FIRE RESISTANCE OF POST-TENSIONED STRUCTURES

Armand H. Gustaferro
Consulting Engineer
The Consulting Engineers Group, Inc.
Glenview, Illinois

Reports on an analysis of 18 full scale fire tests of concrete slabs and beams prestressed by post-tensioning. In addition, results of tendon-anchor assembly tests performed at high temperatures were analyzed. Recommendations on minimum cover thicknesses and member sizes are included for fire endurances of 1, 2, 3, and 4 hr.

More than 140 full scale fire tests of prestressed concrete structural components have been conducted in the United States. In addition, researchers have studied the properties of steel and concrete subjected to high temperatures. They have also measured and calculated analytically the temperatures that occur in structural components during fires. Currently, methods are being developed to calculate the capacity, deflection, expansion, rotation, and other characteristics of structures subjected to fire.

Scope

The purpose of this report is to present an overview of pertinent information concerning the fire resistance of structures with post-tensioned reinforcement. Information has been gathered from a number of sources. Results of fire tests of 18 slabs and beams with post-tensioned reinforcement constitute a major source. These results were compared with other information, i.e., tendon temperature data were compared with data on temperatures within unreinforced slabs and beams, and then the resulting fire endurances were analyzed. Because the temperatures of the tendons were in all cases cooler than temperatures at comparable locations in unreinforced slabs, it was possible to make conservative recommendations on minimum dimensions for various fire endurances. In addition, results of recent high temperature tests of tendon-anchor assemblies makes it possible to determine realistic cover thicknesses for anchors.

Standard fire tests of building construction and materials (ASTM E119)[1]

The fire resistive properties of building components are measured and specified according to this common standard. Performance is defined as the period of exposure to a standard fire before the first critical "end point" is reached.

The standard fire exposure is defined in terms of a time versus temperature relation. At 5 min the furnace atmosphere temperature is 1000 F, at 30 min 1550 F, at 1 hr 1700 F, at 2 hr 1850 F, and at 4 hr 2000 F. The fire represents combustion of about 10 lb of wood (with a heat potential of 8000 BTU per lb) per sq ft of exposure area per hour of test. Actually, the fuel consumed during a fire test is dependent on the furnace design and on the heat capacity of the test assembly. For example, the amount of fuel consumed during a fire test of an exposed concrete floor specimen is likely to be 10 to 20 percent greater than that used for a test of a floor with an insulated ceiling, and considerably greater than that for a combustible assembly.

The standard, ASTM E119, specifies minimum sizes of specimens to be exposed in fire tests. For floors and roofs, at least 180 sq ft must be exposed to fire from beneath, and neither dimension can be less than 12 ft. For tests of walls, either load-bearing or non-load-bearing, the minimum specified area is 100 sq ft with neither dimension less than 9 ft. The minimum length for columns is specified to be 9 ft, while for beams it is 12 ft.

During fire tests of floors, roofs, beams, load-bearing walls, and columns, the maximum permissible superimposed load is applied. Floor and roof specimens are exposed to fire from beneath, beams from the bottom and sides, walls from one side, and columns from all sides.

End point criteria for floors and roofs are:

(a) Specimens must sustain the applied loading—collapse is an obvious end point.

(b) Holes, cracks, or fissures through which flames or gases hot enough to ignite cotton waste must not form.

(c) The temperature of the unexposed surface must not rise an average of 250 F or a maximum 325 F at any one point.

In 1970, new end point criteria were tentatively added to ASTM E119 for floors, roofs, and beams fire tested in a "restrained" condition. "Restrained" in this case means that thermal expansion of the specimen is restricted during the fire test. Two classifications can be derived from fire tests of restrained specimens, "unrestrained" and "restrained." Only "unrestrained" assembly classifications can be obtained from tests of unrestrained specimens. The tentative revision of ASTM E119 includes a guide for classifying constructions as restrained or unrestrained. In the guide, cast-in-place and most precast concrete constructions are considered to be restrained.

The new end point criteria are based on critical steel temperatures. For structural steel and reinforcing bars the critical average temperature is 1100 F while for cold-drawn prestressing steel it is 800 F.

For unrestrained members, the fire endurance is the time at which the critical temperature is reached. For restrained primary beams, which are defined as beams spaced more than 4 ft on centers, the fire endurance is twice the time at which the critical temperature is reached. For restrained slabs and for restrained beams spaced 4 ft or less on centers, the steel temperatures are disregarded.

Rational design procedures

It was noted above that methods are currently being developed for calculating various parameters concerning the behavior of structures during fires. Even though it is not the intent of this report to present a comprehensive treatment of rational design procedures for fire endurance, a review of some of the principles involved may help in understanding the behavior of structures subjected to fire. For illustration the behavior during fire of three types of flexural members will be discussed briefly.

(a) *Simply supported slabs or beams.* Consider a simply supported reinforced concrete slab subjected to fire from below. Assume that the ends of the slab are free to rotate and expansion can occur without restriction. Assume also that the reinforcement consists of straight bars located near the bottom of the slab. With the underside of the slab exposed to fire, the bottom will expand more than the top, and the slab will deflect. Also, the strength of the concrete and steel near the bottom of the slab will decrease as the temperature increases. When the strength of the steel is reduced to that of the stress in the steel, flexural collapse will occur. Such behavior has been clearly demonstrated in prestressed as well as reinforced concrete members.[2]

It is apparent from the above description that the steel temperature at which collapse occurs depends on (1) the stress in the steel, and (2) the type of steel.

