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THE THERMAL MOVEMENT OF CONCRETE FLOOR UNITS UNDER FIRE CONDITIONS

by

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SUMMARY

With some forms of precast building construction the failure of the structural element junctions may cause premature collapse of the building in the event of fire. There is no equipment that can assess under fire conditions the behaviour of a building system and for this reason the equipment normally used for fire resistance tests on individual elements was utilised. The experiment was designed to provide data on the movement that could be expected in concrete floor slabs and to assess whether failure at the supporting walls could result.

KEY WORDS: Building - industrialized, Concrete - reinforced, expansion, Fire - resistance

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1. INTRODUCTION

British Standard 476 : Part 1 : 1953 describes the test procedure which is applied to elements of construction such as walls and floors to assess their fire resistance. The equipment used to conduct these tests has been in use since 1935 and was developed to deal primarily with individual building elements. With the traditional forms of building construction the elements forming the structural framework are adequately jointed together during the erection of the building so that they become an integral part of one another. Therefore tests on the individual elements give a valid assessment of their behaviour when subjected to fire.

It is becoming increasingly apparent that for 'industrialised building' systems using precasting and site assembly techniques it is not sufficient to assess the individual elements under fire test conditions but to study the behaviour of the composite assemblies to establish whether the jointing methods adopted constitute a weakness.

Because the fire resistance test equipment was designed for assessing individual elements it is not possible to reproduce exactly a wall and floor junction arrangement developing the appropriate stresses. Until suitable facilities are available it is only possible to carry out a test as described in this note to measure the degree of longitudinal movement which might occur in a concrete floor when subjected to fire and to assess the effect of this movement on the supporting walls.

The test was conducted in co-operation with Taylor Woodrow - (Anglian Ltd), who supplied two reinforced hollow concrete floor units and four concrete walls.

A specimen floor constructed of similar floor units had been tested in the conventional way according to British Standard 476 : Part 1 : 1953 at an earlier date and the results are recorded on Joint Fire Research Organization File No. F.1025/1/175.

2. TEST ARRANGEMENT

The two floor units in conjunction with their supporting walls were built into the furnace normally used for fire resistance tests on floor constructions.

Two constructions, one with the floor slab tied into the walls, referred to as Construction 1, and the other with the floor slab simply supported on the walls, referred to as Construction 2, were built side by side in the furnace in such a way that there could be no interaction between them during the fire test.

Details of the test arrangement are shown in Figures 1 and 2.

3. DETAILS OF COMPONENTS

3.1. The concrete floor units were 4.57 m (15 ft) long overall x 1.16 m (5 ft 3¼ in) wide x 200 mm (8 in) thick and were constructed of gravel aggregate concrete. The units had 102 mm (4 in) dia. longitudinal cores at 150 mm (6 in) centres and were reinforced with 11 mm (0.437 in) dia. high tensile steel bars.

The bearing edges of the units were provided with 40 mm (1.6 in) deep x 40 mm (1.6 in) wide sloping ribs at 150 mm (6 in) centres and the longitudinal reinforcement extended into these ribs.

The unit for Construction 1 had projecting 9.5 mm (⅜ in) dia. mild steel links at 305 mm (12 in) centres at the bearing edges.

Details of the floor units are shown in the drawing Figure 3.

When the test was conducted the units had been cast approximately 3 months and had been stored outside without cover. It is normal practice to allow concrete constructions to attain a stable moisture condition before test as this may affect their performance but this procedure was not followed in the present case as the main interest was with the measurement of thermal movement.

The concrete wall units had overall dimensions 1.58 m (5 ft 2 in) high x 1.66 m (5 ft 5½ in) wide x 178 mm (7 in) thick and were constructed of gravel aggregate. The units were cast specifically for this test and to enable the units to be handled and used in the test one week after casting a welded steel mesh was incorporated as reinforcement.

The wall units for Construction 1 had projecting 9.5 mm (⅜ in) dia. mild steel links at 305 mm (12 in) centres at the bearing edges and were arranged to be staggered with the links in the floor unit.

Details of the wall units are shown in the drawing Figure 4.

4. ASSEMBLY DETAILS

The wall units for both constructions were bedded onto the floor of the furnace with approximately 25 mm (1 in) clearance between adjacent units and 25 mm (1 in) clearance at the furnace walls. This ensured that there was no interaction between the two constructions and prevented restraint by the furnace wall.

Construction 1. The floor unit was accurately positioned on the wall units so that the bearing was only on the projecting ribs. Two 16 mm ($\frac{5}{8}$ in) dia. mild steel rods 1.63 m (5 ft 4 in) long were passed through the links and wired in position. In situ concrete comprising 3:2:1 mix by volume of gravel (9.5 mm ($\frac{3}{8}$ in) down)/sand/cement was cast to the depth of the floor unit. The cube strength of this concrete on the day of test (11 days after casting) was 135 kg/cm^2 (1930 lb/in^2).

To prevent ingress of the in situ concrete into the cores, asbestos rope packing was pushed in to a depth of approximately 50 mm (2 in).

The joint was the same at each end of the floor unit and is shown in the drawing Figure 5.

Construction 2. The recommended practice at the bearing for this type of floor unit is to interpose hardboard pads between the underside of the ribs and the bearing edge of the wall unit, and this practice was adopted for the test. To ensure that relative movement could occur between the floor and the wall units, building paper was placed along the upper face of the wall units before in situ concrete of the same mix as was used in Construction 1 was cast to the depth of the floor unit. Asbestos rope plugs were positioned in the cores again to a depth of 50 mm (2 in) to prevent ingress of concrete.

The joint, which was the same at each end, is shown in the drawing Figure 6.

The gaps between the wall units and the furnace wall and between the floor units were suitably packed with asbestos rope and the face of the wall units to be exposed to fire were plastered with lightweight aggregate plaster to minimize spalling due to their wet condition.

5. TEST PROCEDURE

Construction 1. Steel sections were fitted between one wall unit and the steel members surrounding the furnace, so that the unit could be retained in a fixed position during the fire test. Dial gauges were fixed at the other end in the positions shown in Figure 7 to measure during the test the longitudinal movement and the vertical movement at the bearing edge.

Construction 2. Similar steel members to Construction 1 were fitted to the furnace surround and to both of the wall units so that they were retained in a fixed position. Relative movement between the floor and wall units could occur due to the presence of the building paper and dial gauges were fixed at each end in the positions shown in Figure 7 to measure longitudinal movement and vertical movement at the junction.

All of the dial gauges were mounted so that they could move in a fixed plane and steel plates were bonded to the concrete at the contact points.

The floor units were drilled from the upper side so that thermocouples to measure the furnace temperature could be passed through. The thermocouples were arranged 76 mm (3 in) below the soffit.

The floor units were loaded with cast iron weights to give a uniformly distributed load of 439 kg/m^2 (90 lb/ft^2); this was the load calculated to develop the maximum permissible stress in the steel in accordance with C.P.114 : 1957. The loading used in the recent fire resistance test on similar units developed 30 per cent of the maximum permissible stress.

The load calculations are given in the appendix and no allowance was made for the continuity effect of Construction 1.

The underside of the floor units and the inner faces of the supporting walls were subjected to the heating conditions of B.S. 476 for fire resistance tests on structural elements.

The temperature of the upper surface of the floor units was measured by means of thermocouples soldered to copper discs and distributed over the surface area.

Vertical deflections at the centre of each floor unit were also recorded during the fire test.

6. TEST OBSERVATIONS

Observations were made during the test of each construction and are tabulated below:

Construction 1 - Tied ends

Time h min	Observation
0 00	Test started
0 20	Transverse crack over bearings between in situ concrete and floor unit
0 26	Longitudinal crack on upper surface of floor unit for full length
0 30	Water dripping from fissures in underside of floor unit

Construction 1 - Tied ends (cont'd)

Time		Observation
h	min	
0	40	Faces of wall units exposed to fire spalling badly
0	45	Transverse cracks over bearings opening slightly
0	55	Transverse cracks between in situ concrete and wall units
0	56	Severe spalling of wall units on faces exposed to fire
1	00	Longitudinal crack becomes wider
1	20	No further significant change
2	00	Increase in rate of deflection of floor unit
2	15	Floor unit deflecting rapidly. Collapse imminent. Test stopped

Construction 2 - Simply supported

Time		Observation
h	min	
0	00	Test started
0	15	Slight spalling on underside of floor unit
0	35	Further spalling on underside of floor unit. Water dripping from fissures in concrete
1	00	No significant change in appearance
1	20	Face of one wall unit exposed to fire beginning to spall
2	00	Rate of deflection of floor unit increasing
2	15	Rapid deflection of floor unit. Collapse imminent. Test stopped.

7. TEST RESULTS

The graphs Figures 8-11 give the data recorded during the test.

From Figure 9 it can be seen that although the test was continued until imminent collapse of the floor units, there was some recovery of deflection after the furnace had been shut off.

A photographic record was kept of the test and is shown in Figures 14-19.

When the furnace had cooled down to ambient conditions a detailed examination of the construction was made and the following points noted.

Construction 1 - Tied ends

Severe spalling had occurred on the face of both wall units exposed to fire to such an extent that at the bearing where the wall unit was not restrained in position the ribs of the floor unit were no longer acting as bearing points. The *in situ* concrete was carefully removed and the cause of the horizontal crack between the wall unit and *in situ* concrete was found to be due to the steel links pulling out of the wall unit. There was no evidence of the steel having yielded.

The detail photographs Figures 17, 18 and 19 show the extent of the spalling and the deterioration at the bearing edge.

Construction 2 - Simply supported ends

As can be seen from Figure 13 most of the longitudinal movement occurred at one bearing edge and detailed examination showed that the floor unit had moved with respect to the wall unit by about 29 mm (1.15 in) and the bearing was on the floor unit itself and the ribs were not in contact.

The wall unit where this relative movement occurred did not spall and there was no indication for the difference in its behaviour from the other three wall units as the plaster protection became detached at approximately the same time in all cases.

