NBS MONOGRAPH 45

Fire Tests of Precast Cellular Concrete Floors and Roofs

U.S. DEPARTMENT OF COMMERCE

NATIONAL BUREAU OF STANDARDS

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J. V. Ryan and E. W. Bender

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Contents

Fire Tests of Precast Cellular Concrete Floors and Roofs

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The results of an investigation of lightweight, precast cellular concrete planks are given. Fire tests were made of two floor and five roof specimens made up of these planks. Variables included density of the cellular concrete, thickness and span of the planks, reinforcement, and cover for the latter. A steel beam encased in blocks of cellular concrete was included in one floor specimen. The flexural strengths of 14 individual planks were determined. The investigation showed fire endurances up to 2 hr for 6-in. thick slabs tested under load and up to 4 hr for other slabs not loaded. Estimates were made of the probable results to be expected for slabs of thicknesses other than those actually tested.

1. Introduction

The framework and foundations of a building represent a large part of the total cost in multistory construction. They must carry both the live loads imposed by occupancy and the dead load of the finished building. Significant economies can be achieved in the cost of buildings by reducing the loads to be carried, and thereby permitting less expensive framework and foundation. Since reduction in allowable live load limits the usefulness of the building, research has been aimed at reducing the dead weight of building elements and materials.

Research aimed at the reduction of the dead weight of concrete structures led to development of lightweight aggregates as substitutes for dense natural aggregates, and to development of cell-forming processes for producing cellular concretes. The cell-forming processes included the use of aluminum powder and other gas-forming agents, the addition of preformed foam into the mixer, the addition of chemicals that form and retain air bubbles produced during mixing, and the use of excess water. Although known as "gas concrete," "foam concrete," or "aerated concrete," according to the particular process employed, all are included in the more general term "cellular concretes" [1].*

Cellular concretes have been developed that have densities as low as 20 lb/ft^3, are sawable, and workable with general carpentry tools. Such cellular concretes are being manufactured commercially and used to produce prefabricated blocks and panels for floor, roof, and wall assemblies.

The economies achieved through the use of cellular concretes are not limited to those associated with dead-weight reduction. Because of their cellular structure, they have significantly lower thermal conductivities than conventional dense concretes. Therefore, the use of cellular concretes often permits the elimination of additional insulating materials that would otherwise be necessary.

A research program was carried out on floor and roof slabs assembled from precast planks of a gas-formed cellular concrete to determine the fire endurances of representative specimens and the effects thereon of variables such as amount of cover for the reinforcing bars, overall thickness, amount and distribution of reinforcement, and density of the concrete. Five standard fire-endurance tests of roof assemblies and two of floor assemblies were conducted in a large floor furnace. Each specimen consisted of several planks grouted together. A steel I-beam, protected by cellular concrete blocks, was tested in conjunction with one of the floor specimens.

2. Test Specimens

2.1. General Description

The floor and roof panel specimens were made up of planks of cellular concrete specially cast in the necessary lengths for this study by a manufacturer. They were representative of commercially produced planks used in floor and roof construction. The mix constituents and ratios used in their production were not revealed. However, the particular cellular concrete is understood to be of the gas-formed type.

The planks were 18 in. wide and had a length of either 13 ft 5 in. (to span the short dimension of the furnace opening), or 17 ft 11 in. (to span the long dimension of the furnace opening). They were supplied in thicknesses of 5, 6, and 8 in., representative of the range of normal production. The nominal density of the planks for floor use was 44 lb/ft^3, and that of the roof planks was 31 lb/ft^3. The measured densities ranged from 42.6 to 46.5 lb/ft^3 for the floor planks, and from 35.8 to 42.2 lb/ft^3 for the roof planks.

For four of the seven standard tests, the specimen panels were made up of twelve 6-in.

*Figures in brackets indicate the literature references on page 11.

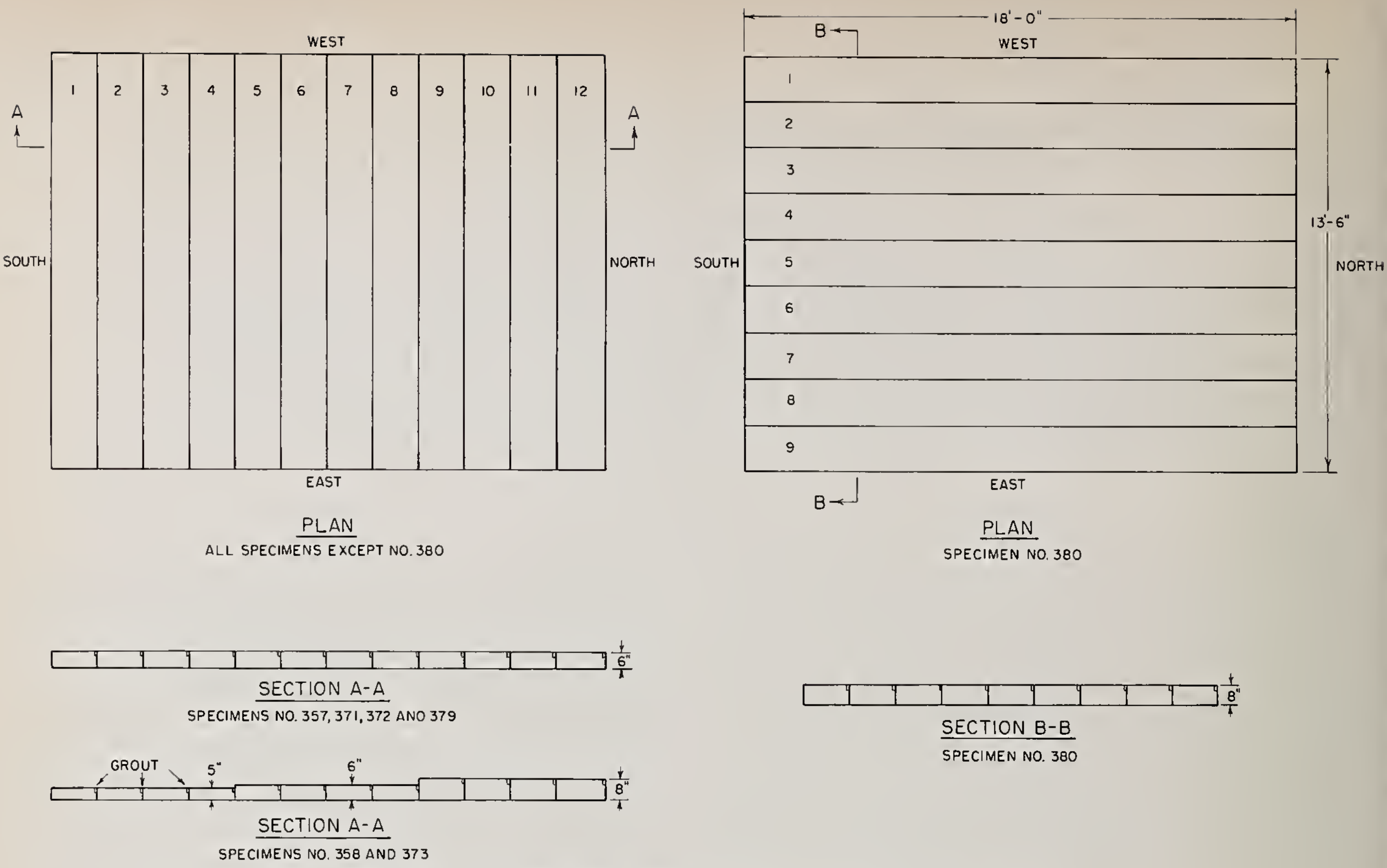


FIGURE 1. *Arrangement of planks within the furnace opening.*

The individual plank numbers are as used in section 8, appendix.

planks laid across the furnace. Three specimens of this group were roof panels and differed either in the reinforcement design of the planks or in the load applied during the test; the fourth was a floor panel. One other roof panel was made up of nine 8-in. planks laid along the length of the furnace opening. For the remaining two full-scale tests (one floor and one roof), four planks of each of the three thicknesses supplied, 5, 6, and 8 in., were laid across the furnace opening, with the 6-in. section in the center. Figure 1 shows the different assemblies.

