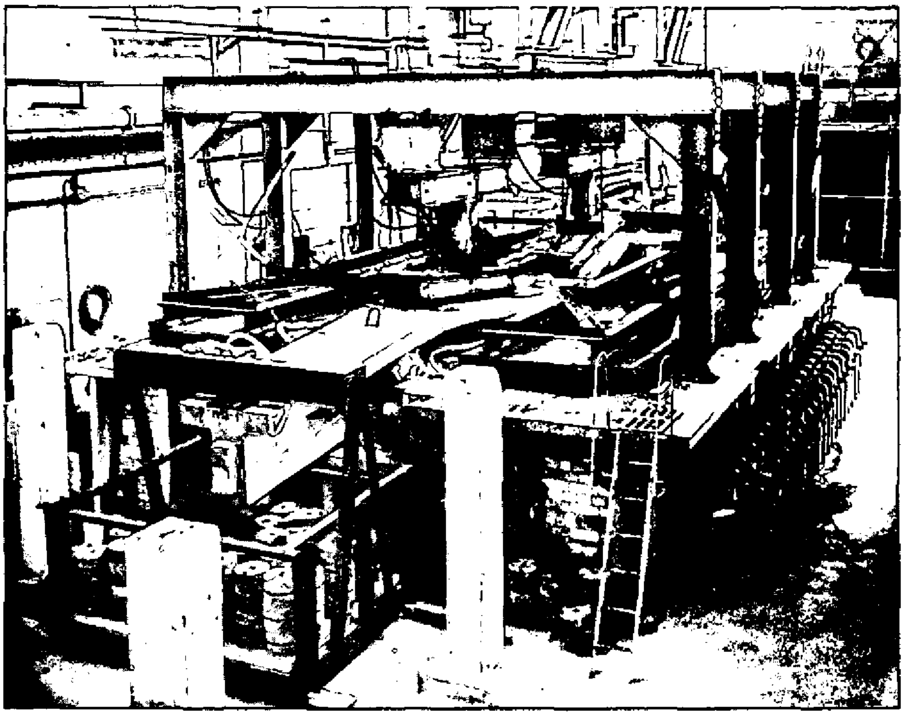


11.3 M CONCRETE ENCASED STEEL BEAM AFTER TEST

PLATE 14

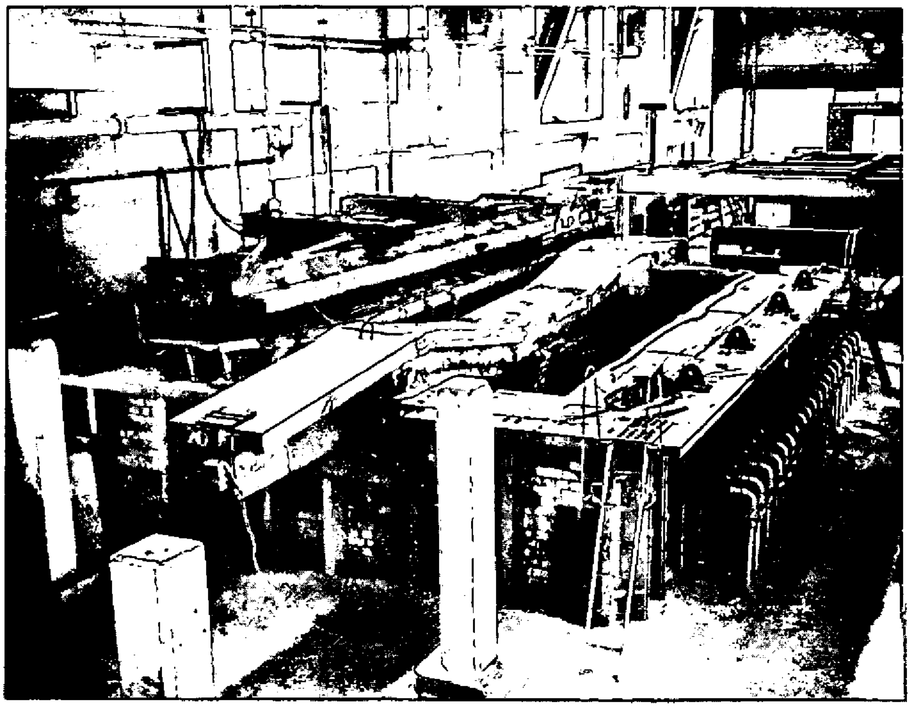
11.3 M REINFORCED CONCRETE BEAM AFTER TEST
SHOWING SEVERE SPALLING

PLATE 15



11.3 M PRESTRESSED CONCRETE BEAM AT END OF TEST

PLATE 16



COLLAPSED BEAM WITH LOADING GEAR REMOVED

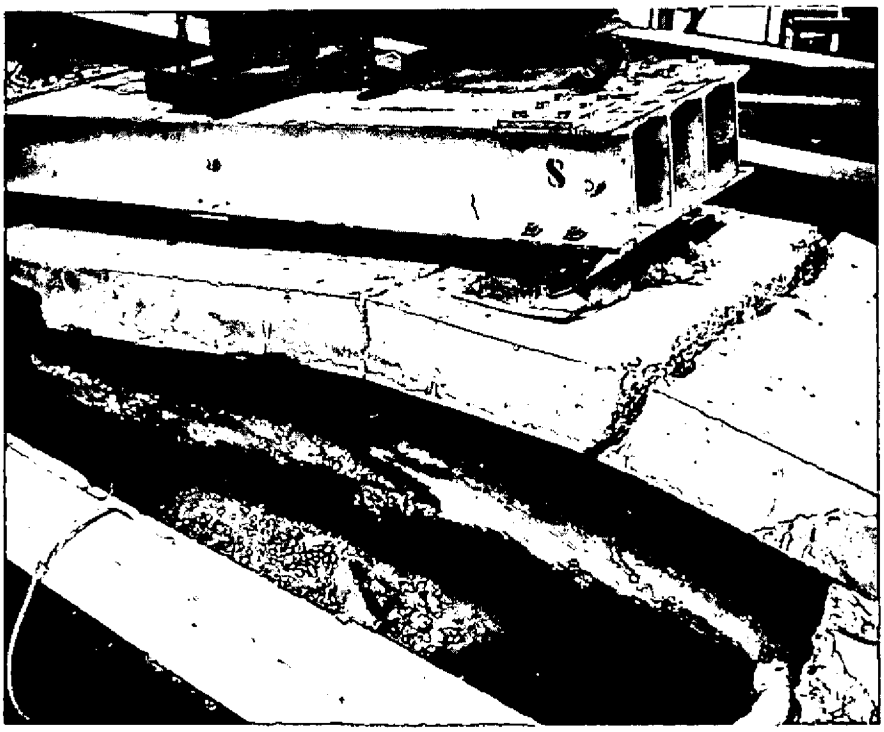
PLATE 17

VIEW SHOWING COLLAPSE AT CENTRE

PLATE 18

CLOSE-UP OF COLLAPSE NEAR SUPPORT

PLATE 19



COLLAPSE OF BEAM NEAR SUPPORT

PLATE 20