8. DISCUSSION OF RESULTS

From the data recorded, the movement of the floor and wall unit can be shown diagrammatically at different stages during the test.

Construction 1 - Tied ends

The relative movement of the floor junction with the wall not restrained in position is shown in Figure 12 at the end of the test when collapse of the floor was imminent. The maximum horizontal outward movement of the wall of 40 mm (1.57 in) was determined graphically from the data in Figure 10. This would result in an angular movement from the vertical of 0°-$56\frac{1}{2}'$ in a storey high wall. The maximum vertical movement of the corner of the floor was similarly computed to be 42 mm (1.65 in).

The angular movement between the floor slab and the wall was due to the inadequacy of the fixity provided. Had the floor edge and the wall remained integral there would have been lower vertical deflection of the slab in the middle. There is no indication, however, that the horizontal movement would have been significantly affected had this been the case.

Construction 2 - Simply supported ends

Figure 13 shows the graphically determined movement of the slab that occurred at each end of the floor unit when collapse of the floor was imminent. At one end this was 4.3 mm (0.17 in) but at the other end outward horizontal movement of the floor unit was 29 mm (1.15 in). This would have resulted in an angular movement of a storey high wall unit from the vertical of $0^{o}-42'$ if it were not restrained in position. The maximum vertical movement of the floor edge was 28 mm (1.1 in).

The total horizontal movement of the floor edge of 33.3 mm (1.31 in) (29 mm + 4.3 mm) was 6.7 mm (0.26 in) less than the 'tied-in' construction, the difference was primarily due to the larger vertical deflection of the simply supported slab in comparison with the specimen with tied ends.

9. CONCLUSIONS

A comparison has been made between the vertical and horizontal movement of two identical precast concrete floor slabs having simply supported and tied-in end conditions. The method of providing structural ties between the floor edge and the wall which was standard industrial practice, was not adequate and relative movement between the two occurred during the fire test. It was, therefore, not possible to obtain complete data on restrained end conditions.

The simply supported floor slab showed greater vertical deflection than the tied-in slab soon after the start of the test; the greatest difference occurred at 2 hours when its deflection was 160 mm (6.3 in) in comparison with 125 mm (4.9 in) for the specimen with tied ends.

The difference between the horizontal movement of the slab in the two cases were small, the slightly larger movement of the restrained unit being primarily due to lower vertical deflection. The movement of 40 mm (1.57 in) shown by this specimen represents an angular movement of less than 1^{o} of a storey high supporting wall. The indications are that movements of this order are unlikely to lead to structural instability.

When one wall was restrained, the whole of the movement occurred at the other wall indicating that in practice such movements are more likely to take place in external walls, but again this corresponded to an angular movement of less than 1^{o} in a storey high wall and would be unlikely to lead to structural instability.

The above investigation did not explore the stresses that might be set up in adjacent slabs when restraint is provided to horizontal movement at both ends.

The investigation has shown a need for further work on this subject, particularly to establish the magnitude of stresses due to horizontal movement and their likely effect on the structural stability of a whole building as well as the design of junctions to withstand deformation of slabs under fire conditions.

APPENDIX

BENDING MOMENT DUE TO DEAD LOAD

$Wt/ft^2 = 70$ lb

Wt/ft run $= 35$ lb

$$M = wl^2 \times 1.5 = 35 \times 15^2 \times 1.5$$
$$= 11{,}810 \text{ lb/in}$$

Bending moment (total imposed load) to produce the maximum allowable stress in the steel of 33,000 lb/in^2 in accordance with CP 114 : 1957

$$M_R = A_{ST} \times P_{ST} \times \left(d1 - \frac{ds}{2}\right)$$
$$= 0.148 \times 33{,}000 \times \left(6.69 - \frac{2.312}{2}\right)$$
$$= 0.148 \times 33{,}000 \times 5.534 = 27{,}000 \text{ lb in}$$

$\therefore$ Bending moment to be developed by applied load

$$= 27{,}000 - 11{,}810 = 15{,}190 \text{ lb in}$$

$$\therefore \quad Wt/ft^2 = \frac{15{,}190}{15^2 \times 1.5} \times \frac{12}{5.9}$$
$$= 91 \text{ lb/ft}^2 \quad (\text{say } 90 \text{ lb/ft}^2)$$