After an exploratory test in a small furnace to determine the feasibility of the procedure, a structural steel **I**-beam protected by cellular concrete blocks was tested along with the floor panel of graduated thickness. The beam was a 10-in. **I**-beam weighing 25.4 lb/ft, of 17 ft $7\frac{3}{4}$ in. length to span the long dimension of the furnace. The floor panel (with planks crosswise of the furnace) rested on the top of the beam. The sides and bottom, or soffit, of the beam were encased in precast blocks to provide a cellular concrete cover of $2\frac{7}{8}$ in. on the bottom and $1\frac{15}{16}$ in. on the sides.

Flexural strength tests were made on several individual planks, which had not been exposed to fire.

2.2. Reinforcement and Cover

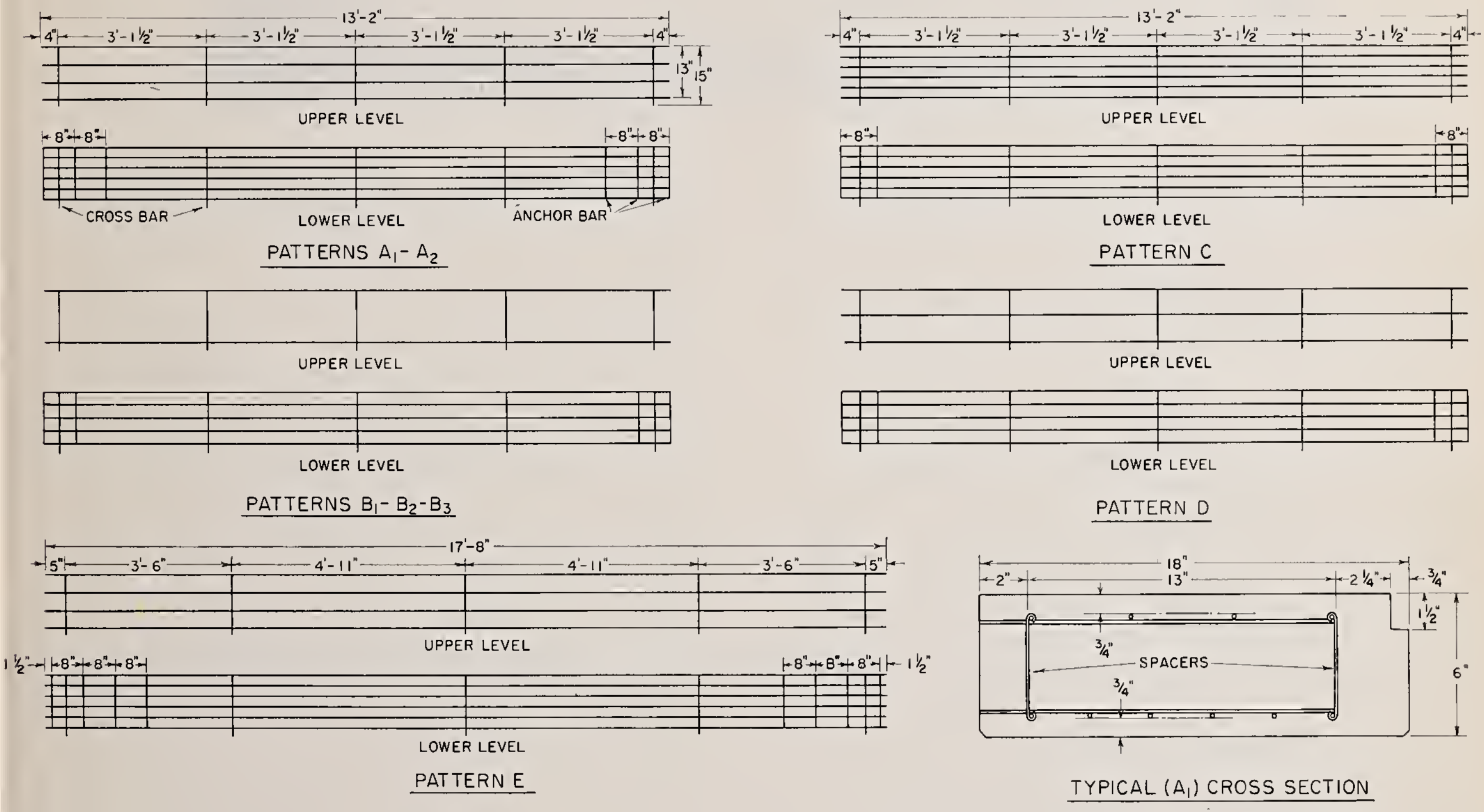
The term *cover* refers to the thickness of concrete between a steel reinforcing bar and the nearest surface of the reinforced concrete assembly, or to the thickness of protective material encasing a structural steel member. The cover of greatest interest in connection with reinforced concrete flexural members is that between the bottom tensile reinforcement and the surface exposed to fire.

The reinforcement in the specimens consisted of nominally round, hot-rolled, plain steel bars, in various patterns or combinations of number and size. The basic pattern of reinforcement consisted of longitudinal bars at two levels (one in the tensile zone and one in the compressive zone), plus crossbars, anchor bars, and spacers. The crossbars maintained the spacing between longitudinal bars at a single level and positioned them in the form. The anchor bars provided anchorage for the ends of the lower level, or tension bars. The spacers, which were formed of sheet metal, maintained the spacing between levels as determined by the overall thickness and cover. The longitudinal reinforcing bars were located and spaced so that those of the upper level had the same cover from the top surface as did those of the lower level from the bottom surface. The amount of cover was varied among the specimens. The variations within the basic pattern of reinforcement are illustrated in figure 2 and the cover for each specimen is given in table 1.

TABLE 1. *Summary of specimen details and results*

No.	Type	Thickness	Clear span		Reinforcement	Concrete			Applied load	Computed tensile steel stress	Time to initial end point	Limiting condition
						Cover	Density					
							Nominal	Measured				
		in.	*ft*	*in.*	[a]	*in.*	*lb/ft³*	*lb/ft³*	*lb/ft²*	*lb/in.²*	*hr: min*	
357	Roof	6	13	5	A_1	¾	31	35.8	74	17,800	0:57	Load failure.
					B_1	[b]1¼			74	37,300		
371	Roof	6	13	5	A_1	¾	31	35.8	56	14,300	0:53	Load failure.
379	Roof	6	13	5	[c]A_1	¾	31	42.2	56	14,900	1:29	Max surf. temp.
380	Roof	8	17	4	E	¾	31	36	70	16,100	1:13	Ignite waste.
358	Roof	5	13	5	B_2	1¾	31	36	None	----------	3:13	Ignite waste.
		6	13	5	B_1	1¾	31	36	None	----------	4:07	Max surf. temp.
		8	13	5	B_1	1¼	31	36	None	----------	----------	----------
372	Floor	6	13	5	A_2	¾	44	44.5	70	12,600	1:40	Max surf. temp.
373	Floor	5	13	5	D	1¾	44	46.5	None	----------	4:20	Max surf. temp.
		6	13	5	B_3	1¾	44	45.4	None	----------	----------	----------
		8	13	5	C	1¾	44	42.6	None	----------	----------	----------
373	I-Beam	9⅞	17	8	----------	[d]1¹⁵⁄₁₆ [d]2⅞	----------	36.7	[e]17,700	20,000	2:22	Load failure.

[a] See figure 2 for details of reinforcement.
[b] Following the test, the cover in three planks was found to have been 1¼ in., rather than the intended ¾ in.
[c] Contained shear reinforcement; see text for description.
[d] Thickness of side and soffit blocks used to protect steel beam.
[e] Load equally divided among four application points.



