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NATIONAL BUREAU OF STANDARDS REPORT

1926

FIRE TESTS OF COLUMNS ENCASED IN PUMICE-AGGREGATE CONCRETE

by

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and
A. F. Robertson

NBS

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ABSTRACT

Fire-endurance tests have been performed on three loaded 6-in. 20 lb steel H section columns protected by 2 in. of monolithic concrete. The concrete used was mixed in the ratio of 1:3 1/8:1 1/2 by volume using pumice as the fine and coarse aggregates. Pumice from a different source was used for preparation of the concrete for each of the specimens. These were representative of material available in New Mexico, Oregon and California. It is shown that the particular specimens tested provided fire-endurance periods of more than four hours. There was little evidence of differences in performance of the various aggregates tested.

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1. INTRODUCTION

Three 6-in. steel H columns protected by 2 inches of pumice-aggregate concrete were subjected to fire-endurance tests 267, 270 and 271, during July and August 1950. Different aggregates were used for each column encasement. These pumice aggregates were representative of those available in New Mexico, Oregon and California respectively. Auxiliary fire-endurance tests were made on a wall panel, 268, and a floor, 269, constructed of the same kind of concrete. The tests were made at the request of the Office of the Chief of Engineers, Department of the Army, to develop information on the behavior of such concretes when exposed to fire.

2. MATERIALS

2.1 Steel Columns

The fabricated columns were made of 6-in. 20 lb stanchion sections having a cross-sectional area of 5.88 in.^{2} The heads of the columns were restrained for 32 1/4 in. by means of 1/2-in. flange cover plates, leaving an effective column length of 123 3/4-in. as shown in figure 1. The column ends were milled square with the column axis, thus allowing direct loading contact with the 1 7/8-in. steel bearing plates. The columns were hot riveted. The least radius of gyration of the columns was 1.50-in. and the ℓ/r ratio was 82.5.

2.2 Concrete

The pumice concrete used for encasing the columns was one part portland cement, 3 1/8 parts fine aggregate and 1 1/2 parts coarse aggregate by volume. The aggregate used in the concrete for encasing one column, the floor, and the wall panel was from New Mexico. Because of the difficulty of placing the harsh mix made with this aggregate, an alkyl aryl sulphonate was used as an air entraining agent in the concretes made with the Oregon and California aggregates for encasing the other two columns. The proportion of chemical used was 1 part to 850,000 parts cement. The aggregates were water soaked before mixing.

After seasoning, the concrete cylinders made with the aggregate from New Mexico weighed 96 lb/ft^3 and had a moisture content of 18 percent; those made with the aggregate from Oregon weighed 101 lb/ft^3 and had a moisture content of 32 percent; and those made with the aggregate from California weighed 103 lb/ft^3 and had a moisture content of 21 percent. The moisture contents seemingly indicated the tenacity with which the aggregates held free water. Sieve analyses and bulk density measurements of the dry aggregates are presented in table 1.

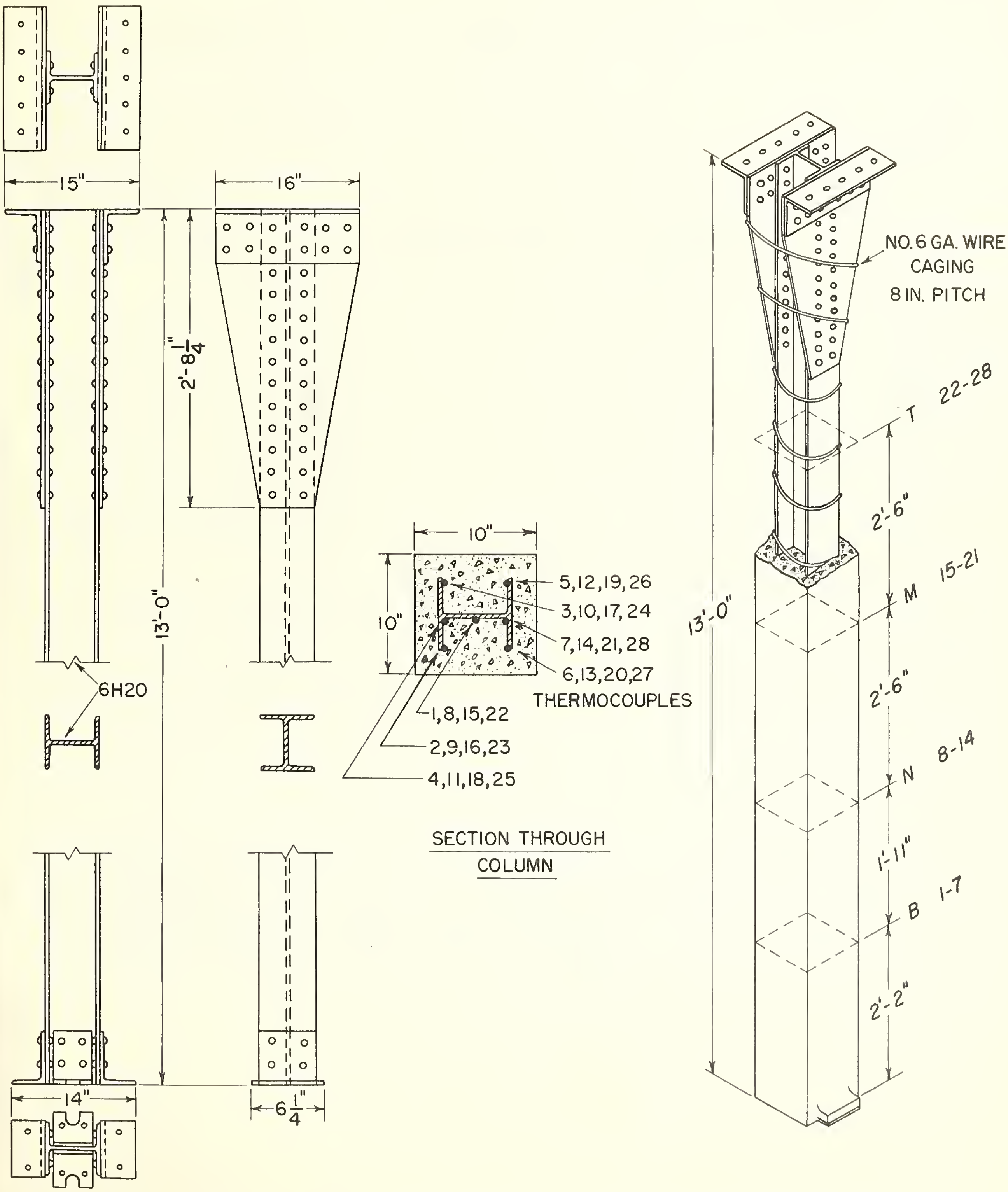

FIG. 1. TESTS 267, 270, 271. CONSTRUCTION DETAILS OF STEEL COLUMNS. FP2983

Table 1. Sieve analysis and specific density of pumice aggregates.

Sieve designation	Amount retained on sieve						
	New Mexico			Oregon		California	
	Coarse	Fine No.1*	Fine No.2*	Coarse	Fine	Coarse	Fine
	percent			percent		percent	
1 in.	0	0	0	0	0	0	0
3/4 in.	0	0	0	0	0	11	0
1/2 in.	0	0	0	0	0	68	0
3/8 in.	4	0	0	0	0	95	0
No. 4	96	2	0	52	1	99	13
8	97	36	20	87	32	99	42
16	97	69	46	88	59	99	58
30	97	88	77	89	62	99	68
50	97	94	77	89	70	99	76
100	98	95	83	90	71	99	81
Specific density lb/ft^3	26	39	58	29	38	32	50

*New Mexico fines No. 1 used for columns and fines No. 2 used for slabs.

2.3 Wire and Rods

Steel wire of No. 6 ga (0.192 in. diam) was used for the spiral reinforcement to bond the concrete to the columns. Welded steel fabric of 6- by 6-in. mesh by No. 4 ga (0.225 in. diam) wire was used as reinforcement in the concrete slabs. Three 3/4-in. diam hot rolled steel rods were also used to reinforce the floor slab.