Imposed load to develop maximum permissible stress $= 90$ lb/ft^2

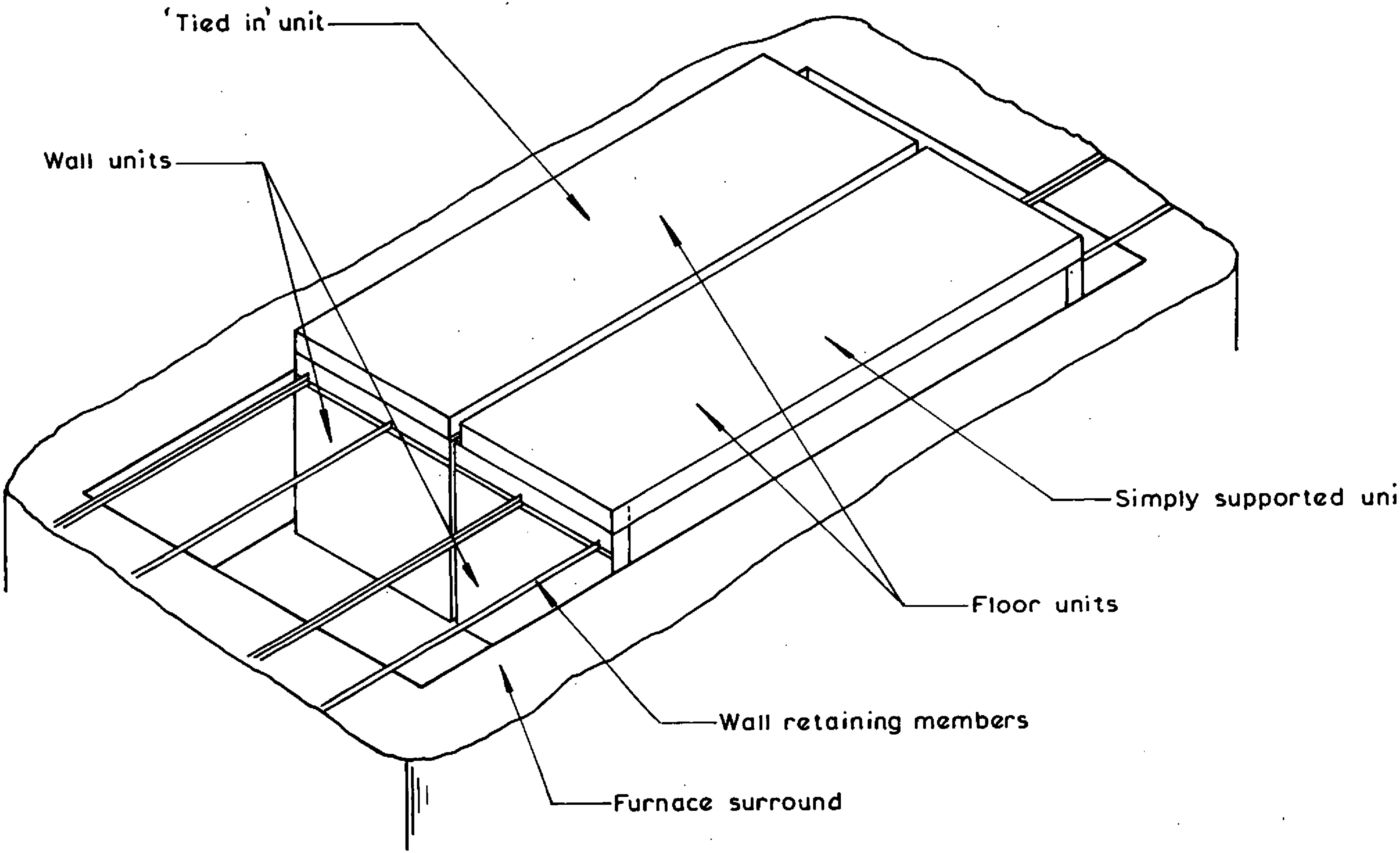

FIG.1 GENERAL ARRANGEMENT OF FLOOR UNITS IN FURNACE

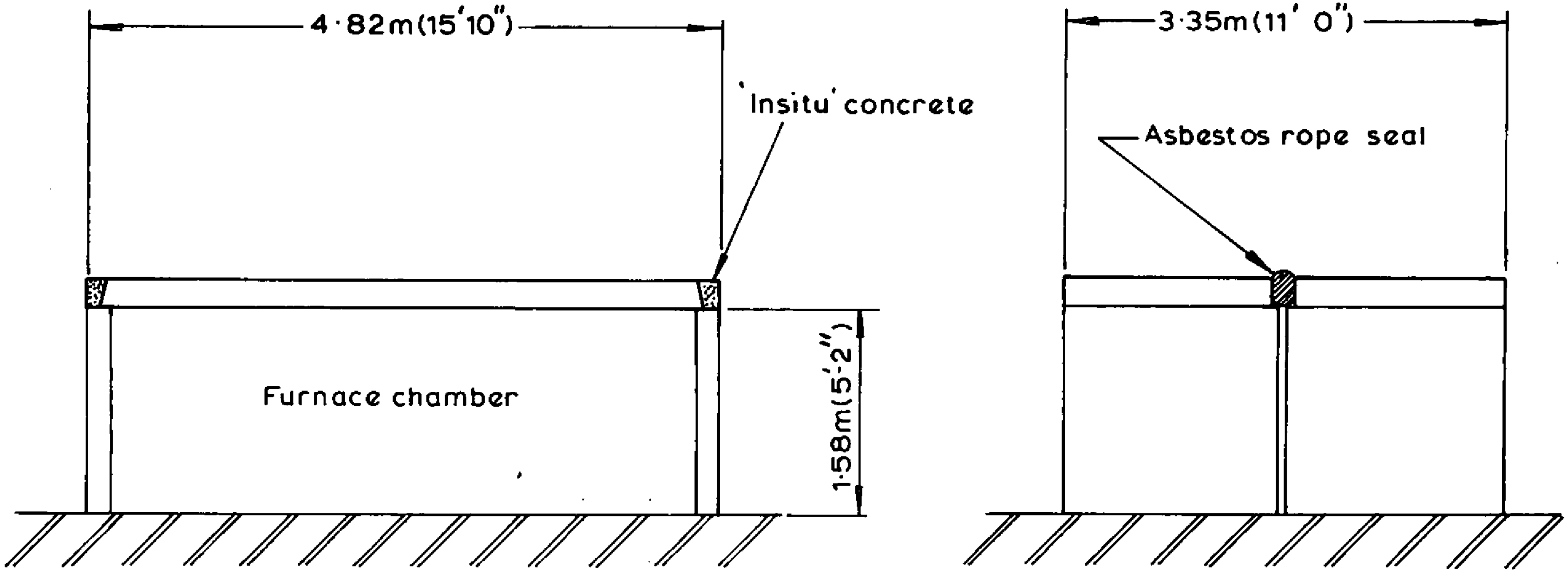

FIG. 2 DETAIL OF FLOOR UNITS