PATTERN	A_1	A_2	B_1	B_2	B_3	C	D	E
UPPER	¼	5⁄16	¼	5⁄16	¼	¼	¼	5⁄16
LOWER	5⁄16	⅜	¼	5⁄16	5⁄16	⅜	5⁄16	⅜
CROSS	¼	¼	¼	¼	3⁄16	3⁄16	3⁄16	5⁄16
ANCHOR	5⁄16	⅜	¼	5⁄16	5⁄16	⅜	5⁄16	5⁄16

NOMINAL DIAMETERS OF BARS IN INCHES

FIGURE 2. *Reinforcement details.*

In the planks used for one of the 6-in. roof panels (test No. 379), shear reinforcement was provided in addition to the reinforcement already described. The shear reinforcement consisted of ⅛-in.-diam rod bent in a multiple-V shape, each V section thus formed being about 4½ in. deep and 5¼ in. long. A continuous length of this shear reinforcement was placed along each side of the assembly of reinforcement bars and wire tied to the upper and lower bars nearest each side of the plank. The

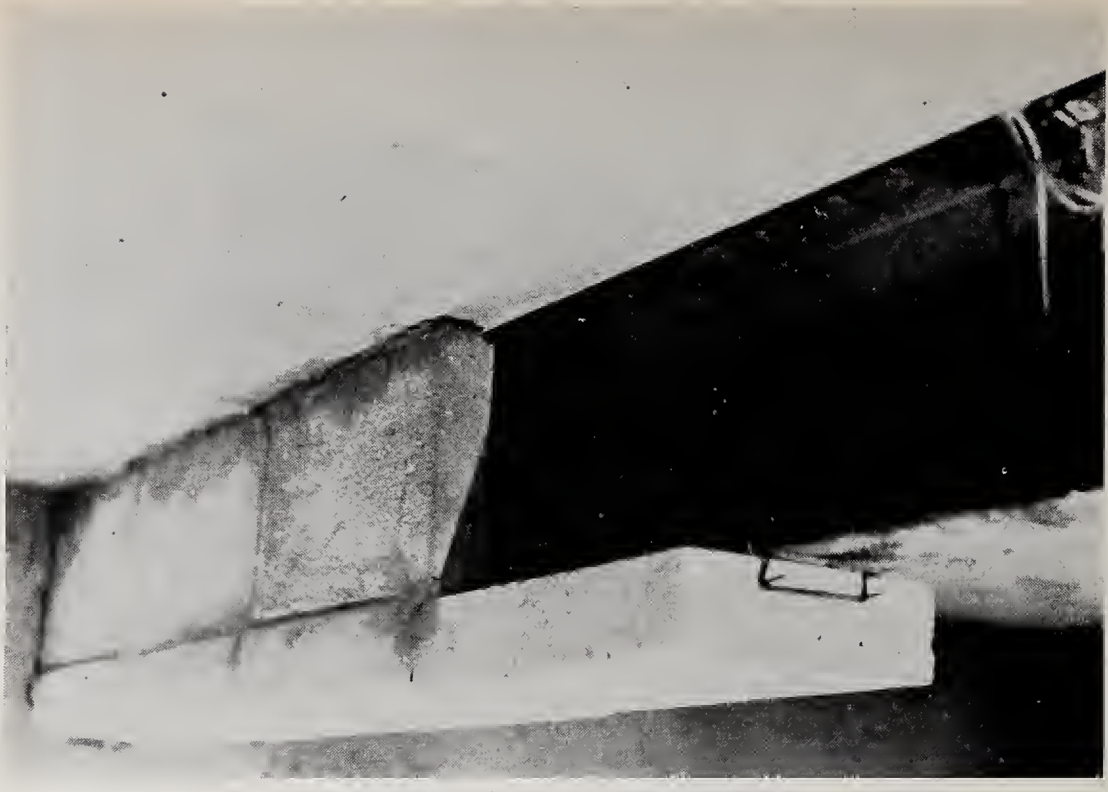


Figure 3. *Steel beam partly encased.*

ties were single strands of 0.027-in. wire and varied as to spacing, some being 5¼ in. and others 10½ in. apart. In these planks the sheet metal spacing clips were supplemented by ¼-in. diam steel rods welded to the upper and lower bars nearest the sides of the plank, 3 in. from their ends.

2.3. Construction of Specimens

In making up the test panels in the furnace, the planks were placed in contact with one another, and a grout consisting of 3 parts portland cement to 1 part sand, by weight, was poured between the panel and the furnace restraining frame and into the groove formed along each joint between planks by a kerf ¾ in. wide by 1½ in. deep along one edge of each plank. The kerfs and adjoining edges of the planks were wet down immediately before pouring the grout, and periodically for a week or ten days thereafter, to prevent unduly rapid drying of the grout, and thereby to improve the bond. An electric heater was placed in the furnace, and for the week immediately preceding each test a temperature of 125 to 150 °F was maintained in an attempt to remove excess moisture from the assembly.

The steel I-beam, tested in conjunction with the floor panel of graduated thickness, was placed along the longitudinal center line of the furnace and attached at both ends to steel plates by means of standard AISC B2 connectors [2]. These plates, in turn, were bolted to the furnace frame in such a way that the beam was supported, restrained against rotation of the ends, but could expand longitudinally. The beam was encased in precast blocks of cellular concrete, after the floor planks were in place. The soffit blocks (9 in. wide, 17$^{13}/_{16}$ in. long, and 2⅞ in. thick) were attached to the beam by means of formed wire clips. Transverse wires of the clips were driven into the ends of the blocks, holding them in good to fair contact with the beam. The soffit blocks extended about 2¼ in. beyond the beam flange on each side. The joints between blocks were filled with mortar consisting of 1 part lime, 1 part portland cement, and 5 parts crushed cellular concrete by volume. A bed of this mortar was placed on the upper surface of the soffit blocks, and the side blocks (9 in. wide, 18 in. long, and 1$^{15}/_{16}$ in. thick) were placed thereon. The vertical joints between blocks and the spaces between blocks and floor planks were filled with mortar. The joints were wetted before the application of the mortar and periodically thereafter for the next 10 days to prevent excessively rapid drying. Throughout the following week, an electric heater was kept in the furnace chamber and temperatures in the range of 125 to 150 °F were maintained in an attempt to remove some of the excess moisture. Figure 3 shows the beam partly encased.

3. Test Method and Equipment

The tests were conducted in accordance with the accepted standard test methods (ASTM designation E119) [3]. Not all the tests were stopped when the initial end-point criterion was reached, but some were continued in order to obtain more information about the behavior of the specimen.

This practice deviated from the standard test method, under which tests are stopped when the initial end point is reached and the specimens are subjected to the hose-stream and the double-reloading tests.[1] Inasmuch as the results would not have been indicative of the true performance of the specimens, because of their extended fire exposure, neither the hose-stream nor the double-reloading test was performed on any of the specimens. The fire-endurance limits were determined by the initial end point and are therefore in accordance with the standard test method, even though the fire exposure was continued.

The specimens were tested in the furnaces at the National Bureau of Standards designed for the purpose of testing floor and roof specimens. The small furnace, used for the exploratory test, took specimens 2 ft by 2 ft in size, while the large furnace, used for the standard tests, was constructed in such a manner as to accommodate a specimen 18 ft long by 13½ ft wide. Both furnaces were box-shaped, open at the top, so that the bottom face of the specimen was exposed to the flames.

To measure the temperature in the chamber of the large floor furnace, twelve thermocouples, encased in porcelain insulators and enclosed in iron pipes sealed at one end, were placed in the furnace so that the junctions of the thermocouples were approximately 1 ft below the exposed surface of the specimen. In the small furnace, four thermocouples were used. They were of the same type and were placed in the same manner as those in the large furnace. The furnace fires were controlled to approximate as nearly as feasible the temperatures set forth in the standard test method, which include 1,000 °F at 5 min, 1,300 °F at 10 min, 1,550 °F at 30 min, 1,700 °F at 1 hr, 1,850 °F at 2 hr, 2,000 °F at 4 hr, and 2,300 °F at 8 hr and longer.

The fire-exposure severity, which is defined as the ratio of the area under the curve of average furnace temperature to the area under the standard time temperature curve, is required to be between 90 and 110 percent for tests of 1 hr or less, between 92.5 and 107.5 percent for tests over 1 hr but not over 2 hr, and between 95 and 105 percent for tests longer than 2 hr.

In order to obtain temperature data on the unexposed surfaces of the specimens, a total of 12 chromel-alumel thermocouples whose junctions were in contact with the unexposed surface of the specimen were used in each test. They were distributed over the surface in a symmetrical pattern, with junctions and short lengths of the thermocouple wire coiled under standard felted asbestos pads 6 in. square by 0.4 in. thick.