3. CONSTRUCTION

3.1 Preparation of Columns for Covering

The columns were prepared for covering with concrete by mounting thermocouples and winding a 6-ga wire (0.192 in. diam) spirally around the column on 8-in. pitch as shown in figure 1. The wire spiral had about 1-in. clearance at the center of the column flanges, leaving a 7/8-in. clearance between the wire and the outside of the concrete covering. Each turn of the wire was tack-welded at one of the four points where it came in contact with the column edges.

3.2 Placement of Concrete

Plywood forms were spaced 2 in. on all sides from the vertical column shafts. Concrete poured into the tops of the forms was compacted around the columns by vibrating the form sides with an air hammer. In mixing the concrete for the first column, the aggregate from New Mexico was incorporated into the batch as received from the producer. Because of difficulties encountered in handling the concrete prepared in this manner, the two remaining concretes were prepared with an air entraining agent and aggregates which had been water soaked for twelve hours.

The two 4-in. thick concrete slabs were also prepared in wood forms. The wall panel was 4- by 8-ft and the floor, 4-ft 4-in. by 9-ft. Both were reinforced with wire fabric, the latter with rods as well.

Each concrete batch was mixed for not less than 1 1/2 min before the water was added and not less than 2 min afterward. The water content was controlled to produce a 5-in. slump. Three 6-in. diam by 12-in. cylinders were cast from each of the three concretes. Compressive tests were made at the time of the fire tests of the columns; the results of these tests are presented in table 2. The low strength of two of the three cylinders of concrete made with the pumice aggregate from New Mexico is evidence of the difficulties experienced when working with this aggregate without previous wetting and subsequent treatment of the mix.

Table 2. Compressive tests on concrete cylinders.

Aggregate type	Compressive strength			Moisture content of cylinders
	lb/in.^{2}			percent
	1	2	3	
New Mexico	1210*	2260	1550*	18
Oregon	2140	2100	2090	32
California	2200	2310	2190	21

* Large voids in interior of cylinders probably caused by rodding of harsh mix during filling of cylinder molds.

3.3 Workmanship

The fabrication of the steel columns was of good commercial quality. The concrete was also of good quality for both the Oregon and California mixes. The difficulties of working the pumice aggregate from New Mexico in the dry condition caused some small voids in the column encasement and rough surfaces on the column and the two slabs. These surfaces were smoothed by the use of mortar composed of cement with pumice fines after removal of the forms.

3.4 Seasoning of Concrete

The forms for the columns and cylinders were stripped after one day, and the concretes dampened daily for the next 6 to 13 days, after which the columns were weighed periodically for fifteen days, during which time the laboratory was heated to about 130°F at night and allowed to cool to about 85°F during the day. The weight of the concrete column coverings made with pumices from New Mexico and Oregon remained reasonably constant after 13 days and 10 days, respectively. However, that made with pumice from California did not remain constant, but lost three pounds during the last seven days of seasoning, of which one pound was lost during the last two days. The column coverings made with pumice from New Mexico lost 12 pounds and those made with pumices from Oregon and California lost 13 pounds each during seasoning.

4. EQUIPMENT AND TEST METHOD

4.1 Equipment

A gas-fired furnace was used for testing the columns. Load was applied to the columns by means of a hydraulic jack under the furnace floor. It was maintained by a pump which supplied oil to the jack at constant pressure. The movable section of the floor of the furnace was supported on a spherical bearing block resting on the jack piston.

Temperatures in the furnace combustion space were measured by 12 thermocouples protected by iron tubes. These tubes projected from the furnace walls and roof. The temperatures at 28 points in the steel column were measured by thermocouples made from No. 24 ga duplex chromel-alumel wire. The wires were protected with an insulation of woven glass impregnated with silicone varnish. The ends of the pair of wires were twisted and welded together and the resulting junction inserted to a depth of about 1/8-in. in a hole in the steel column and secured therein by swaging the steel around the hole. The junctions were arranged with seven thermocouples at each of the four levels of the column as indicated by positions B, N, M and T, in figure 2.

The concrete floor was tested in a furnace having a combustion space 4-ft 10-in. wide, 8-ft long, and 3-ft high. The six thermocouples were protected by iron tubes as in the column furnace. Temperature measurements in the steel rod reinforcement were obtained by means of thermocouples inserted into each rod at mid-length. The leads of these thermocouples were cast into the concrete floor to emerge on the unexposed side. Surface temperatures were measured at six locations on the top of the concrete floor. The junctions and a short length of the termocouple leads were placed under 6-in. square by 0.4-in. thick felted asbestos pads.

The concrete wall panel was tested in a furnace with a combustion space 4-ft 8-in. wide, 10-ft high, and 2-ft deep. The furnace temperatures were measured with nine chromel-alumel thermocouples protected in iron tubes. Surface temperatures were measured with six thermocouples held in contact with the unexposed wall surface by felted asbestos pads.