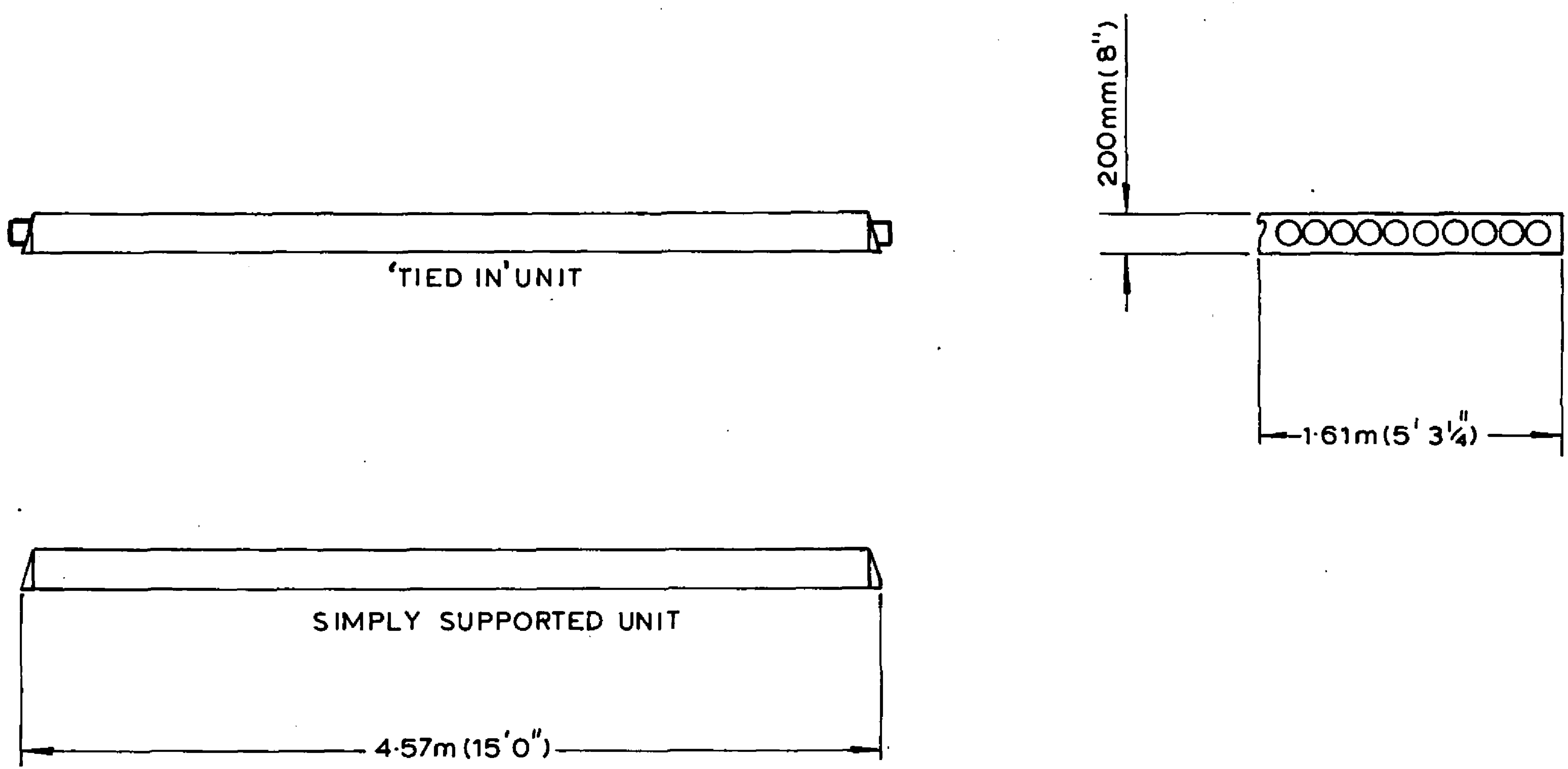

FIG.3 DETAIL OF FLOOR UNITS

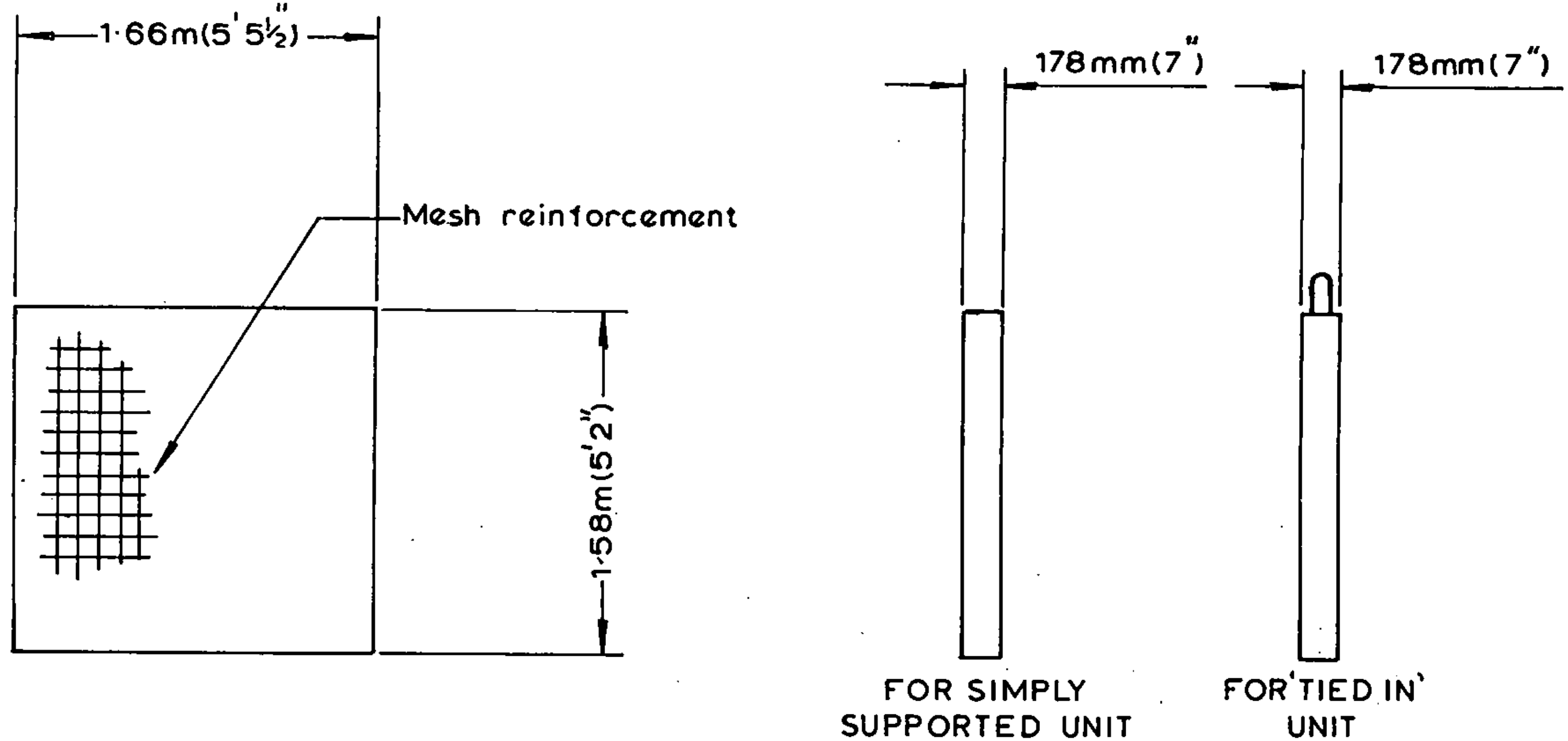

FIG.4 DETAIL OF WALL UNIT

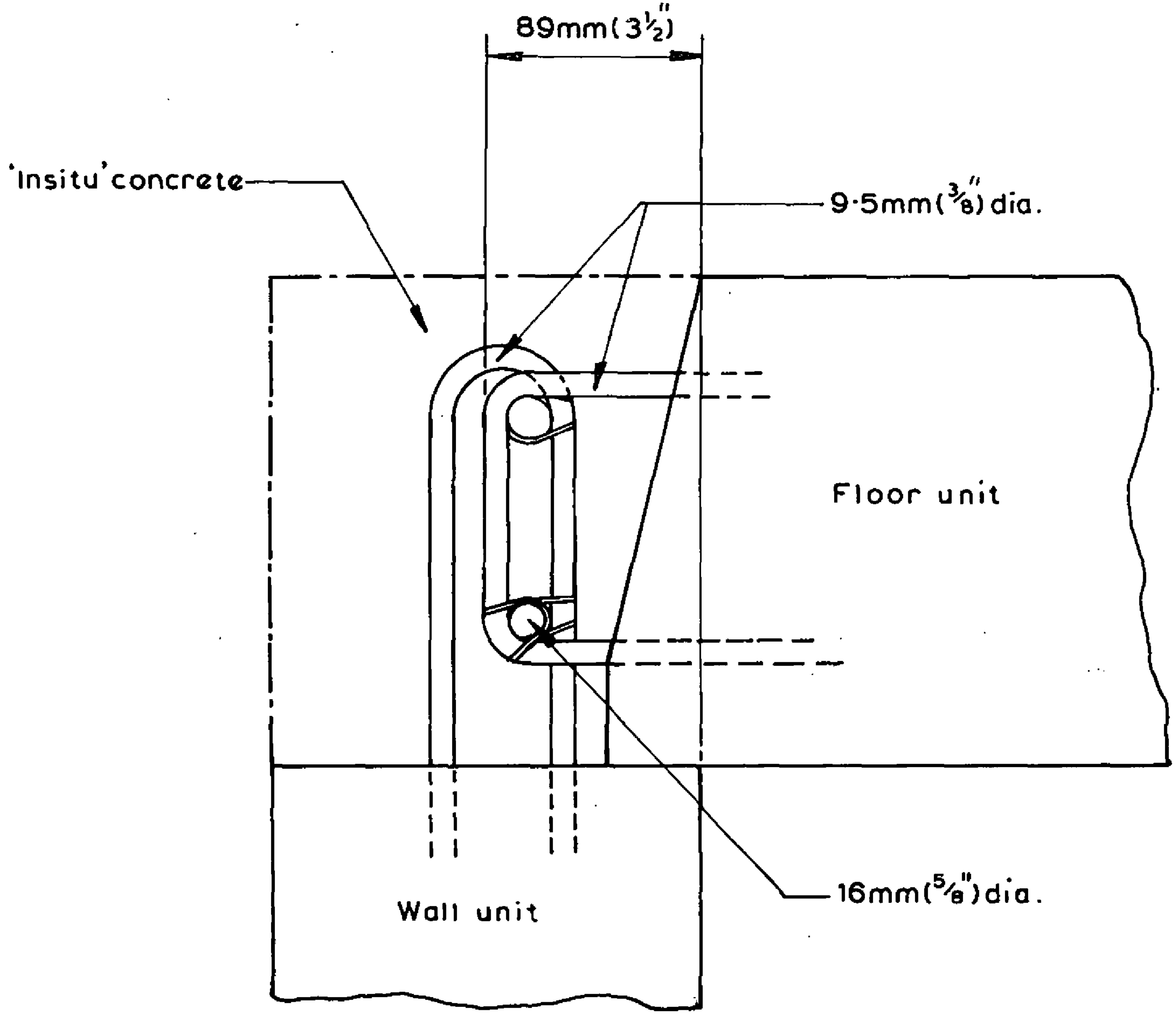

FIG.5 JUNCTION DETAIL—CONSTRUCTION 1