Temperatures of the reinforcing bars in the specimens were measured by means of chromel-alumel thermocouples tied to the bars of some planks, at the time of fabrication. There were 5 thermocouples on the lower bars, and 3 on the upper bars, of each of the selected planks. Planks with thermocouples were located in positions 3, 6, and 9 (see fig. 1) of specimens 357, 371, and 372; positions 3, 5, and 7 of specimen 380; and positions 3, 4, 5, 7, 9, and 10 of specimens 358 and 373. No planks with thermocouples were provided for specimen 379.

The temperatures on the I-beams in the exploratory and full-scale tests were measured by chromel-alumel thermocouples which had their junctions peened in small holes drilled into the beams. Twenty-four thermocouples were used on the beam in the full-scale test (373). The thermocouples were located in four groups of six, the groups being spaced 42 to 43 in. on centers. Two of the thermocouples of each group on the beam were placed on the bottom of the top flange, one on each side of the web, and two on the top of the lower flange.

The recommendations of the manufacturer who produced the cellular concrete planks were followed in loading the specimens. The superimposed loads were applied by means of hydraulic jacks and were distributed evenly over the specimens. The manufacturer indicated that the "normal" loads were based on a design tensile stress of 18,000 lb/in.^{2} in the lower reinforcing steel. However, some specimens were tested at loads other than normal, to provide an indication of the effect of extent of loading on fire endurance. Values of the stresses in the lower reinforcing bars, as computed on the basis of simple beam theory, are given in table 1.

The four-point load applied to the steel I-beam was transmitted through four planks of the graduated floor specimen, thereby partially loading the latter. No other external load was applied to either of the graduated specimens.

The beam was loaded with a live load intended to produce a steel stress of 20,000 lb/in.^{2} The deflections were measured by a system of wires attached to the specimen and passed over pulleys to a scale, where riders on the wires indicated the amount of change from the initial level.

3.1. Moisture-Content Determinations

Samples were taken from the specimens in the furnace a day before each test in the study, except the first, and from extra planks. The samples were cylindrical cores, cut from the full thickness of the individual slabs. They were taken from near the corners of the specimens in the furnace. The holes were filled with the mixture used to grout the planks together.

Moisture determinations were made by drying specimens at 105 °C (221 °F). The specimens were the cylindrical cores in some cases, and thin wafers cut from the cores in others. Some were from planks as received, others as tested (after heating of the furnace chamber) and others from

[1] Both these supplementary tests have been deleted from the Standard E119 since the study reported herein was started.

planks stored at 50 percent relative humidity. Some of the specimens were conditioned in various ways, after removal from the planks, others were not. One group of wafer specimens were placed in 50 percent relative humidity after having been dried at 105 °C in order to permit measurement of the moisture regain. The results of these measurements are summarized in table 2.

It is doubtful that the moisture contents for the fire-test specimens can be related to the observed fire endurances, or to behavior in the fire tests. The moisture-content specimens were taken from near the edges of the fire-test specimens. Their moisture contents may therefore show somewhat higher values than those of planks nearer the center of the exposed area.

Table 2. *Moisture content data determined by oven-drying, at 105 °C for 24 to 72 hr, cores removed from planks heated after assembly into fire test specimens*

Test	Age	Duration of heating in furnace	Number of specimens	Mean moisture content by weight
	mos	*Days*		%
358	2½	7	1	5.4
371	7½	10	4	8.1
372	8	8	2	1.9
373	9	6	2	2.2
379	1½	7	2	14.6
380	10	6	2	2.5

Additional tests with cores, and thin wafers from cores, indicated equilibrium moisture content of about 2 percent when stored at 50 percent relative humidity, both for specimens as received and oven-dried. It required much more than 30 days for equilibrium to be reached throughout a 6-in. plank.

4. End-Point Criteria

The previously mentioned standard test methods state that the endurance of a specimen is the earliest time when any one of the following end-point criteria is reached or observed:

(a) The specimen shall no longer sustain its design load;

(b) The average temperature on the unexposed surface rises 250 °F above the initial temperature;

(c) The maximum one-point temperature on the unexposed surface rises 325 °F above the initial temperature;

(d) Cracks or openings shall develop in such a manner as to allow the passage of flames or gases hot enough to ignite cotton waste.

Although not included in the test methods as criteria for failure, the following data are considered to be of general interest and are tabulated, in part, in this report.

(e) The times at which steel structural members or reinforcement attained an average temperature of 1,000 °F at any appropriate level or section.

(f) The times at which steel structural members or reinforcement attained a temperature of 1,200 °F at any one point.

It should be noted here that while criteria (b), (c), and (d) are quite specific in defining failure, (a) is not, and it is sometimes difficult to judge when load failure occurs.

5. Results and Discussion

However, an objective method of determining this end point has been developed which defines a critical deflection and a critical rate of deflection. The method has been shown to give results reasonably consistent with those determined by an experienced operator in charge of test [4]. The critical deflection is defined as $D=L^2/800d$, and the critical rate of deflection as $R=L^2/150d$ per hour, where L is the span, or length of the specimen between supports, and d is the depth [2] of the specimen. The time when both of these critical values has been reached is reported in this study as the time of load failure.

The principal results of the tests made in the large furnace are given in table 1, and representative time-temperature and time-deflection curves are shown in figures 4 and 5. The observations of the behaviors of the individual specimens, made during the tests, are summarized in the test logs in the appendix. The results of flexural strength tests on individual planks are summarized in table 3.

[2] "Depth" is determined by the type of construction: for reinforced concrete slabs it is measured from the top of the slab to the bottom of the main reinforcement. This differs from "effective depth," which is measured to the centroid of tension reinforcement and which was used to compute the steel stresses given in table 1.

Table 3. *Results of flexural tests*

Individual planks placed on supports at 12 ft spacing and subjected to load at center only. The failures of the 13 ft 5 in. planks were judged due to yielding of the steel; those of the 17 ft 11 in. planks to shear in the concrete.

Thickness	Length	Reinf. pattern [a]	Nominal density of concrete	Test No. [b]	Max. load [c]	Deflection at max. load
in.	*ft:in.*		*lb/ft³*		*lb*	*in.*
6	13:5	B_1	31	358	2440	1.83
6	13:5	A_2	44	372	4275	1.90
8	13:5	B_1	31	358	1900	1.72
8	13:5	C	44	373	4590	1.40
8	17:11	E	31	380	5790	[d] 1.30

[a] See figure 2.
[b] Number of test in which other similar planks were exposed to fire.
[c] Mean of 2 to 5 specimens.
[d] Sudden failure, deflection reading not possible for 4 out of 5.

Of the five panels tested under load, the performances of two were limited by load failure, two by unexposed surface temperature rise, and one by ignition of cotton waste over a crack. The only criterion applicable to a beam tested under load is load failure; this was reached on the one I-beam tested. The two panels assembled of 5-, 6-, and 8-in. planks were not subject to the limiting condition of load failure; other limiting conditions were reached on the 5-in. section of each specimen and on the 6-in. section of one. For these two specimens, the attainment of the defined