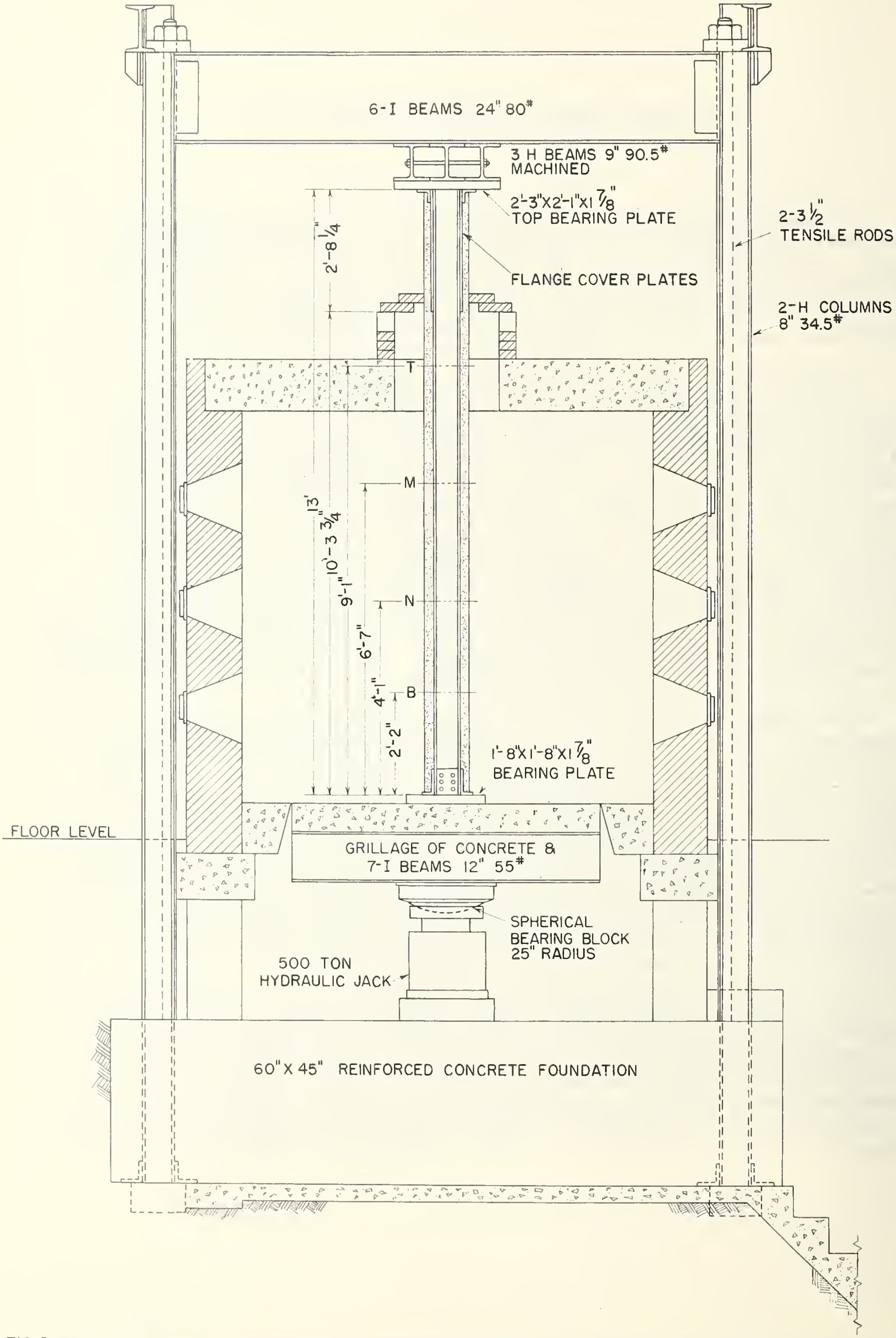

FIG. 2. TESTS 267, 270, 271. FURNACE AND LOADING EQUIPMENT WITH TYPICAL COLUMN IN PLACE. FP2983

4.2 Test Specifications

The tests were conducted in accordance with the Standard Methods of Fire Tests of Building Construction and Materials, A.S.T.M. Designation E119-47, except that the floor and wall were of substandard size. These methods require that one side of the wall, the underside of the floor, and the complete external surface for at least 9-ft of column length be exposed to a fire controlled to conform closely to the time temperature curve defined by the following points:

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

and rising 75 degrees F per hr thereafter.

The specifications require that during the fire test the column be subjected to the normal working load or, alternatively tested without load but with the temperatures measured in a prescribed manner. The fire-endurance limit is considered as being reached when the column fails to support the applied load or, in the alternate test, when the average temperature of the steel at any one level in the column reaches 1000°F (538°C), or the temperature of the steel at any one of the measured points reaches 1200°F (649°C).

Walls and floors are considered to have reached their fire-endurance limit when flame breaks through the structure, when gases hot enough to ignite cotton waste pass through, when the average temperature rise of five or more locations on the unexposed surface amounts to 250 degrees F, or the increase of temperature at any one of the locations becomes 325 degrees F.

4.3 Bearing of Columns

Steel bearing plates were bolted to each end of the columns. The lower plate was 20-in. square and 1 7/8-in. thick. The 25- by 27- by 1 7/8-in. top plate was fastened to the column with fourteen 7/8-in. bolts, and it in turn was bolted to the overhead structure of the loading device. Three-fourths inch of grout was used between the bottom bearing plate of the column and the furnace floor. As the floor rested on a well-lubricated ball and socket joint whose center of curvature fell on the center line of the column at its lower end, the column can be considered as having one spherical end bearing.

4.4 Restraint of Columns

The method of fastening the column head gave substantially effective restraint against bending of the upper 32 1/4-in. The upper end of the 10-ft 3 3/4-in. length can be considered as fixed, figures 1 and 2.

4.5 Loads on Columns

The load applied to the columns during test was computed from a formula recommended by the American Institute of Steel Construction, as follows:

$$P = A\left[17{,}000 - 0.485\left(\frac{\ell}{r}\right)^2\right]$$

in which P is the total load on the column, A is the cross sectional area of the steel shaft, ℓ its effective length, and r its least radius of gyration. For these columns, the values are:

$$A = 5.88\text{-in.}^2$$
$$\ell = 123\ 3/4\text{-in.}$$
$$r = 1.50\text{-in.}$$

Entering these in the above formula gives the value of P as 80,500 lbs total load on A, equivalent to a stress of 13,700 lbs/in.^{2}

This load, which was applied before starting the test fire, was maintained during test until rapid yielding of the column occurred. Load was removed when the yield of a column from its maximum expansion amounted to about 0.6 in., or about three-fourths of its thermal expansion.

4.6 Measurement of Deformations

Changes in length of the column were indicated by observations of a dial micrometer arranged to measure movements of the piston of the hydraulic loading jack. Movement of the framework supporting the overhead bearing plate, due to expansion, was small in comparison with the expansion of the exposed portion of the column. No measurements of lateral deflections were made during the tests.

4.7 Mounting of Floor

The floor was mounted on the furnace with each end bedded on 3/4-in. of portland cement mortar. No load was applied during the test, but deflections were measured at two locations at the midspan of the floor. These deflections were measured between a stationary beam and the top of the floor by means of a rule.

4.8 Mounting of Wall

The wall was mounted in a steel and concrete frame which supported it at the front of the furnace. It was held in the frame by 5/16-in. bolts, spaced about 18-in. apart, in contact with the wall surfaces along each vertical edge. This method of mounting served to restrain the wall somewhat against lengthwise bending. Deflections were measured at the center of the panel surface and at 12-in. each side thereof along the horizontal center line.

5. RESULTS OF TESTS

5.1 Columns (General)

Although shrinkage cracks developed in the concrete encasements, they remained in place even after load failure of the columns. These cracks are evident in the photographs of the tested columns, figures 3 and 4. None of the cracks in the concrete exceeded 3/8-in. and most of them were less than 1/8 in.-wide. The test results on the individual columns are presented in the first section of table 3 and the following test logs.

Fig. 3--Column after test.

Fig. 4--Column after test.

5.2 Test 267 Column, New Mexico Pumice Concrete

5.2(a) Test Log

Time hr:min	Observations
0:00	Test started 10:54 A.M., July 31, 1950. Initial temperature 86°F.
0:05	Corners of column red.
0:15	Column is dull red.
0:30	Cracks on north and east sides.
0:42	Two cracks on north, 3-ft and 1-ft in length; many fine cracks on other sides.
0:52	Large cracks on all sides about 1/16-in. wide and 6- to 12-in. in length.
1:21	Cracks larger on all sides.
1:32	Crack on north continuous for almost entire column length.
2:00	Cracks on south about 1/8-in. wide.
3:06	Very large crack on north, column bright orange.
3:57	Some cracks on east about 1/4-in.
4:45	Oil supply to jack shut off.
4:47	Gas off.

Examination of the column after test indicated that the covering had been well distributed around the column. The thickness of the shrunken concrete over the steel averaged 1.9-in. with a maximum deviation of 3/16-in.

5.2(b) Temperatures

Furnace and column temperatures are shown in figure 5. Temperatures of the steel column are plotted as (a) the highest at any level (b) the minimum average at any level and (c) the average at level N.

The rapid rise of temperature of the steel to 212°F (100°C) within 10 min at level B was probably caused by a fault in the concrete covering. It may, however, have been caused by the separation of a thermocouple from the steel.