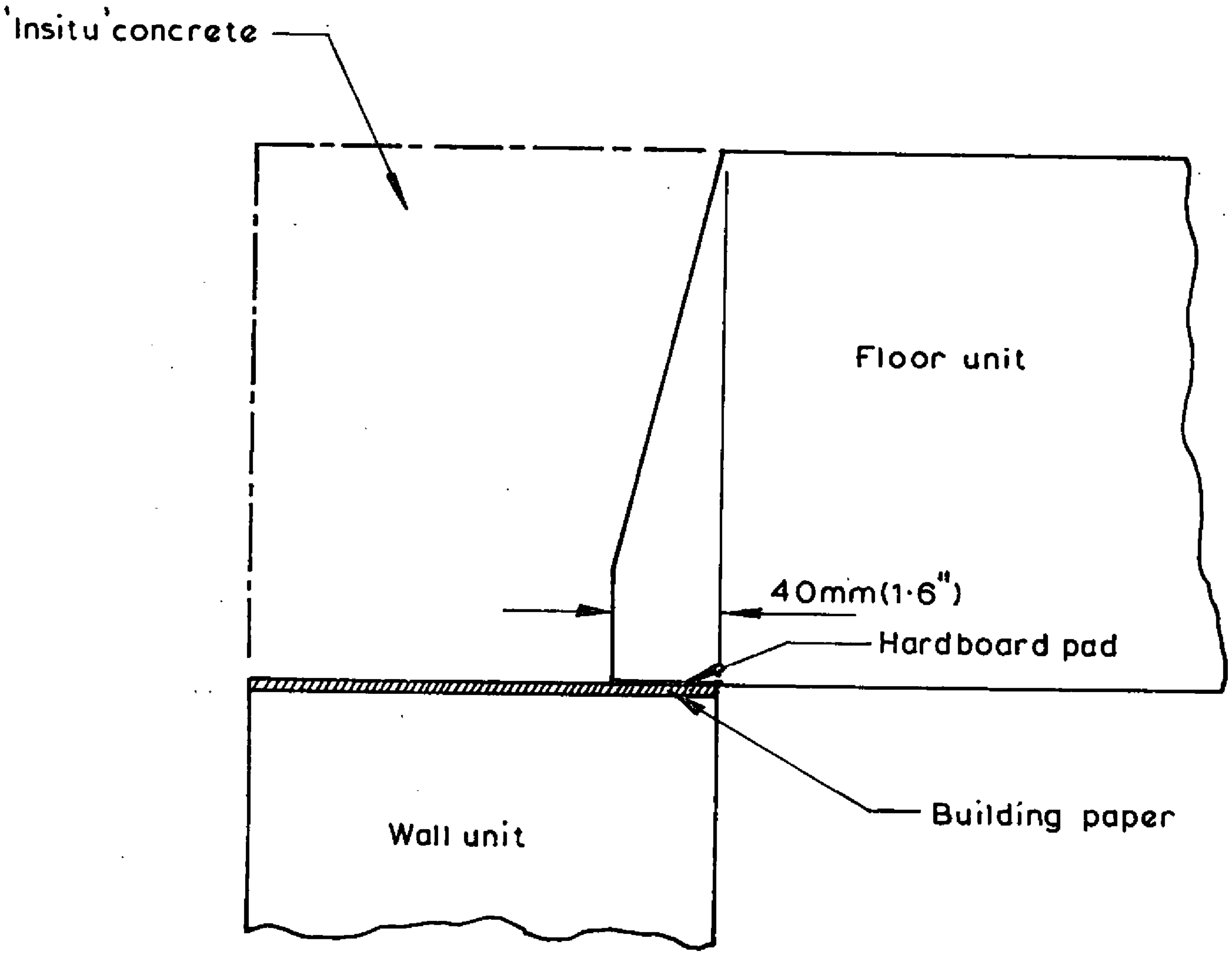

FIG. 6 JUNCTION DETAIL–CONSTRUCTION 2

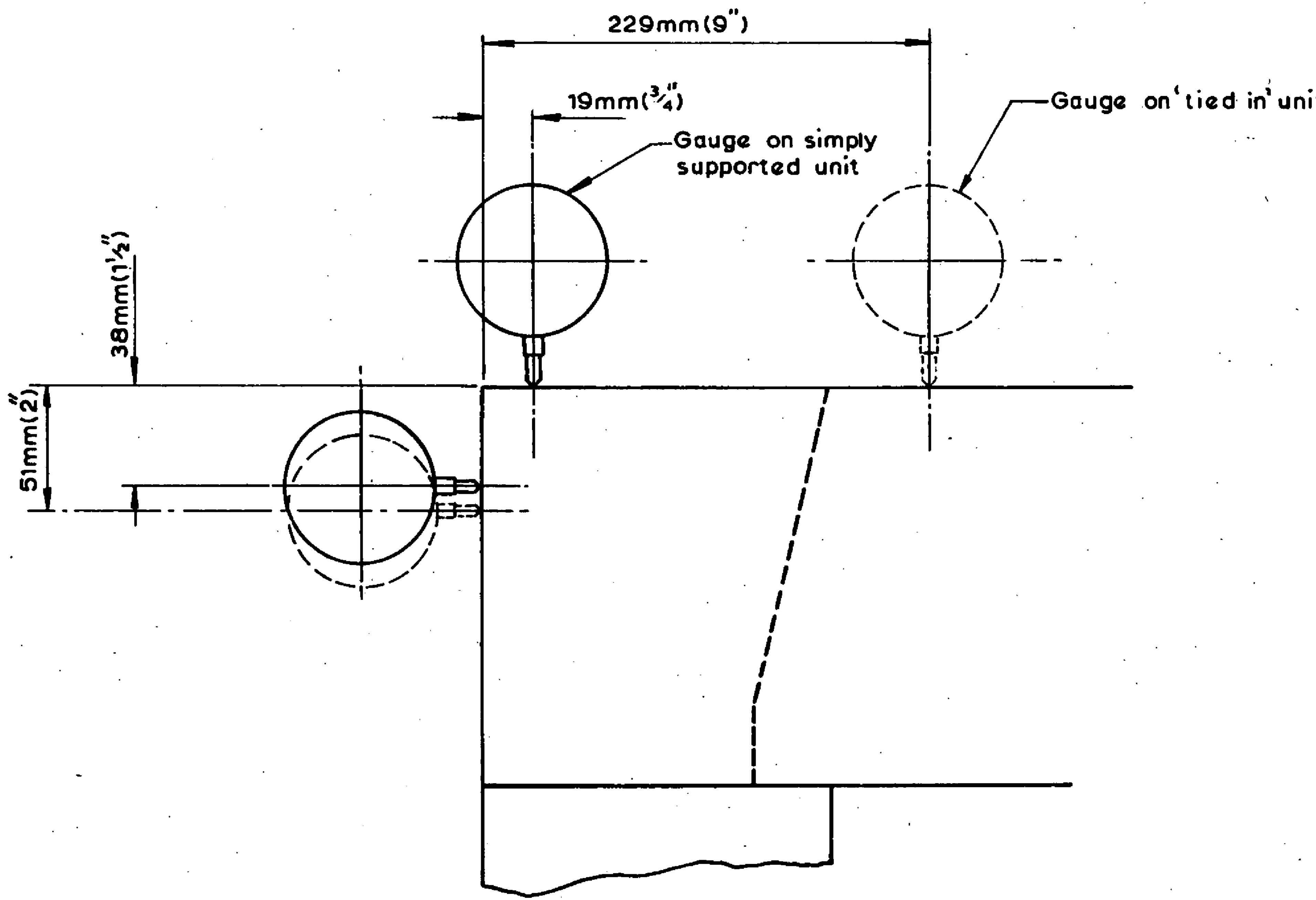

FIG.7 LOCATION OF DIAL GAUGES

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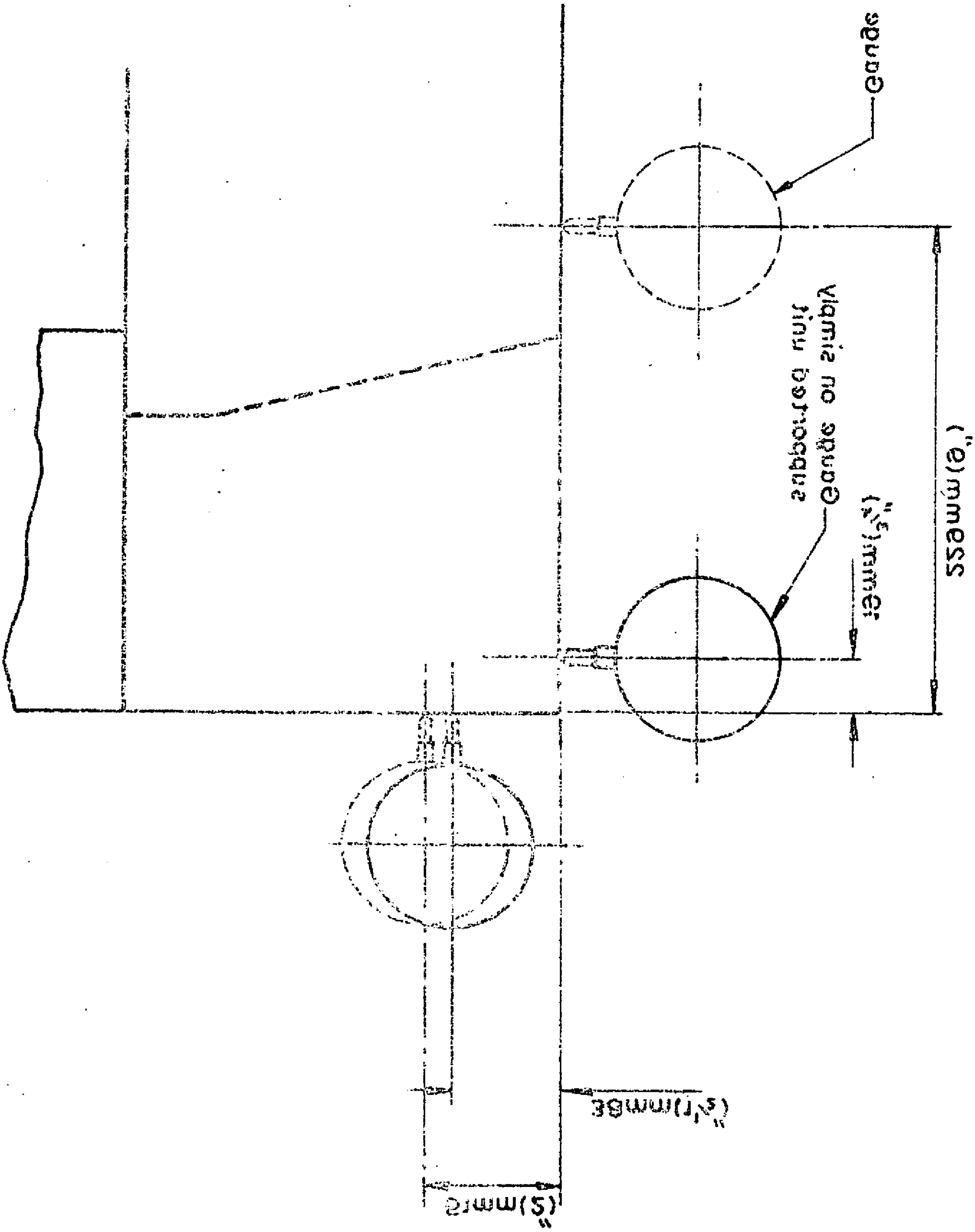