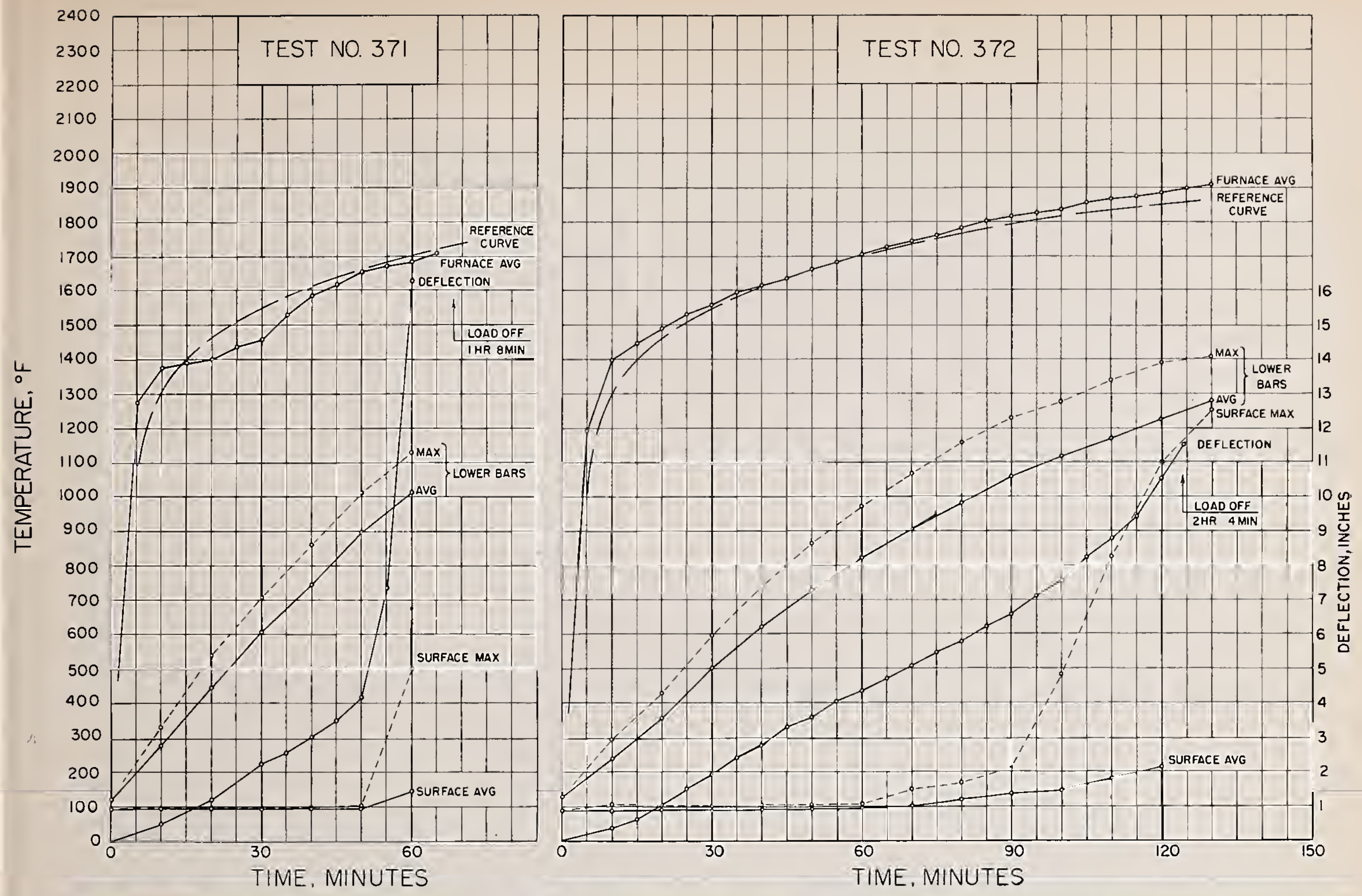


FIGURE 4. *Time-temperature and time-deflection curves for 6-in. roof (371) and floor (372) specimens.*

criteria was secondary in importance to the development of information on the effect of thickness. Although the ignition of cotton waste occurred in one of these tests, it is not considered useful data for comparisons.

During the test (373) of the floor without load, flames issued from the exposed surface of one of the 5-in. planks, possibly from burning of plastic tape used with the thermocouples. This was an exception to the general behavior observed in this study. The temperatures of the reinforcing bars in one of the 5-in. planks in this specimen rose much more rapidly than those for the other, presumably due to the flaming. Since the flaming was not representative, and in fact cannot be definitely explained, the data from this plank have been disregarded.

5.1. Plank Thickness and Cover

Each of the two graduated specimens, tests 358 and 373, provides a basis for comparisons of performance of planks of different thickness. Except for the 8-in. roof planks in test 358, all planks had the same cover. Within each test, all the individual planks were of the same nominal density There were differences in the number and size of reinforcing bars, but only those resulting from design on the basis of span, thickness, and other variables.

Among the roof planks (358), the limiting temperature rises on the unexposed surface were reached on the 5-in. planks at 3 hr 39 min for the average and 3 hr 37 min for the maximum, and at 4 hr 10 min and 4 hr 7 min respectively on the 6-in. planks, but were not reached on the 8-in. planks. Temperatures of 1,000 °F average and 1,200 °F maximum respectively were reached on the lower level reinforcing bars at 2 hr 9 min and 2 hr 14 min respectively for the 5-in. planks, and 2 hr 6 min and 2 hr 17 min for the 6-in. planks. The data for temperatures of the reinforcing bars in the 8-in. planks were not applicable in evaluation of the effect of thickness because the amount of cover used differed from that for the 5- and 6-in. planks.

In order to avoid complete collapse of the I-beam, test 373 was stopped immediately after the 3 hr 50 min data readings, although no limiting condition applicable to floor slabs had been attained. It was estimated by extraploation of the data obtained that the limiting maximum temperature rise would have been reached on the 5-in. planks at 4 hr 20 min. Temperatures of 1,000 °F average and 1,200 °F maximum were reached at 2 hr 50 min and 3 hr 6 min respectively on the

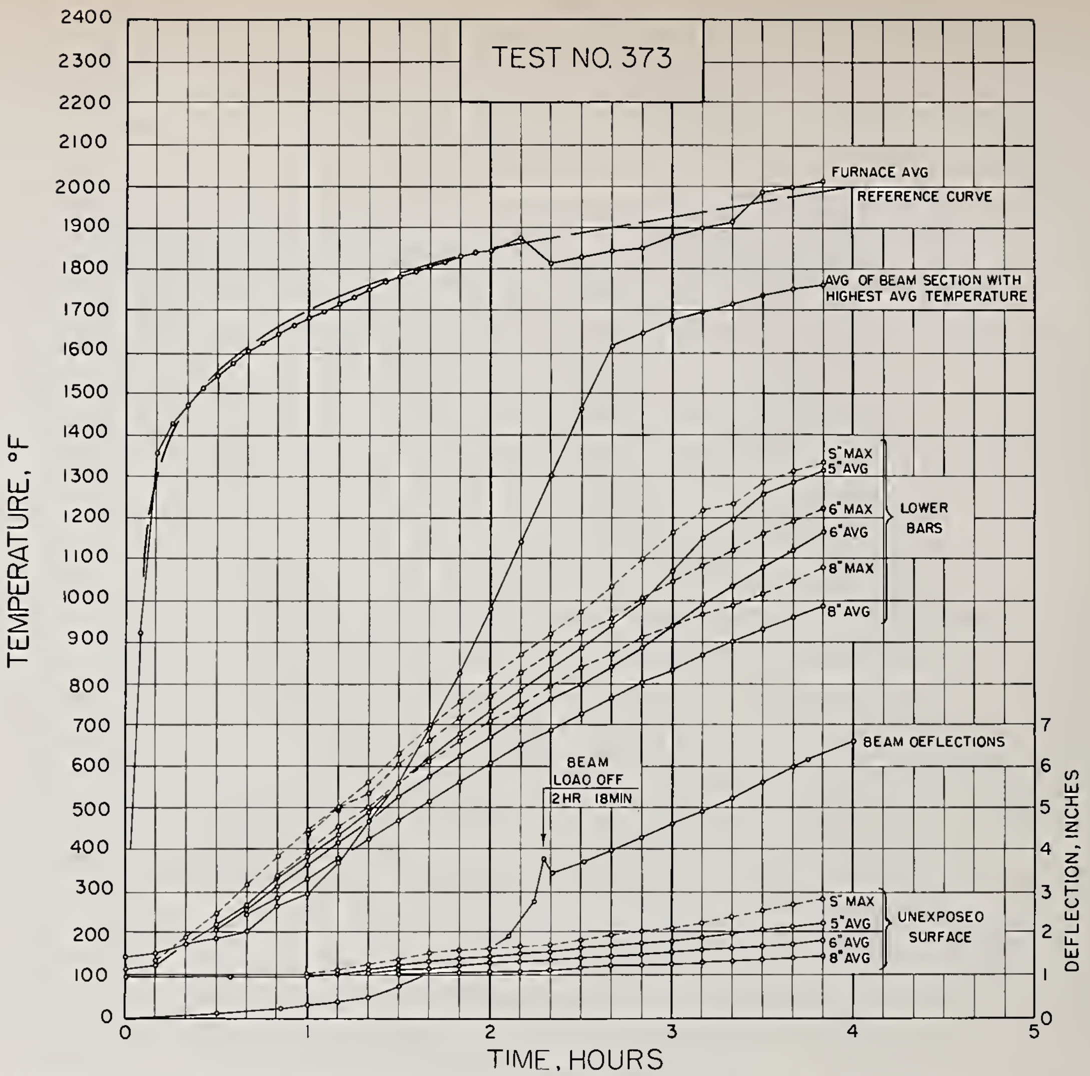


FIGURE 5. *Time-temperature and time-deflection curves for steel beam and floor slab made up of 5-, 6-, and 8-inch thick planks.*

The load was concentrated on planks directly over the beam and was removed when the latter failed.

lower-level reinforcing bars of the 5-in. planks, and at 3 hr 12 min and 3 hr 40 min for the 6-in. planks. It was estimated that the 1,000 °F average would have been reached at 3 hr 52 min for the 8-in. planks; the 1,200 °F maximum was not reached.