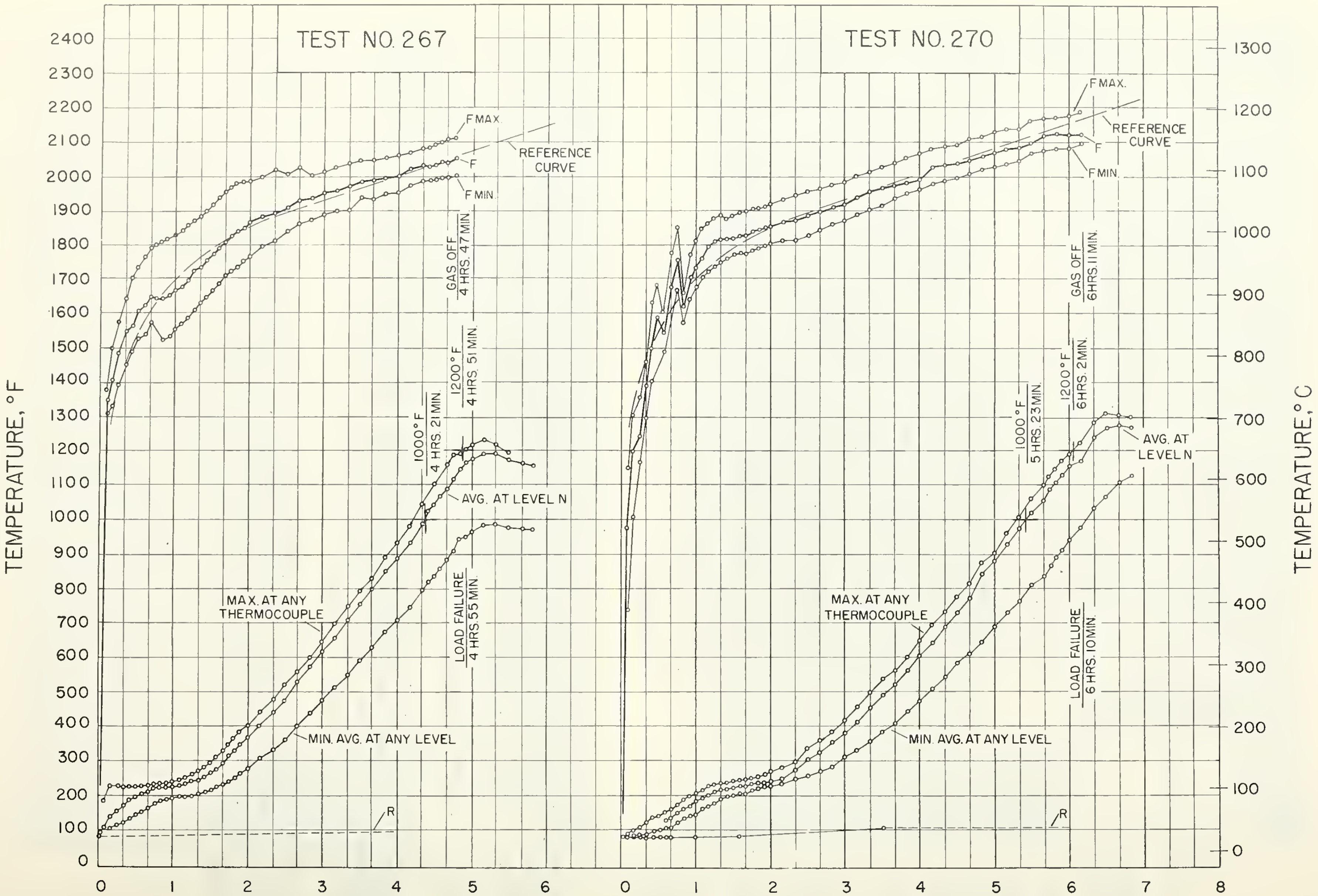

FIG. 5. TESTS 267 AND 270. TEMPERATURES DEVELOPED IN COLUMNS AND CONCRETE ENCASEMENTS MADE WITH PUMICE AGGREGATES FROM NEW MEXICO AND OREGON. FP2983

The average temperature at level N reached 1000°F at 4 hr 21 min and thus represents the limit as defined by an average temperature at one level. The temperature of 1200°F at one thermocouple was reached after 4 hr 51 min fire exposure. The fire exposure during the test, as represented by the area under the curve of average temperatures, was 101 percent of that under the standard reference curve.

5.2(c) Expansion of Column

The changes in column length are shown in figure 6. The column reached its maximum length in expansion after 4 hr 35 min of fire exposure. The load was removed after 4 hr 45 min fire exposure at which time the column length had been reduced by about 0.1-in. Through inadvertence, the test was terminated at this time, estimated as about 10 min before the usual limit of yield of 0.6-in. in compression would have occurred; therefore the time of failure of this column under load is recorded as 4 hr 55 min.

5.3 Test 270 Column, Oregon Pumice Concrete

5.3(a) Test Log

Time hr:min	Observations
0:00	Test started 9:49 A.M., August 4, 1950. Initial temperature 77°F.
0:10	Fine cracks outlined by moist darker areas.
0:20	Corners of column red.
0:30	Prominent cracks on west about 3-to 4-ft up.
0:35	Larger crack on east 3-ft up
0:47	Cracks better defined over the column surface.
1:26	Pronounced vertical crack on north.
1:41	Vertical crack on south.
1:50	Cracks a little larger; no evidence of moisture.
2:15	Crack on north side about 1/16-in. wide (column web lies in north south plane).
2:30	Cracks on north and south about 1/8-in. wide.

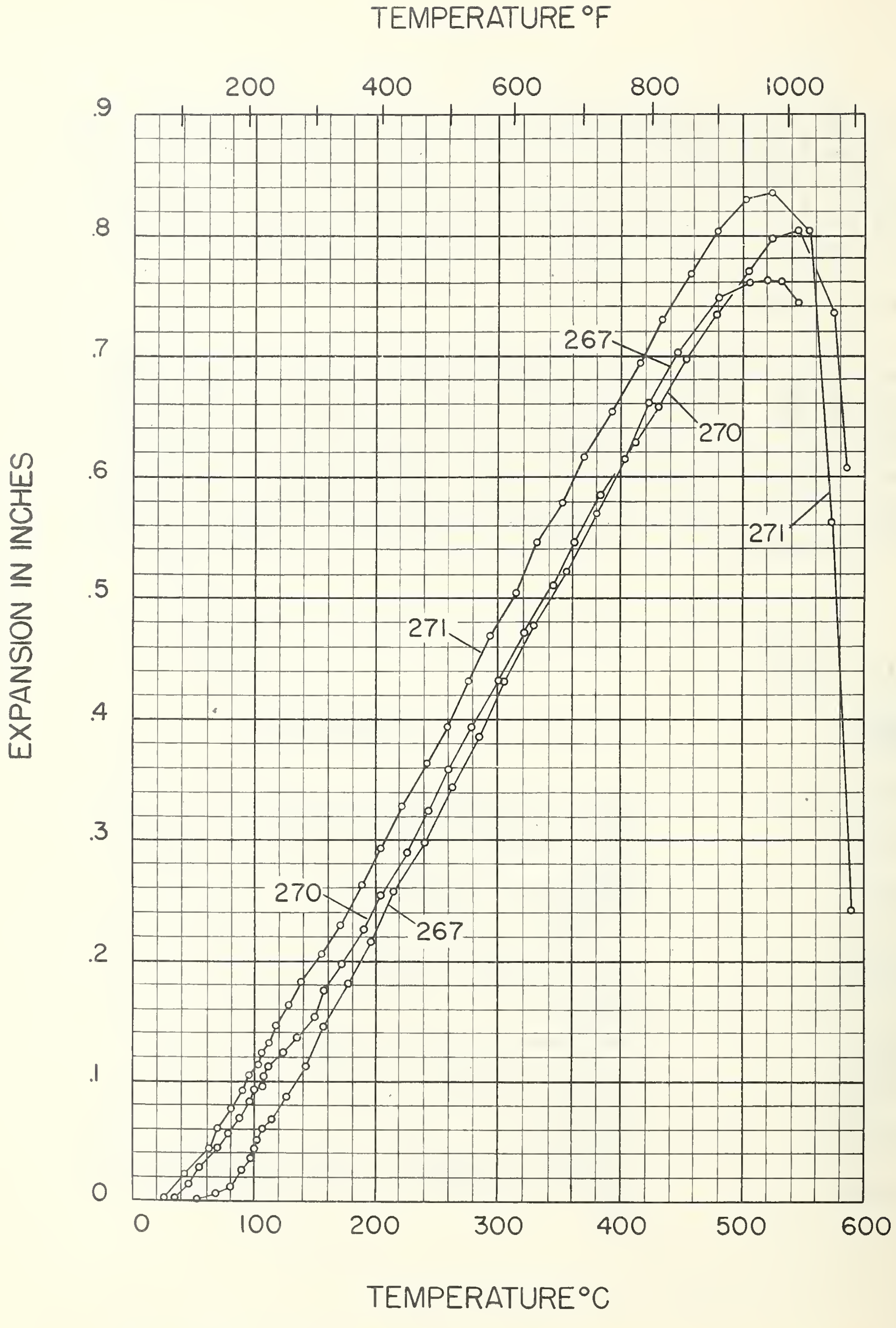

FIG. 6. TESTS 267, 270, 271. EXPANSION OF LOADED COLUMNS WITH INCREASING TEMPERATURES. FP2983

Time hr:min	Observations
5:10	Cracks on south about 1/4-in. wide. North windows of furnace opaque.
5:50	Column has stopped expanding.
6:00	Small yellow flame on column surface about 6-ft up. Evidently gases coming from large horizontal crack are burning.
6:10	Oil shut off after about 0.6-in. contraction of column. Load failure.
6:11 1/2	Gas fire off.