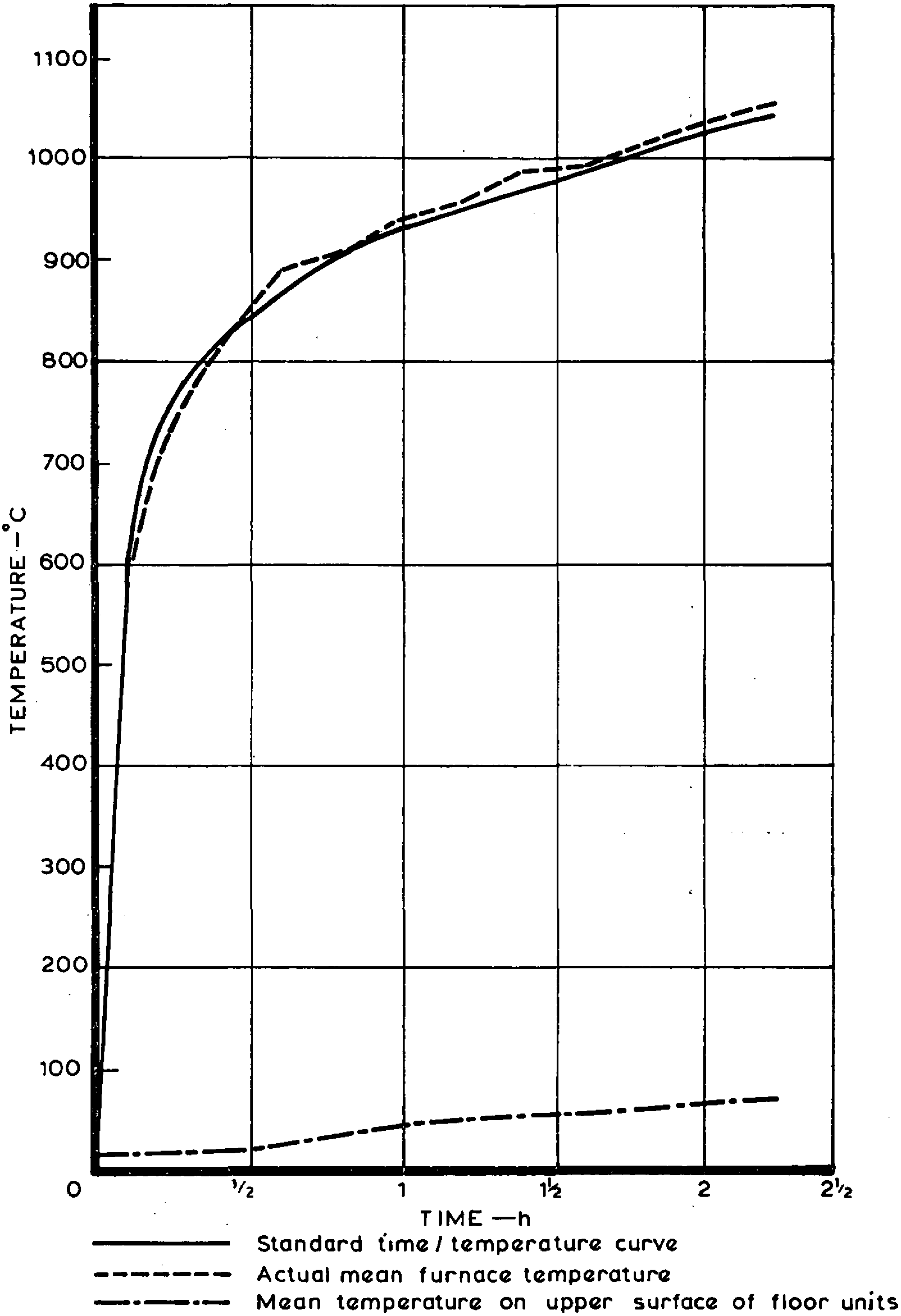

FIG. 8 TIME–TEMPERATURE CURVES

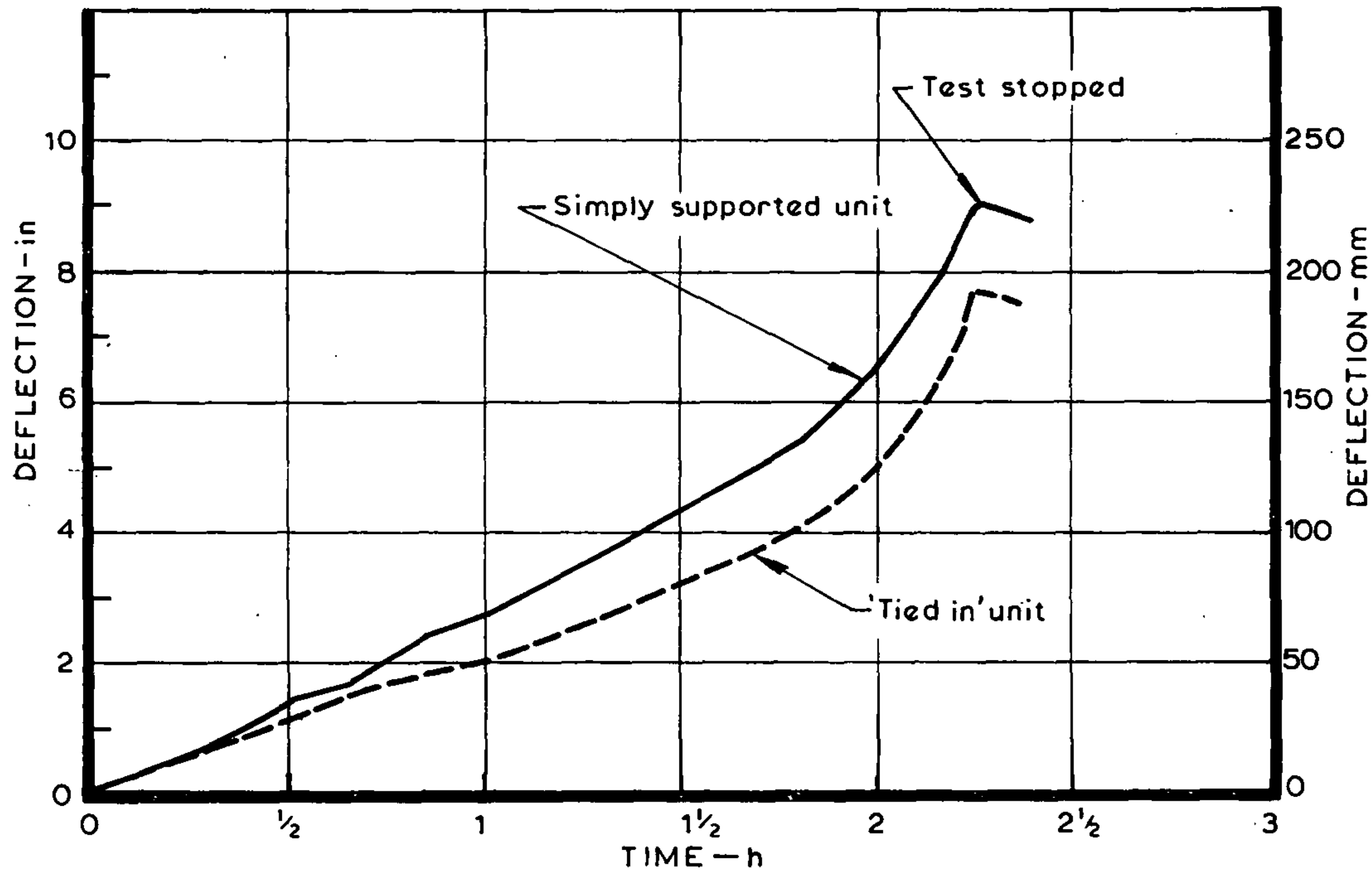

FIG.9 CENTRAL VERTICAL DEFLECTION

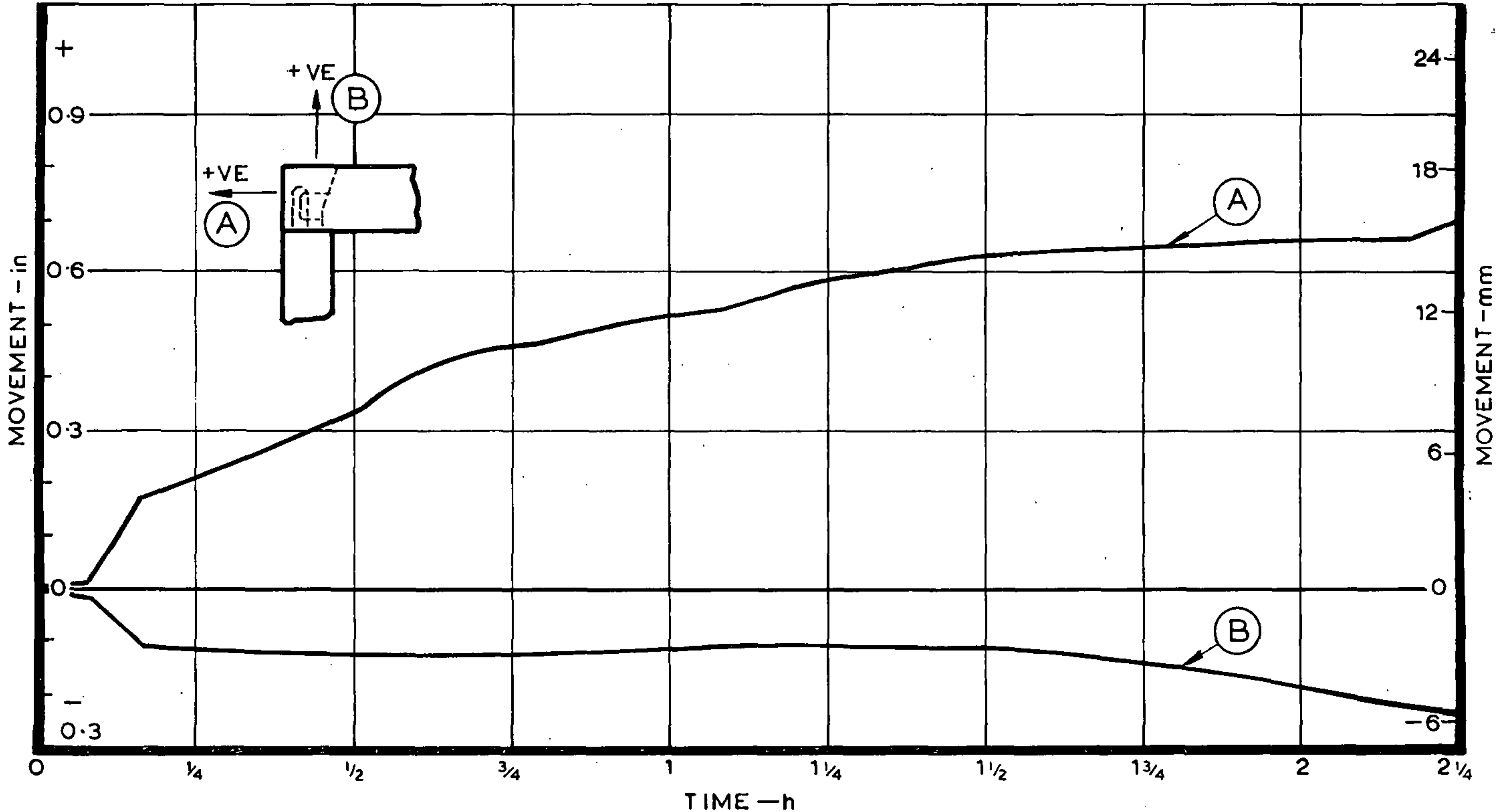

FIG.10 VERTICAL AND HORIZONTAL MOVEMENT–CONSTRUCTION 1

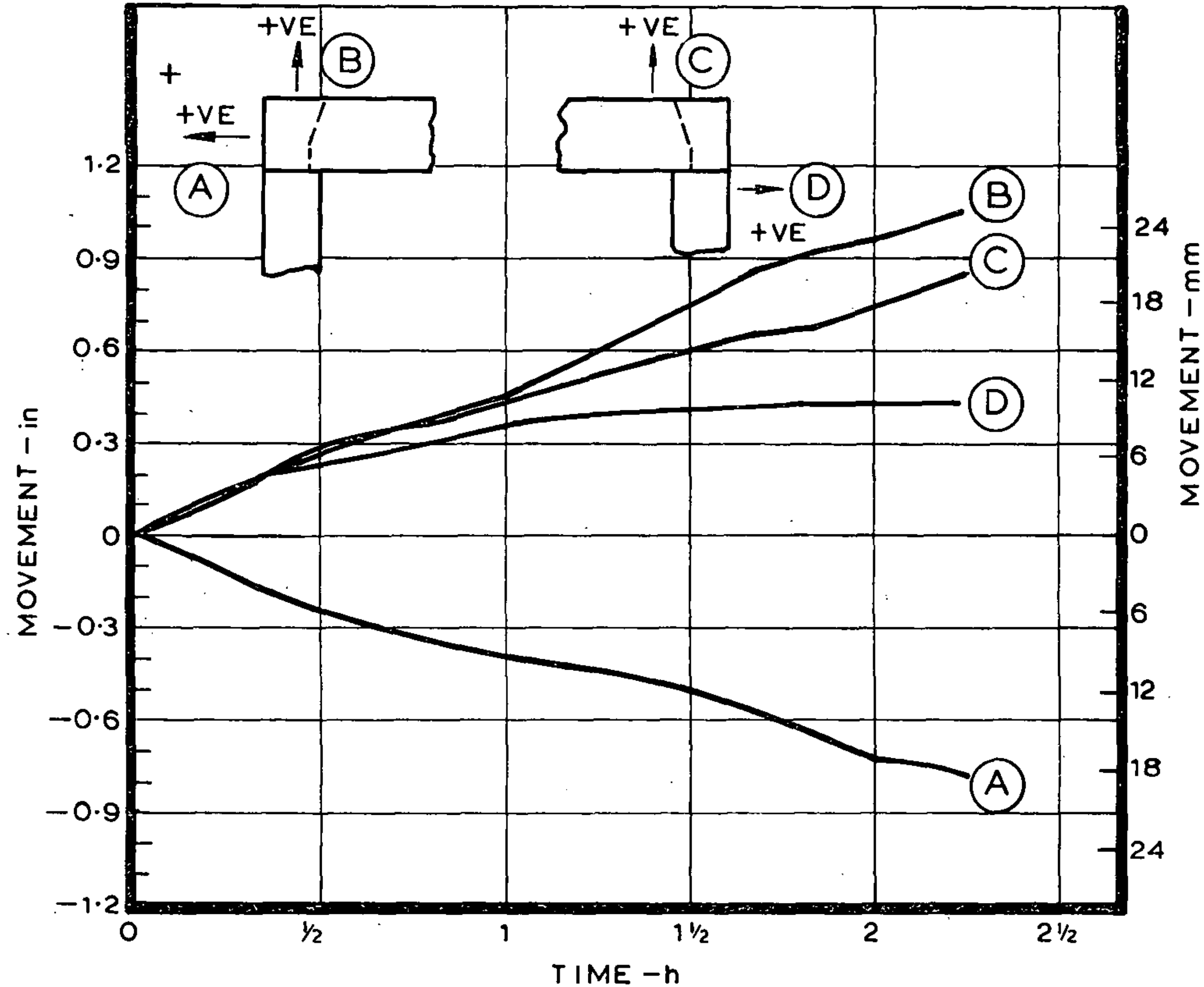

FIG.11 VERTICAL AND HORIZONTAL MOVEMENT — CONSTRUCTION 2