The steel temperatures chosen are generally associated with loss of steel strenghth such that load failure is imminent for reinforced concrete structural elements designed with a safety factor of two for the reinforcing bars. Since neither of the two graduated specimens were subjected to loads appropriate to the plank designs, the times at which the reinforcing bar temperatures reached 1,000 °F average and 1,200 °F maximum may differ appreciably from those that would have been observed in tests of similar planks loaded in accordance with accepted design practices. Under such load the development of cracks and the sloughing or spalling of concrete cover may differ from and lead to earlier and more rapid steel-temperature rise than would be observed under comparatively light load. However, the times observed in these tests should be valid indications of the relative performances to be expected for the various thicknesses.

The effect of cover was not easy to evaluate because all specimens with ¾-in. cover were tested under load and all those with 1¾-in. cover were tested without load. However, it was discovered after test 357 that three planks with 1¼-in. cover had been supplied inadvertently along with the nine planks with the intended ¾-in. cover. Moreover, the former had only half as much reinforcement as the latter. Therefore, another test, 371, was conducted essentially duplicating the earlier specimen except that all planks had ¾-in. cover and the applied load was reduced by 25 percent. Despite the reduced load in the latter test, the specimen failed under load at 53 min, compared with 57 min in the earlier test. Therefore the greater cover gave appreciably longer fire endurance to the assembly.

The method of varying cover by changing the effective depth of the reinforcing bars would change the allowable load, the steel stress, or require a change in the amount of reinforcing steel. Ordinarily all of these would be held constant and change of cover would be accomplished by change of overall thickness. The unusual procedure in this study resulted in the effect of cover variation's not being masked by the effect of thickness.

5.2. Reinforcement

Longitudinal reinforcing bars having nominal diameters of 1/4, 5/16, and 3/8 in. were used in the specimens. Shear reinforcement was also used in one specimen. For the most part, the number and size of reinforcing bars incorporated in the test specimens were such as to satisfy design criteria related to span, load, and stress in the steel. In general, the number and size of bars were the same among planks having the same thickness, span, concrete density, and cover for the bars. However, there was an exception to this general rule, as can be seen from comparison of reinforcement in specimens 372 and 379. The spans, thickness, concrete density, number of reinforcing bars, and cover were equal (within practical limits) for these specimens, but the former had 3/8-in. lower level and 5/16-in. upper-level bars, whereas the latter had 5/16-in. lower and 1/4-in. upper bars plus full-length shear reinforcement. The specimen in test 372 was subjected to an applied load equal to 1 1/4 that applied in test 379. The initial limiting criterion in each test was unexposed surface temperature rise, that in test 372 having been reached at 1 hr 40 min and that in test 379 at 1 hr 30 min.

The deflection data would be more indicative of the effect of variation of the reinforcement than would be the surface-temperature data. Unfortunately, these tests were not continued to load failure. The available deflection data do indicate that load failure, for the specimen in test 372, would have been about 40 min later than that for the specimen in test 379. From this, it appears that the 1/16-in. increase in bar diameter was much more effective than the addition of shear reinforcement.

5.3. Concrete Density

The nominal density of the cellular concrete specified for floors was 44 lb/ft^3, whereas that for roofs was 31 lb/ft^3. The actual densities of the concretes as received were obtained from full-sized planks, with correction for the weight and volume of the reinforcement, or from cores removed from planks; these were within ± 2.5 lb/ft^3 of the nominal densities for the floor planks and were about 5 lb/ft^3 high for the roof planks, with one exception (test 379). In this specimen the average density of the cellular concrete, based on the total weight of two extra planks, was found to be 42.2 lb/ft^3 although they were marked as roof planks. Even after correction for moisture content, the densities of the roof planks were in the range of 33 to 37 lb/ft^3.

The specimens of tests 371 and 379 were very similar, differing only as to concrete density, and the presence of shear reinforcement in the latter. Each was subjected to the same applied load. Each was limited by temperature rise on the unexposed surface, reached at 1 hr in test 371 (density 35.8 lb/ft^3) and at 1 hr 30 min in test 379 (density 42.2 lb/ft^3). The times at which 1,000 °F average and 1,200 °F maximum temperatures were reached on the lower level reinforcing bars were 59 min and 1 hr 4 min, respectively, in the former test, and over 1 hr 30 min in the latter. Similar comparisons of the effect of concrete density may be made from the unexposed surface temperature data of the 5-in. planks in tests 358 and 373 as well as from the temperature data for the reinforcing bars of the 5-in. and 6-in. planks of the same tests.

These data indicate that, within the ranges of density and other variables covered in this study, the cellular concrete will provide increased resistance to heat penetration at higher densities.

Such behavior agrees with that to be expected if the thermal diffusivity of the concrete decreased as the density increased. Thermal diffusivity α is defined as $\alpha=k/\rho c$, where k is thermal conductivity, ρ is density, and c is specific heat. Since the cellulur concretes of various densities may be presumed not to vary greatly as to chemical composition, the values of specific heat should be reasonably constant. Although the values of thermal conductivity tend to increase with density, apparently the net effect for the cellular concrete in the particular range of densities investigated is one of lower diffusivity for that of high density than for that of low density. This is in agreement with previously published data [5]. However, the same data indicate that this trend is reversed at densities of about 60 lb/ft^3 and that diffusivity increases with increased density above that level.

5.4. Estimates of Fire Endurance

Data from standard fire tests are often used as the basis for estimates of the probable fire endurances of similar constructions not actually tested. The usefulness of such estimates is dependent on the amount of test data available on similar constructions, the simplicity of the basic construction, and the degree of similarity between the constructions actually tested and those for which estimates are made. Another important factor is the basis for the method employed in making the estimates.

A method has been developed on the basis of many years of experience in tests of solid, essentially homogeneous walls [6]. It relates fire endurance to the 5/3 power of the thickness. Since the cellular concrete floors and roofs approximated homogeneity very closely, it was believed that a similar relationship might hold for them, although probably with the thickness raised to a different power than that for walls. A plot of the data confirmed this assumption. Data for load failure, unexposed surface temperature limiting rise, and 1,000 °F average and 1,200 °F maximum temperatures on the lower-level reinforcement were plotted and lines faired through the points. The exponents for the various lines were in the range 0.64 to 0.80.

Specimens were tested at 5-, 6-, and 8-in. thicknesses. The lines developed from the data plots were extended from 3 to 10 in., representing extrapolation over 2 in. of thickness at each end. The times at which the aforementioned load and temperature conditions might be expected were picked off the lines and are given in table 4. This table gives only the values from the lines; hence the values at 5, 6, and 8 in. do not agree in all cases with those given in table 1, the actual values for the particular specimens tested. The data from table 4 were analyzed to determine which failure criterion would be reached first; the resulting estimated fire endurances are given in table 5. Although fire endurance ordinarily is stated to the nearest minute, the estimates obtained from table 4 were rounded off to the nearest 5 min up to 2 hr and to the nearest 10 min for longer times. This was done because the uncertainties in the estimates in table 5, due to the combined effects of uncertainty in each data point, in fairing the lines through the data, and in rounding off the figures, are dependent on the extent of extrapolation and the time. An estimate of about 6 hr for a 10-in. nonloaded roof slab is probably within 30 min of the true value whereas for a 10-in. loaded roof slab the estimate of 1 hr 30 min is probably within 10 min. For 3-in. slabs, the uncertainty would be about 10 min in 2 hr and 5 min in 30 min.