Examination of the column after the test showed the concrete covering to be denser than that for test 267. The aggregates had become vitrified to a depth of 1/2-in. from the surface. After the test, the thickness of the shrunken concrete was measured at 19 locations along the column edges and found to average 1.9-in.

5.3(b) Temperatures

Temperature data are presented in figure 5. The average temperature of 1000°F occurred at level N after 5 hr 23 min fire exposure. A maximum of 1200°F occurred at level B after 6 hr 2 min fire exposure.

The fire severity during the test period was 100 percent of the prescribed normal.

5.3(c) Expansion of Column

Data on the expansion of the column along its length are presented in figure 6. The column reached its maximum expansion in length after 5 hr 50 min fire exposure. The load was removed after 6 hr 10 min of exposure, at which time the column length had been reduced by 0.6 in., marking its failure under load.

5.4 Test 271 Column, California Pumice Concrete

5.4(a) Test Log

Time hr:min	Observations
0:00	Test started 9:47 A.M., on August 9, 1950. Initial temperature 81°F.
0:15	Small cracks visible on west side 4-ft up.
0:28	Small crack 3-ft up on east side.
0:32	Small cracks near bottom of east and south sides.
0:35	Network of fine cracks on north.
0:40	One crack on east 1/32-in. wide; others finer.
1:13	Many horizontal cracks.
1:23	Crack on south about 1/16-in. Most are horizontal on south and east; craze cracks on north and west.
1:53	Cracks on north and west better defined; vertical cracks on north and west developing.
2:15	Vertical cracks north and west.
3:35	Cracks a little wider and better defined.
3:54	Cracks on south about 1/8-in. wide.
4:50	Some cracks about 1/4-in. wide.
5:20	Cracks on east appear to be 3/8-in. wide.
5:30	Column not expanding.
5:55	Load removed from column.
5:57	Gas off.

Examination of the column after the test showed the covering to be compact and well supported with a small amount of vitrification. Measurements of the concrete at 24 locations along the column edges indicated an average thickness after shrinkage of 1.95-in., with a maximum deviation of 3/16-in.

5.4(b) Temperatures

Temperature data for this test are presented in figure 7. The average temperature of 1000°F occurred at level B after 5 hr 25 min fire exposure. A maximum of 1200°F occurred

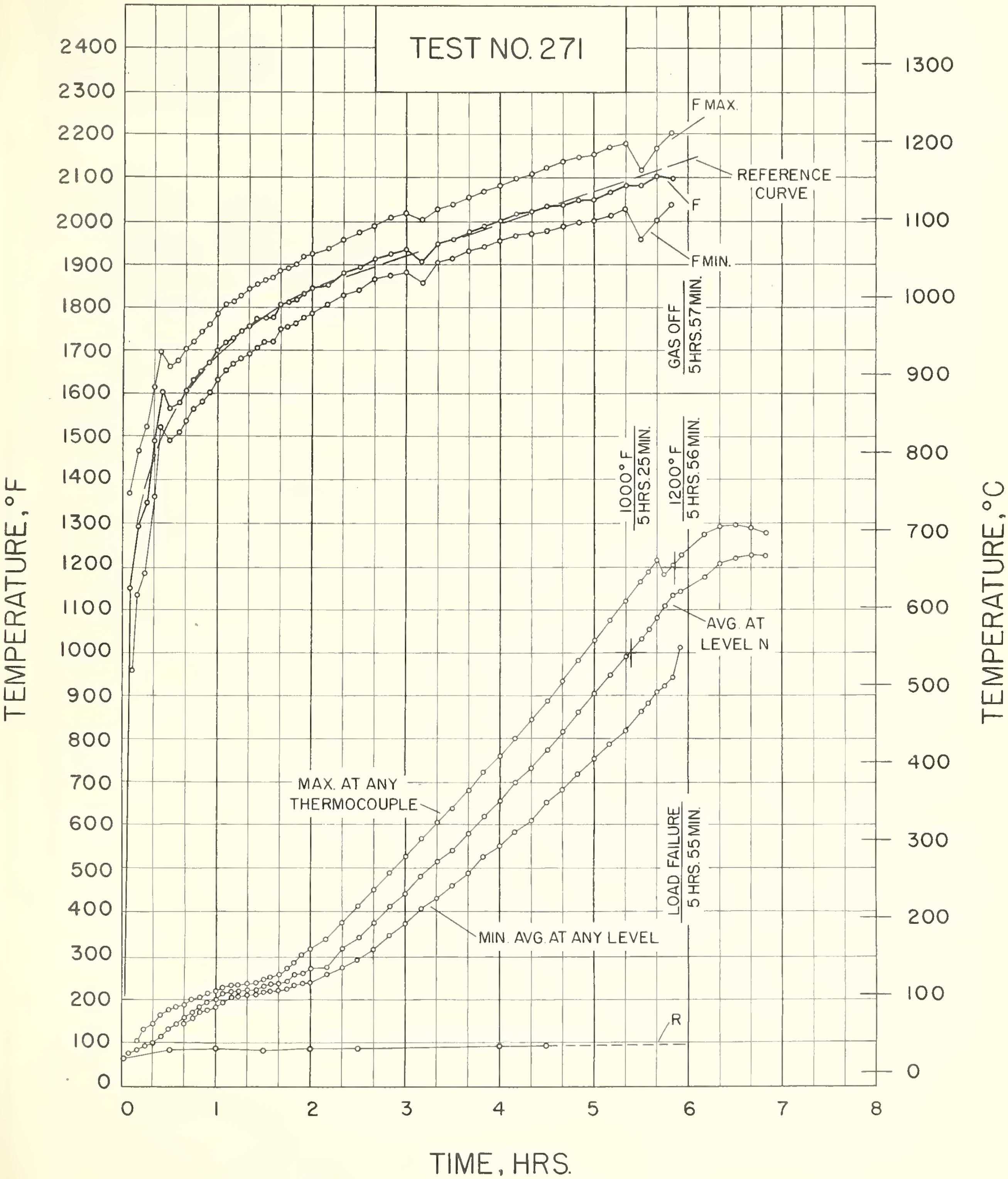

FIG. 7. TEST 271. TEMPERATURES DEVELOPED IN COLUMN AND CONCRETE ENCASEMENT MADE WITH PUMICE AGGREGATE FROM CALIFORNIA. FP2983

at level N after 5 hr 56 min fire exposure. Thus these both represent column fire resistance periods as limited by temperature of the steel in accordance with the alternate method of test.

5.4(c) Expansion of Column

Longitudinal expansion data for the column are presented in figure 6. The column reached its maximum length after 5 hr 30 min fire exposure. A total contraction of about 0.6-in. had taken place at 5 hr 55 min, marking failure of the column under load, whereupon the load was removed.

5.5 Test 268
Exploratory test of New Mexico Pumice Concrete Wall Panel

5.5(a) Test Log

Time hr:min	Observations
0:00	Test started 9:30 A.M., August 1, 1950. Initial temperature 86°F.
0:24	Small crack across panel outlined by damp area.
0:28	Small amount of water bubbling from crack on unexposed surface.
0:45	Water stopped bubbling from crack.
0:50	Fine cracks appear in exposed surface of panel.
1:10	Additional cracks, including two on fire exposed side.
1:27	Horizontal crack near bottom appears to extend through wall.
2:25	Steam from cracks.
2:30	Initial cracks now 1/16-in. wide.
3:00	Many cracks on exposed surface some of which seem to pass through slab.
3:46	Original horizontal crack now dry.
4:50	Cracking sounds.
5:08	Gas off.

During the test, the supporting frame bowed out at both ends from 1-to 2-in. from the front of the furnace. The panel naturally followed the frame curvature. After the test the frame resumed its normal plane shape and the panel likewise straightened out leaving almost no sign of cracks on the unexposed surface. Figure 8 shows the fine network of cracks on the fire-exposed surface after the test.