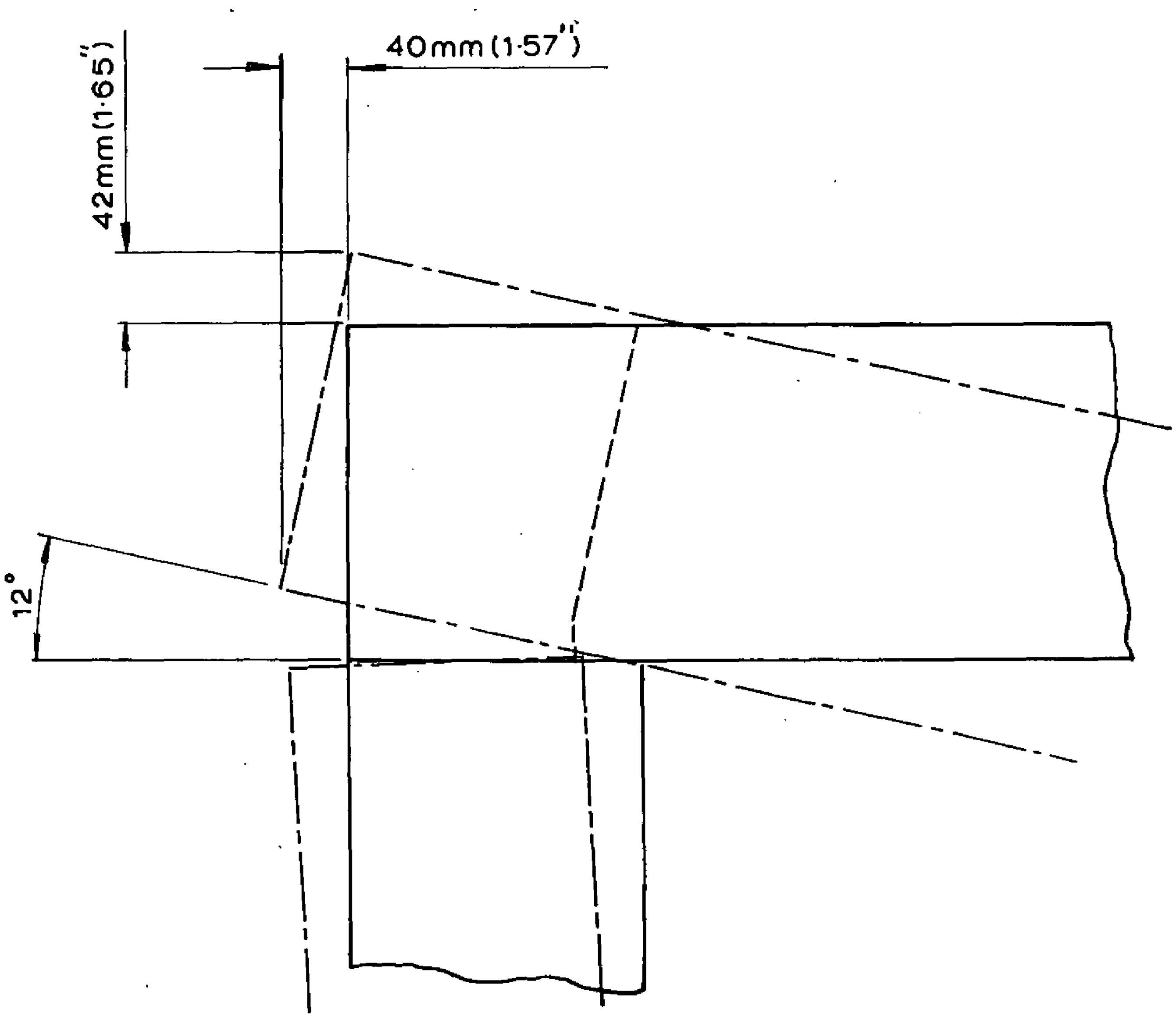

FIG.12 MOVEMENT OF FLOOR AND WALL UNIT–CONSTRUCTION 1

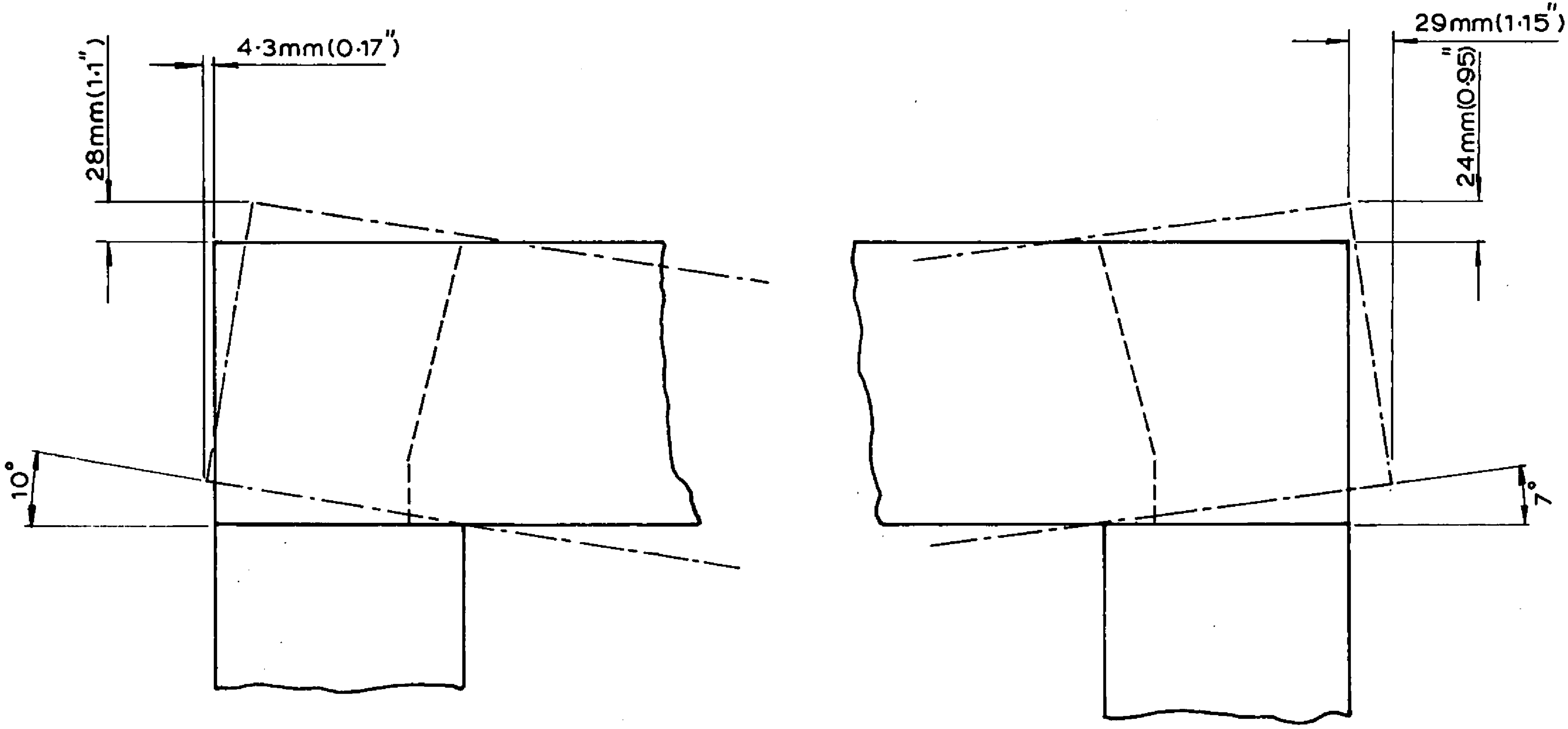

FIG.13 MOVEMENT OF FLOOR UNIT – CONSTRUCTION 2

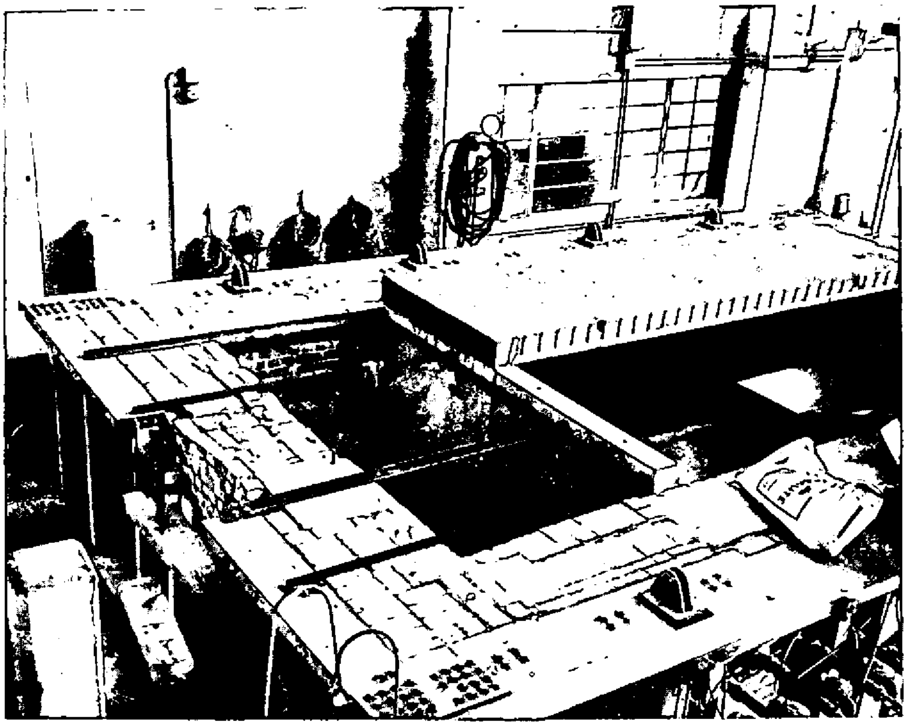

FIG. 14. CONSTRUCTION 1 COMPLETED IN FURNACE (STEEL RETAINING MEMBERS FOR WALL UNITS ON LEFT OF PHOTOGRAPH)