The times at which the high temperatures of the reinforcing bars were reached were not considered in developing table 5, since the reinforcement temperature is not a defined limiting condition in the standard test method [3]. Since no specimens with 1¾-in. cover were tested under load, load-failure estimates were not made for them in table 4. However, the tabulated estimates for 1,000 °F average and 1,200 °F maximum may be taken as rough indications of when such failure might occur, provided that the specimens were loaded to develop the design stress in the steel.

The ignition of cotton waste on the unexposed surface is an applicable end point, and occurred twice in the study reported herein. It always occurs at a local failure, such as a crack or spall. Such local failures often indicate a point not representative of the overall specimen; therefore they are difficult to predict. Consequently, no attempt was made to estimate times for this failure in drawing up tables 4 and 5.

TABLE 4. *Estimated times to various conditions*

The estimates were obtained from lines faired through data in the range of 5 to 8 in. and extrapolated at both ends.

Overall thickness	Unexposed surf. temperature rise		Load failure	Lower reinf. bar temperature	
	250 avg	325 max		1,000 avg	1,200 max
Roofs, loaded, ¾ in. cover					
in.	*hr : min*	*hr : min*	*hr : min*	*hr : min*	*hr : min*
3	0:50	0:42	0:36	0:36	0:37
4	1:01	0:52	0:44	0:44	0:45
5	1:11	1:00	0:51	0:51	0:52
6	1:20	1:08	0:58	0:58	0:59
7	1:29	1:16	1:04	1:04	1:05
8	1:37	1:23	1:10	1:10	1:11
9	1:45	1:30	1:16	1:16	1:17
10	1:53	1:36	1:22	1:22	1:22
Roofs, nonloaded, 1¾ in. cover					
3	2:35	2:33	----------	1:27	1:30
4	3:08	3:06	----------	1:45	1:50
5	3:39	3:37	----------	2:02	2:09
6	4:08	4:06	----------	2:18	2:26
7	4:35	4:33	----------	2:33	2:43
8	5:01	4:58	----------	2:47	2:59
9	5:26	5:23	----------	3:01	3:15
10	5:50	5:47	----------	3:14	3:30

TABLE 5. *Estimated fire endurances of cellular concrete roofs*

These were derived from table 4, rounding off as described in the text.

Thickness	Loaded [1]		Nonloaded [2]	
	Time	Limiting criterion	Time	Limiting [3] criterion
in.	*h : m*		*h : m*	
3	0:35	Load	2:30	ST
4	0:45	Load	3:10	ST
5	0:50	Load	3:40	ST
6	1:00	Load	4:10	ST
7	1:05	Load	4:30	325
8	1:10	Load	5:00	ST
9	1:15	Load	5:20	325
10	1:20	Load	5:50	ST

[1] With ¾-in. cover, reinforcement appropriate for span, and load based on design stress of 15,000 lb/in.^{2} for roofs.

[2] With 1¾-in. cover, reinforcement at least 60 percent of that design live load of 56 lb/ft^2.

[3] Limiting conditions: Load=load failure, 250=average surface temperature rise, 325=maximum surface temperature rise; ST=both 250 and 325 degree temperature rises at same time.

Practical considerations made it obvious that estimates on nonloaded floors were of no value, and might be misleading. Only one floor specimen was tested under load and this was not deemed an adequate basis for estimates. Therefore, tables 4 and 5 were confined to estimates on roofs.

6. Conclusions

The results of the tests clearly indicate that cellular concrete slabs, made up of precast planks similar to those employed in this study, can be designed to provide fire endurances of 1 to 2 hr, and probably up to 4 hr.

The use of greater cover for the reinforcing bars, in the range of ¾ to 1¼ in., even without increasing the total thickness will result in longer fire endurance. In practice, this fact must be balanced against the effect on the load-carrying capacity under normal conditions. In normal practice, a slab is designed to have a particular load-carrying capacity. Once a combination of percent reinforcement and effective depth have been determined to satisfy this requirement, they would be held constant and increased cover would be achieved by increased overall thickness. Such procedure would lead to longer fire endurance corresponding to the increased cover and overall thickness.

Within the density range of 35 to 50 lb/ft^3, slabs made of higher density concrete will provide longer fire endurance than will those of lower density, both in terms of heat transfer through the slab and in terms of continued structural stability.

Although slabs of greater thickness may be expected to provide longer fire endurance than those of lesser thickness on the same span, no simple conclusion can be drawn from this study for the case of greater thickness made necessary by the provision of longer span members having load-carrying capacities equal to those of shorter spans.

The use of reinforcing bars of larger diameter and consequently of greater ratio of reinforcement results in longer fire endurance than the combination of smaller bars plus shear reinforcement employed in this study.

7. References

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[5] Lightweight Aggregate Concretes, HHFA, U.S. Government Printing Office, Washington 25, D.C. (1942).

[6] Fire resistance classifications of building constructions, Appendix B, NBS BMS 92, U.S. Government Printing Office, Washington 25, D.C. (1942).

8. Appendix

Information presented here includes the more important observations from the test logs, statements of the various end points, and the times at which they were reached. These times have been corrected for deviation, if any, of the furnace time-temperature curves from that defined in ASTM E 119 [3]. The correction formula is given in the same standard. Representative plots of time-temperature and deflection data, plus tabulation of end-point and other data, have been given in the body of this report.

The planks were numbered from the south end of the furnace in all but Test 380. In that test they were numbered from the west. This was done to facilitate the recording of observations and has been carried over in the following summaries of the test logs.

Roof Test 357—At 11 min, furnace and specimen luminous; 17 min, light smoke issued from joints between slabs; 41 min, diagonal cracks in unexposed surface across four corners of specimen running across two or three planks, also planks 1, 4, 10, 11, 12 were cracked longitudinally; 48 min, cracks much wider, heavy smoke from crack in plank 10; 53½ min, deflection about 7.7 in. and was increasing at the rate of 67 in./hr, causing loading pressure to fall off rapidly; 55 min, planks 7–10 broke 3 ft from west end, but reinforcement prevented complete collapse; load off; 1 hr, gas off.

The initial limiting condition in this test was load failure; it occurred at 57 min (corrected) when the deflection and rate of deflection became excessive. The limiting temperatures were not reached on the unexposed surface; but 1,200 °F maximum was reached on the reinforcing bars at 54 min and 1,000 °F average at 56 min.

Roof Test 371—At 9 min, fine crack across southwest corner of plank 2, slight separation between filler grout and furnace along south and east sides; 21 min, transverse cracks at west end of plank 1 and east end of planks 7, 8, 9, 10, longitudinal crack in center of plank 12, all in unexposed surface; 39 min, crack in plank 12 extended full length of plank, sides of crack in west end of plank 2 offset ⅜ in.; 51 min, two parallel cracks 8 in. apart extended almost full length of specimen about 3 to 4 ft from east side, heavy smoke rose from cracks; 57 min, planks buckled along transverse cracks which extended through to exposed surface; 1 hr 8 min, pressure in loading system unstable, partial collapse of specimen, load off, gas off.

The initial limiting condition in this test was load failure at 53 min. The maximum one-point rise of 325 °F on the unexposed surface occurred at 1 hr, and, by 1 hr 5 min, it was 700 °F. At 59 min, the 1,000 °F avg. on the lower reinforcing bars was reached. At the end of the test it appeared that all of the planks except 1, 2, 11, 12 were broken 3 to 4 ft from the east end.

Roof Test 379—At 37 min, planks 1 and 12 cracked diagonally 3 ft from each end and longitudinally near center, all cracks in unexposed surface; 58 min, planks 2, 10, 11 had short fine cracks at ends; 1 hr 11 min, planks 1 and 12 had fine but moderately long cracks, plank 2 had fine cracks 3 ft from west end; 1 hr 22 min, joints between planks 3 and 4, 4 and 5, and 9 and 10 cracked, smoke was seeping out, plank 3 cracked longitudinally, and considerable amount of smoke issued forth; 1 hr 31 min, maximum deflection 9.4 in., load off, gas off.