5.5(b) Temperatures

Furnace and surface temperatures are presented in figure 9.

The limit of surface temperature rise was reached after 4 hr 48 min fire exposure when thermocouple No. 4 showed a rise of 325 degrees F. This thermocouple was located directly over one of the larger cracks which evidently was the cause for early failure. The average surface temperature rise limit of 250 degrees F was reached after 5 hr 6 min of fire exposure.

The severity of the fire exposure was 100 percent of the normal specified by the A.S.T.M. standard test methods.

5.5(c) Deflections

Deflections as measured during the test were minor and of no particular significance because of the restraint furnished by the mounting frame.

5.6 Test 269
Exploratory test of New Mexico Pumice Concrete Floor Panel

5.6(a) Test Log

Time hr:min	Observations
0:00	Test started 9:18 A.M., August 2, 1950. Initial temperature 88°F.
0:09	Cracking sounds. Fire exposed surface has two spalled areas about 1/4 in. deep.
0:11	Cracking continues; one reinforcing rod partly exposed to fire.
0:16	Loud cracking. About 1/3 length of center reinforcing rod now exposed to fire.
0:18	Cracking sounds frequent. About 1-in. thickness of ceiling spalled off.
0:26	About 1/3 width of under surface of floor had spalled off.
0:30	Crack and moist areas across floor surface.
0:37	Spalling appears to have nearly stopped.

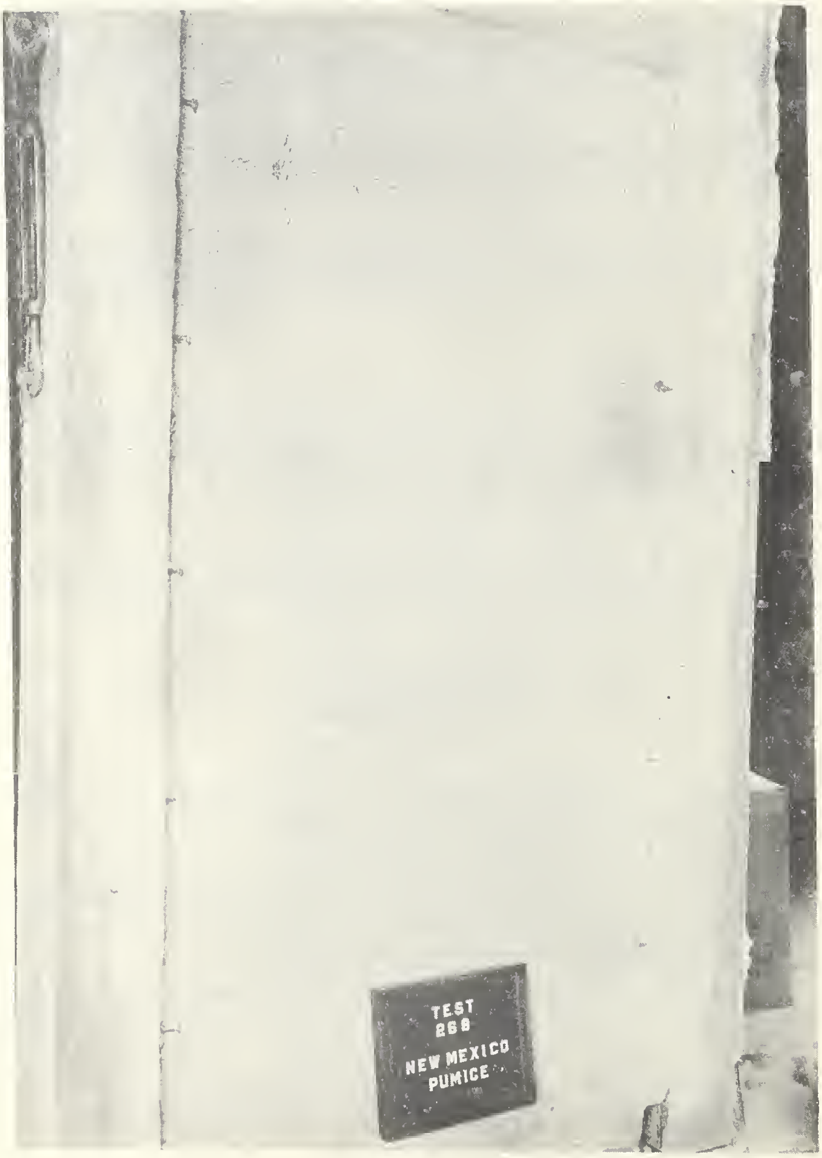

Fig. 8--Fire-exposed surface after test.

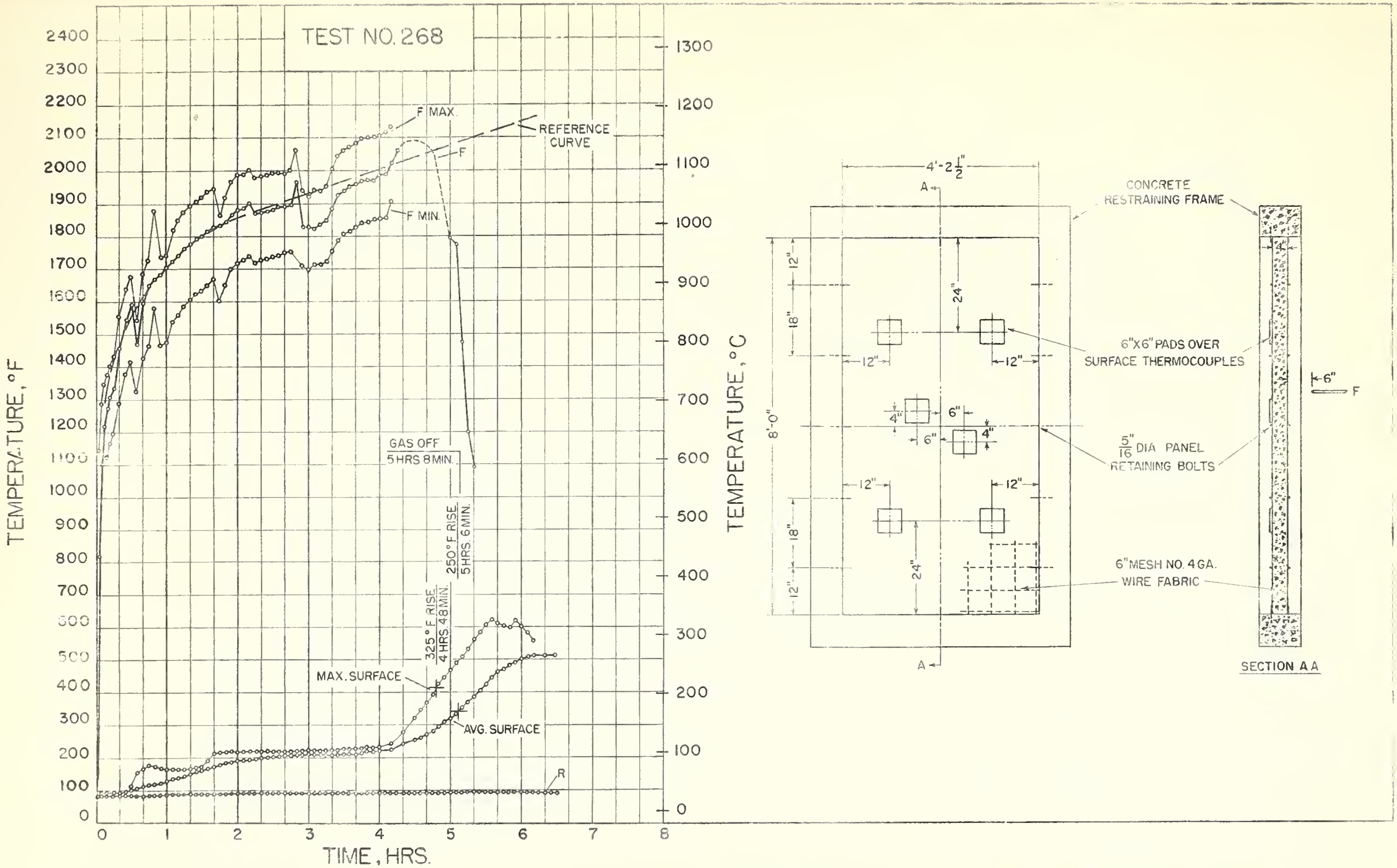

FIG. 9. TEST 268. TEMPERATURES DEVELOPED IN THE FIRE TEST OF A WALL; DETAILS OF THE WALL CONSTRUCTION AND LOCATIONS OF THE THERMOCOUPLES. FP2383