FIG. 15. CONSTRUCTIONS 1 & 2 FULLY LOADED PRIOR TO TEST

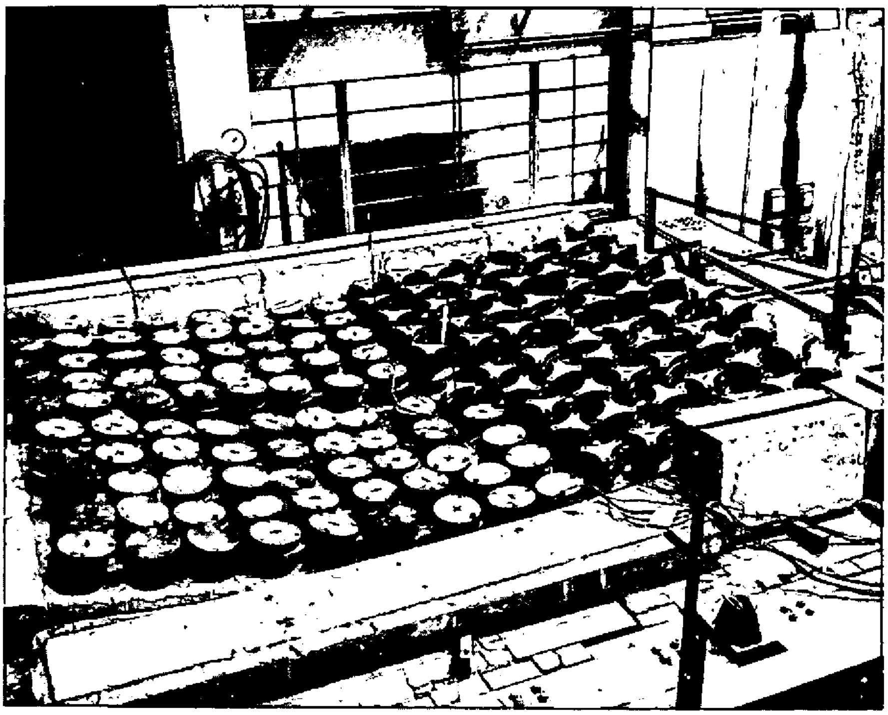

FIG. 16. IMMINENT COLLAPSE OF BOTH FLOOR UNITS

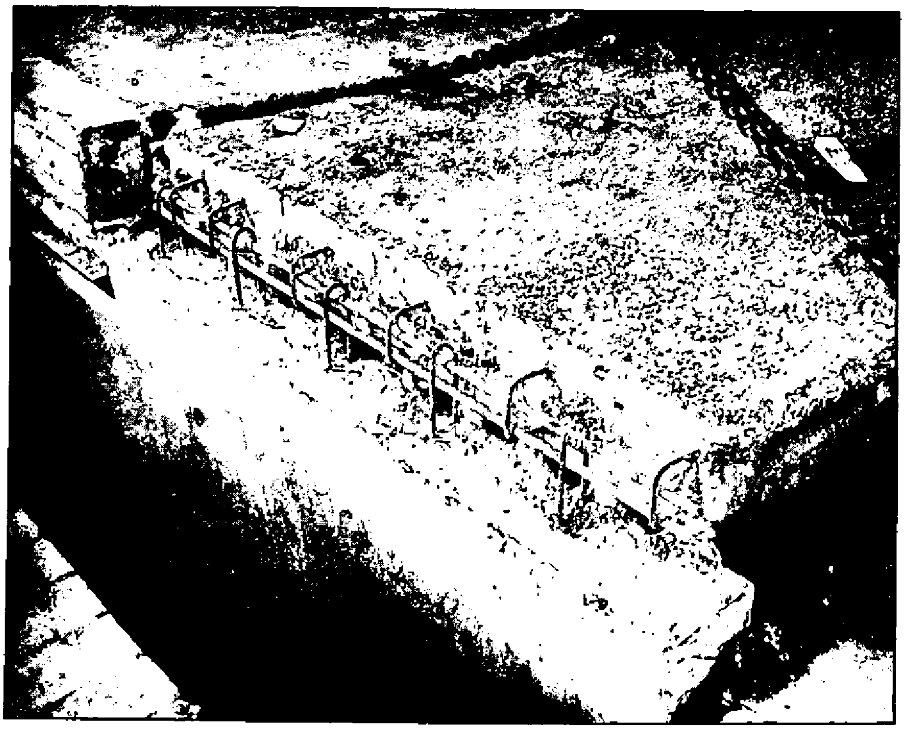

FIG. 17. DETAIL PHOTOGRAPH SHOWING BEARING EDGE DETAIL OF CONSTRUCTION 1 WITH IN SITU CONCRETE REMOVED AFTER TEST

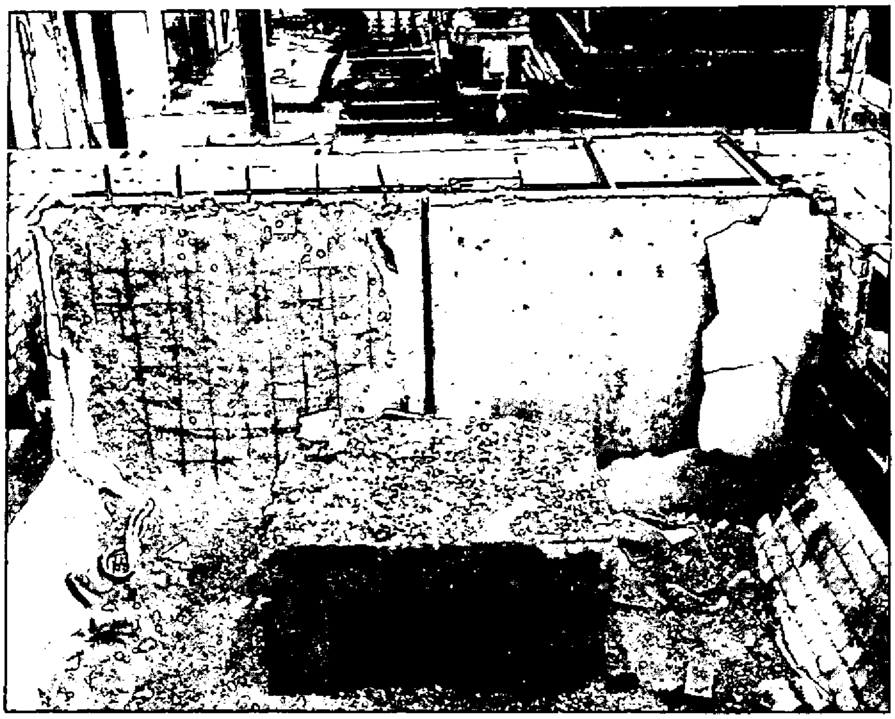

FIG. 18. APPEARANCE OF FACE OF WALL UNITS EXPOSED TO FIRE AFTER TEST (CONSTRUCTION 1 ON LEFT)

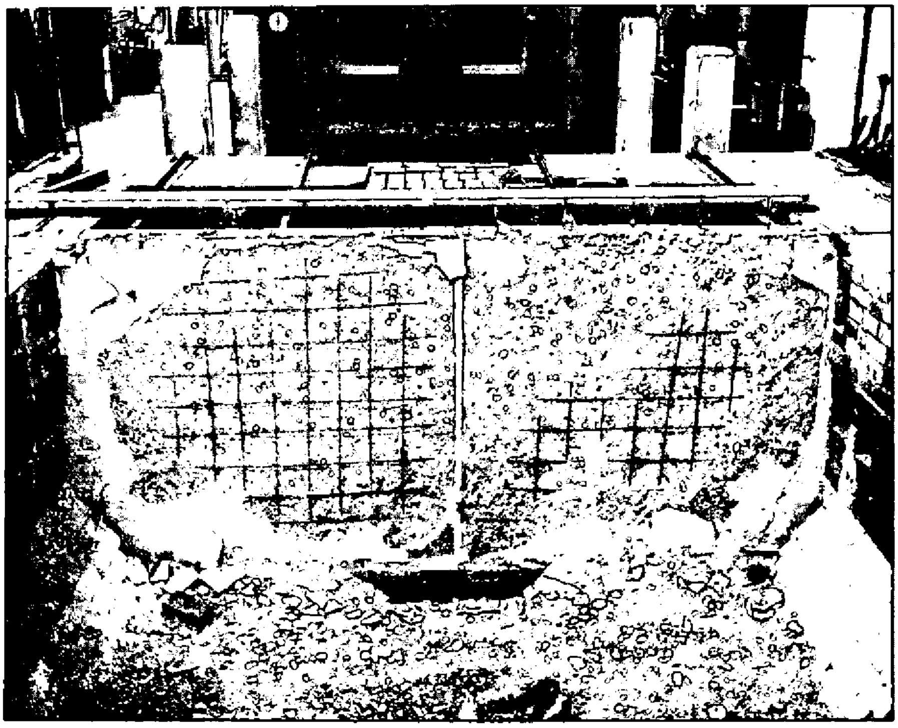

FIG. 19. APPEARANCE OF FACE OF WALL UNITS EXPOSED TO FIRE AFTER TEST (CONSTRUCTION 1 ON RIGHT)