The initial limiting condition in this test was the maximum one-point temperature rise of 325 °F on the unexposed surface. This occurred at 1 hr 29 min near the crack in plank 3. By 1 hr 31 min, it was obvious that load failure was imminent. At this time, the maximum temperature on the unexposed surface was 650 °F. Cotton waste was placed over the crack in plank 3 at 1 hr 24½ min. This waste did not ignite immediately. However, observations after the test showed that it had been charred to a great extent.

Roof Test 380—At 33 min, a few cracks 1 to 3 ft long in joints and across planks near edge of unexposed surface, plank 4 cracked longitudinally 8½ to 9 ft long; 52 min, 15-ft-long crack in plank 4. Plank 8 had crack 10 ft long, plank 2 had crack 3 ft long in south end, maximum deflection 6.0 in.; 58 min, deflection 7.5 in.; 1 hr 12 min, sides of crack in plank 8 offset 4 in., joint between planks 8 and 9 offset 6 in., planks 2 through 6 bowed up at about 3 ft from south end, plank 9 cracked across exposed surface at center span; maximum deflection 11.4 in.; 1 hr 13 min, cotton waste placed over crack in plank 8 and ignited; 1 hr 16 min, load off, maximum deflection exceeded 12 in.; 1 hr 17 min, gas off.

The initial limiting condition in this test was the ignition of cotton waste over the crack in plank 8 at 1 hr 13 min. The 1,200 °F maximum temperature on the reinforcing bars was reached at 1 hr 11 min; the 1,000 °F average temperature was reached at 1 hr 10 min. Load failure was imminent at 1 hr 16 min, at which time the center span came into contact with the furnace structure 1 ft below the initial level of the specimen.

Roof Test 358, planks 5, 6, and 8 in. thick—At 12 min, wisps of smoke issued from joint between planks 2 and 3; 52 min, separation between planks and furnace, ends of planks raised ⅛ in.; 1 hr 7 min, cracks at east end of planks 3, 4, and 8, and at west end of plank 4; 1 hr 15 min, 6-in. planks (5–8) bowed up ⅜ in. at ends, 5- and 6-in. planks visibly dished; 1 hr 49 min, all planks had dished appearance, plank 4 had ¼-in.-wide transverse crack near east end; planks 2, 3, 4, and 8 had fine transverse cracks near east end, planks 2, 3, 4, 9, 10 and 12 had fine transverse cracks at west end, all cracks in unexposed surface; 2 hr 12 min, longitudinal crack 2 ft long in west end of plank 4, plank 10 raised 1⅛ in. at west end, plank 12 broken 3 to 4 ft from west end; 2 hr 29 min, plank 5 raised 1½ in. at west end, glow of furnace visible through crack in end of plank 10, 2 hr 55 min; transverse cracks in exposed surfaces of three planks 3 ft from east end, also several short cracks in planks 3 and 4; 3 hr 13 min, plank 4 cracked so that cotton rag ignited after few seconds, planks 3 and 4 sagged at each side of center; 3 hr 15 min, several pieces of concrete 1- to 4-in. diam fell from plank 4; 3 hr 40 min, flames passing through joint between planks 4 and 5, plank 4 offset 3 to 4 in. below plank 5 at centerspan; 3 hr 40 min, furnace glow visible from crack in plank 3 at 2 to 3 in. from east end; 3 hr 45 min, gas off.

After the specimen had cooled, examination showed that the exposed surface of the 5-in. planks had vitrified to a depth of 2 in., the 6-in. planks to a depth of ¾ in., and the 8-in. planks only on a thin surface layer. The 1,000 °F average occurred on the reinforcing bars of the 5-in. plank at 2 hr 9 min, the 6-in. plank at 2 hr 6 min, and the 8-in. plank at 2 hr 53 min. The 1,200 °F maximum occurred in the 5-in. planks at 2 hr 14 min, in the 6-in. planks at 2 hr 17 min and in the 8-in. planks at 3 hr 4 min. The maximum one-point rise of 325 °F on the unexposed surface was reached on the 5-in. planks at 3 hr 37 min and the average rise of 250 °F was reached at 3 hr 39 min. These limits were not reached on the 6-in. planks nor on the 8-in. planks. A cotton rag was ignited over a crack through a 5-in. plank at 3 hr 13 min, and, at 3 hr 40 min, flames were passing through the joint between planks 5 and 6. However, the times to limiting conditions resulting from cracking should not be taken as representative of nor compared with those observed in the tests of specimens of uniform thickness under design load.

Floor Test 372—At 25 min, crack along joint between planks 1 and 2; 37 min, plank 1 cracked along south side where supported by furnace frame, 55 min; blister of about 6 in. diam formed; 1 hr 8 min, plank 3 had 7-in. crack along center, maximum deflection (at south quarter point) 5.0 in.; 1 hr 21 min, sides of crack in plank 3 offset ¼ in.; 1 hr 39 min, maximum deflection 7.5 in.; 1 hr 59 min, maximum deflection exceeded 10 in. and was increasing very rapidly; 2 hr 4½ min, load off; 2 hr 12 min, gas off.

The initial limiting condition in this test occurred at 1 hr 40 min when the maximum allowable one-point temperature rise of 325 °F on the unexposed surface was reached. Load failure was reached at 2 hr 2 min. The average temperature on the reinforcing bars reached 1,000 °F at 1 hr 24 min, and the maximum temperature reached 1,200 °F at 1 hr 28 min.

Test 373—Floor planks 5 in., 6 in., and 8 in. thick. Note: the observations for the steel I-beam in this test are given separately. At 7 min, smoke issued from joints between planks 8 and 9 and between 9 and 10; 14 min, flames (possibly from encasing material of thermocouples) issued into furnace from southeast corner, continuing until after 40 min; 1 hr, cracks $\frac{1}{16}$ in. wide between specimen and furnace frame along east and west edges; 1 hr 15 min, short cracks in extreme west ends of planks 3 and 4 on unexposed surface; 2 hrs, two diagonal cracks across center of plank 12, 8-in. planks raised ½ to 1 in. at each end; 2 hr 23 min, long crack in middle of plank 11, diagonal cracks in southeast and southwest corners extending across planks 1 through 4; 3 hr 8 min, 8-in. planks raised 1 to 1½ in., 6-in. planks raised ⅝ in., separations between planks and furnace increased, some joints between planks cracked; 3 hr 44 min, considerable smoke issued from joint between planks 2 and 3; 3 hr 50 min, gas off.

The 1,000 °F average on the lower reinforcing bars was reached at 2 hr 50 min for the 5-in. planks and at 3 hr 12 min for the 6-in. planks. The maximum of 1,200 °F was reached at 3 hr 6 min for the 5-in. planks and at 3 hr 40 min for the 6-in. planks. Neither of these temperatures were reached in the 8-in. planks, but it was estimated that 1,000 °F average would have been reached at 3 hr 52 min. Again, this test was not intended to represent a single sample of construction and was not uniformly loaded. Consequently, there were no definite criteria which could be used to determine the limiting conditions of the specimen.

Test 373—Steel I-beam encased in blocks. At 37 min, horizontal and vertical hairline cracks in several joints between blocks, vertical cracks across 2 blocks near center of east face; 1 hr, more cracks in joints, vertical cracks across block in west face; 1 hr 41 min, one soffit block cracked through and was sagging; 2 hr 11 min, third soffit block from south cracked and was sagging in 3-in. deep V, side block on west face above this dropped about 1 in., 2 hr 15 min, cracked soffit block and three blocks from each side fell; 2 hr 22 min, load off; 2 hr 23 min, soffit and side blocks from half of beam fallen, 2 hr 26 min, separation of ½ to 1 in. between beam and floor planks; 2 hr 51 min, only beam protection still in place were blocks in south 1½ ft and blocks in north 2 to 3 ft, test continued to obtain data on floor planks.

The limiting condition was load failure at 2 hr 22 min. The average temperature of 1,000 °F on the beam was first reached on one section of the beam at 2 hr 1 min. The maximum one-point temperature of 1,200 °F was reached at 2 hr 9 min. By 2 hr 30 min, the average temperatures of all 4 sections of the beam were well above 1,000 °F; the highest section average was over 2,000 °F. The temperature of the steel does not constitute a limiting factor in tests of beams carrying design loads.

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