Time hr:min	Observations
0:44	A large crack formed through the concrete near the south and at the east reinforcing rod.
0:46	Spalled area 2-in. deep in spots, wire fabric visible.
0:49	Most of top surface cracks now dry.
1:13	All moist spots and most of the cracks on the top surface are near the west side of the floor.
1:19	Steaming has stopped.
1:58	Large cracks on fire exposed surface. West reinforcing rod exposed.
2:21	Large crack across ceiling.
2:29	Piece of cotton fabric placed on a hot spot about 10-in. south of center floor.
2:30	Cotton scorched.
2:31	Cotton burst into flame.
3:17	Fire visible through crack at hot spot.
3:30	Large crack near northwest corner of floor.
4:00	Cracks larger.
4:23	Gas off.

It was obvious that this floor slab would have failed much sooner if it had been loaded. Figure 10 shows the spalling and cracking of the fire exposed side of the floor.

5.6(b) Temperatures

Figure 11 shows the temperatures of the one steel reinforcing rod that remained protected by the concrete. The temperatures in the central rod, which was exposed to the flame shortly after the start of the test, rose very rapidly, and after 25 min were about the same as those measured by the furnace thermocouples.

Figure 11 also shows average and maximum surface temperatures. It is evident that during the test the hot spot mentioned in the test log was at a much higher temperature than any of the surface thermocouples. A piece of cotton placed on this spot 2 hr 29 min after the start of the test caught fire and burned in about 2 min, thus defining the end

Fig. 10--Test 269. Exposed surface of floor after test.

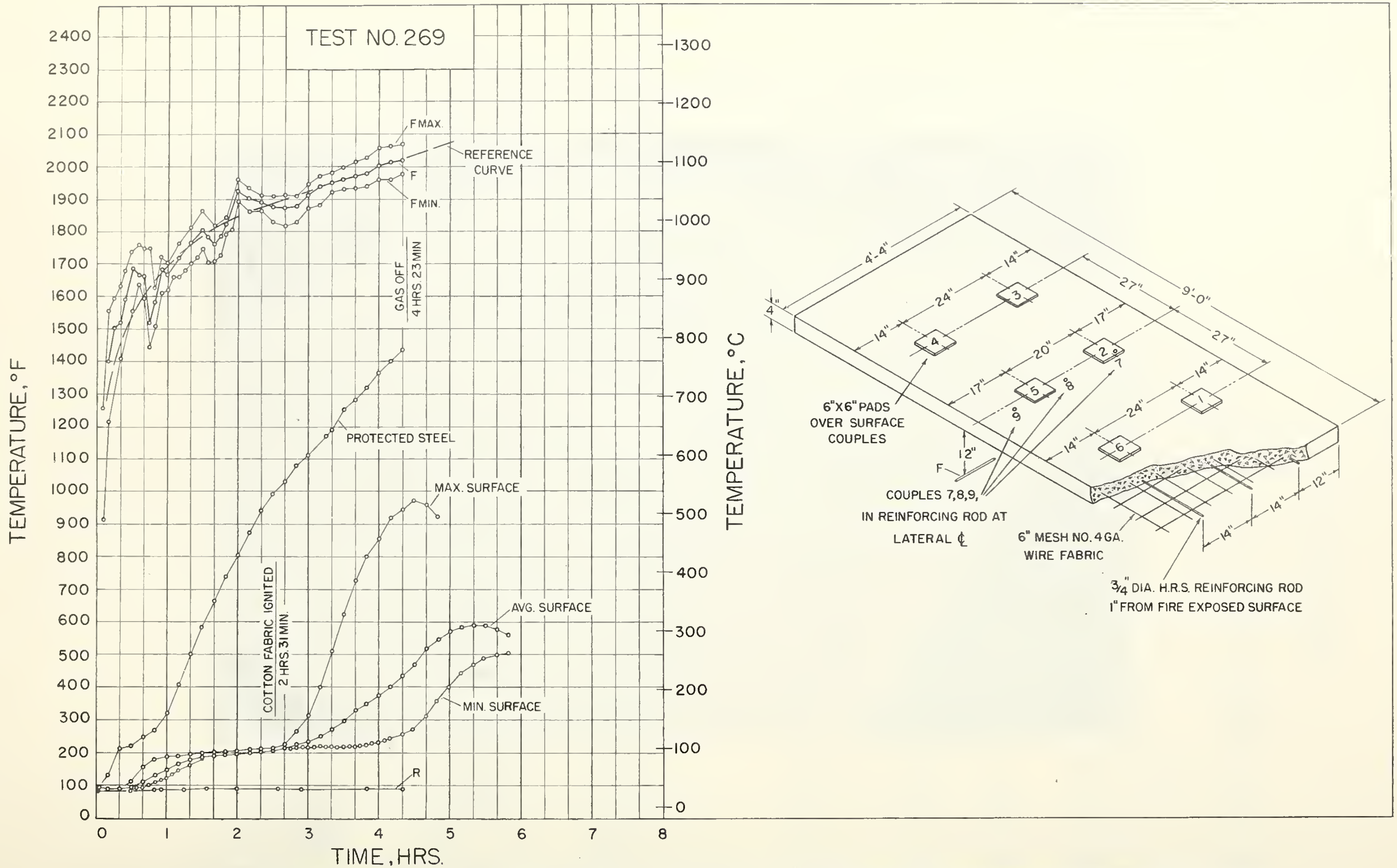

FIG. 11. TEST 269. TEMPERATURES DEVELOPED IN THE FIRE TEST OF A FLOOR; DETAILS OF THE FLOOR CONSTRUCTION AND LOCATIONS OF THE THERMOCOUPLES. FP 2983

point for this test. The limiting temperature rise of 325 degrees F occurred at one thermocouple location after 3 hr 11 min of fire exposure. The limit of average surface temperature rise of 250 degrees F occurred after 3 hr 45 min of fire exposure.

The severity of fire exposure was 101 percent of that specified by the A.S.T.M. standard test method.

5.6(c) Deflections

An abbreviated table of deflections at the mid-span of the floor are tabulated below:

Time	Deflections	
	East of center	West of center
hr:min	in.	in.
0:00	0.0	0.0
0:30	0.8	1.3
1:00	1.1	1.2
1:30	1.9	2.4
2:00	3.0	3.8
2:30	4.3	5.8
3:00	4.9	7.3
3:30	5.6	9.1
3:45	5.9	10.0
4:00	6.3	10.8

Although no load was applied to the floor during this test, the deflections began early and became excessive after 3 hr 45 min fire exposure, at which time the limiting rise of average surface temperature, 250 degrees F (139 degrees C) was reached. An unfortunate arrangement in the placement of the steel fabric at the middepth of the slab instead of below the rod reinforcement is thought to have not only contributed to the rapid deflection of the slab but also to have been a factor in inducing early spalling of the ceiling surface to expose the reinforcing rods.

TABLE 3. SUMMARY OF DATA FROM FIRE TESTS OF PROTECTED STEEL COLUMNS.

GROUP REFERENCE	COLUMN NO.	SHAFT SECTION	LOAD	PROTECTIVE COVERING		FIRE EXPOSURE PERCENT OF STANDARD	PERIOD OF EXPANSION	TIME TO FAILURE UNDER LOAD	TEMPERATURES AT TIME OF FAILURE UNDER LOAD		
				Kind	Thickness Over Flange				Max in Column Flanges	Avg of Hottest Section	Avg Sections N, M & T
			lb/in.^{2}		in.		hr:min	hr:min	^{o}F	^{o}F	^{o}F
Present tests	267	6 in. 20 lb H	13,685	Concrete, New Mexico pumice aggregate	2	100.8	4:35	4:55	1234	1162	1040
	270	do	13,685	Concrete, Oregon pumice aggregate	2	100.2	5:50	6:10	1224	1162	1054
	271	do	13,685	Concrete, California pumice aggregate	2	100.4	5:30	5:55	1234	1182	1065
1	1	6 7/16 x 6 1/2 fabricated H	14,100	Concrete; wood-fibered gypsum-plastered	2½	100	6:30	6:54	1292	1148	1029
	6	do	12,800	Solid gypsum block; no fill, no plaster	2	103	2:32	2:33	1089	1038	946
	5	do	12,800	Solid gypsum block; no fill, plastered	2½	100	4:05	4:21	1065	1047	1009
2	28	8 in. 35 lb H	11,750	1:2:4 Limestone Concrete	2	96.9	3:30	6:33¾	1382	1321	1232
	28A	do	11,750	1:2:4 Limestone Concrete	2	95.2	3:31	7:09¼	1330	1274	1250
	29	do	11,750	1:2:4 Trap Rock Concrete	2	99.5	3:25	4:38½	1355	1279	1250
	30	do	11,750	1:2:4 Joliet Gravel Concrete	2	99.2	3:45	7:16	1410	1410 [1]	1320
	31	do	11,750	1:2:4 Sandstone Concrete	2	99.4	2:46	4:11½	1350	1296	1260
	32	do	11,750	1:2:5 Cinder Concrete	2	101.3	2:32	3:44	1332	1265	1200
	36	6 5/8 x 6 5/8 fabricated H	8,900	1:2:4 Trap Rock Concrete	2	100.5	3:10	3:53¼	1283	1227	1214
3	1	6 in. 20 lb H	13,685	1:2 1/2:3 1/2 Potomac River Gravel Concrete	2		2:44	3:34	1427	1411	1346 [2]
	2	do	13,685	1:2 1/2:3 1/2 Potomac River Gravel Concrete	2		2:46	3:32	1400	1342	1337 [2]
	3	do	13,685	1:3:2 Crushed Quartz (milky) Concrete	2		2:35	3:10	1346	1207	1157 [2]
	4	do	13,685	1:3:2 Crushed Quartz (clear) Concrete	2		2:17	2:50	1391	1274	1148 [2]

[1] Only one thermocouple remained at this level; others failed during the test.

[2] Average for all sections - N, M, T, and B.

References
1. Fire Tests of Columns Protected with Gypsum, J. Research NBS 10, 737 (1933).
2. Fire Tests of Building Columns, Tech. Pap. BS T184 (1921).
3. Fire Tests of Steel Columns Protected with Siliceous Aggregate Concrete, NBS Building Materials and Structures Report BMS 124.

6. DISCUSSION

6.1 Type of Aggregate

Table 3 summarizes the data from the column tests and from three previous series of tests. Those data are presented which fit the following requirements:

1. The column shafts were of H section.
2. The cross sections were either square or nearly square.
3. The thickness of the covering over the steel was 2 in. (note two exceptions).

Examination of the data reveals that with one exception the fire exposure time to the end of the expansion was greatest for the columns protected by the pumice concrete. The exception is the column protected by the wood-fibered gypsum concrete, column 1, reference 1, table 3. This column initially had 1/2 in. greater thickness of protection than the columns protected by pumice concrete; it was also subjected to higher unit stress during the test. If fire resistance is proportional to the 1.7 power of the covering thickness, this 1/2 in. additional thickness of insulation would more than compensate for the observed difference in fire exposure time required to produce the full column expansion. However, in that test the 1/2 in. of sanded gypsum plaster fell from place early and could no longer be considered effective. Even considering this, it can be said that for a given thickness, the pumice concrete seems to provide thermal protection for the steel shaft approximately equal to any of those listed.

The difference in fire-exposure time between the end of the expansion period and failure under load is greater for the limestone concrete and other dense concrete coverings than for the other coverings tested. It is evident that this additional fire resistance results from the fact that this concrete carries a large portion of the applied load, which was not the case with the pumice concrete, the gypsum concrete or gypsum block coverings shown in table 3.

Tests 28, 28A and 30 of reference 2 show greater fire resistance for the limestone-concrete covered column than for those protected by the pumice concrete. The difference between the 8-in. 34 lb H sections of these three test columns and the 6-in. 20 lb H sections of the pumice concrete protected columns has a marked effect on the fire resistance of the columns.

In general, the pumice concrete coverings protect columns by their high thermal-insulation properties. The protection afforded by limestone and other denser concretes results from moderate insulation properties coupled with the comparatively high load-carrying ability of the covering even when exposed to high temperatures.

6.2 Floor and Wall Panels

Table 4 presents a summary of the data from the tests on the floor and wall panel of pumice concrete and from one test on a wall of denser concrete. The time required for failure in this case is governed by the temperature rise of the unexposed wall surface. Both pumice concrete specimens showed greater fire resistance than that exhibited by the dense concrete wall previously tested. This again emphasizes the good thermal protection afforded by the pumice-aggregate concretes.

Table 4. Summary of data from fire tests of floor and wall panels

Test No.	Structure		Material	Exposure, percent of standard	Temperature limits	
	Type	Thickness			Avg surface	Max surface
		in.			hr:min	hr:min
268	Wall	4	1:3 1/8: 1 1/2 pumice concrete	100	5:04	4:48
269	Floor	4	"	101	3:45	2:29*
E2**	Wall	4	1:2 1/2: 3 1/2 Potomac river gravel concrete	100	1:18	1:15

* Localized hot spot caused by spalling
**Previously unpublished data

During tests of the cylinders, it was noticed that a shell of about 1/2 in. thickness around the outside of the cylinder appeared dry while the central core of the cylinder was darker in color and seemed damp. Samples were therefore removed from both the surface and interior of the cylinders and dried in an oven heated to 220°F (104°C). An attempt was made to select the samples in such a manner that they would be representative of the cylinder as a whole rather than either the surface or the interior. The weight loss of the samples was 18, 32 and 21 percent of the weight of the dried concrete made from aggregates from New Mexico, Oregon and California, respectively.

The effectiveness of the coverings in providing protection was probably influenced by the amount of free water retained during the seasoning period. Therefore, the differences in protection provided by the concretes made with the three pumice aggregates can be attributed in part to the differences in moisture remaining in the column encasements. Since the failure time of the columns protected by the concretes made with pumice aggregates and the moisture contents of these concretes were in the same order, it may be concluded that the differences in time of failure of these columns can be attributed in part to the differences in moisture content, and that there is likely to be little difference in the fire-resisting properties of the concretes made with the three pumice aggregates. The low strength and porous nature of the New Mexico pumice cylinders, as shown in table 2, are evidence of the poor workability of the concrete without the air entraining agent and soaking of the aggregates before mixing the concrete.

7. CONCLUSIONS

The high moisture content discovered in testing the cylinders, while probably representative of conditions in the concrete coverings for the columns, is not considered representative of conditions to be expected in buildings which have been in service for several years. Consequently, the fire-resistant performance observed in these tests was higher than would be expected from aged installations. This has been recognized in arriving at the following conclusions:

1. A column of 6 in. H section steel protected by pumice-aggregate concrete as described will resist fire for 4 hours or more before failure occurs under load.

2. Protection of steel columns afforded by pumice-aggregate concretes is mainly of a thermal character. It is probable that the concrete does not carry a significant portion of the load at the time of failure from fire exposure.

3. Taking into consideration lower moisture content and the harsh mix characteristics of the concrete made with the pumice from New Mexico, it will be found that there is likely to be little, if any, difference in the fire resisting properties of concretes made with the pumices from New Mexico, Oregon or California.

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Tests Nos.267, 270 and 271

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