The stress in the steel depends on the load intensity on the member. For example, if the steel stress is 50 percent of the initial yield strength, the critical temperature will be about 1120 F. However, if the steel stress is one-third of the yield strength, the critical temperature will be about 1220 F. The temperatures would be different for cold-drawn steel or high strength alloy steel bars. Thus, it can be seen that if the load intensity is decreased the fire endurance will increase. Through rational design procedures, it is possible to estimate the increase in fire endurance due to a decrease in load intensity.

(b) *Continuous slabs and beams.* Structures that are continuous or otherwise statically indeterminate, undergo changes in stresses when subjected to fire.[3] It should be noted that this is different than simply supported members where the applied moments at a section remain constant during fire exposure.

Consider a two-span continuous slab with rocker-rollers at the outer supports. During fire exposure from beneath, the underside of the slab expands more than the top. This differential heating causes the ends of the slab to tend to lift from the outer supports thus increasing the reaction at the interior support. This action results in a redistribution of moments, i.e., the negative moment at the interior support increases while the positive moments decrease.

During the course of a fire, the negative moment reinforcement remains cooler than the positive moment reinforcement because it is further from the fire. Thus, the increase in negative moment can be accommodated. The resulting decrease in positive moment means that the positive moment steel can withstand a higher temperature before failure will occur. Thus the fire endurance of a continuous member is generally significantly longer than that of a simply supported member having the same cover and load intensity.

(c) *Members in which restraint to thermal expansion occurs.* If a fire occurs beneath a small interior portion of a large reinforced concrete slab, the heated portion will tend to expand and push against the surrounding part of the slab. In turn, the unheated part of the slab exerts compressive forces on the heated portion. The compressive force, or thrust, acts near the bottom of the slab when the fire first occurs, but as the fire progresses the line of action of the thrust rises as the heated concrete deteriorates.[4] If the surrounding slab is thick and heavily reinforced, the thrust forces that occur can be quite large, but considerably less than that calculated by use of elastic prop-

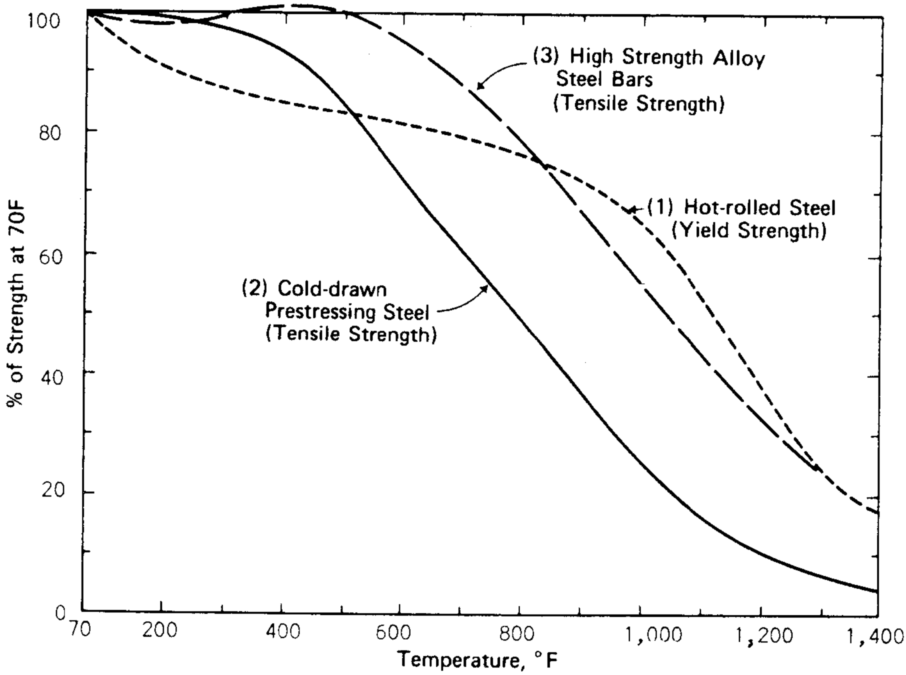


Fig. 1. Temperature-strength relation for hot-rolled, cold-drawn, and high strength alloy steels. (Curves 1, 2, and 3 from References 5, 6, and 7, respectively.)

erties of concrete and steel together with appropriate coefficients of expansion. At high temperatures, creep and stress relaxation play an important role. Nevertheless, the thrust is generally great enough to increase the fire endurance significantly. In most fire tests of restrained assemblies, the fire endurance is determined by temperature rise of the unexposed surface rather than by structural considerations, even though the steel temperatures often exceed 1500 F.

PROPERTIES OF STEEL AND CONCRETE AT HIGH TEMPERATURES

Physical properties of steel and concrete are affected by the temperatures encountered in fires. Strength, modulus of elasticity, expansion, thermal conductivity, creep, and stress relaxation are all affected to some degree. Insofar as ultimate capacity during fires is concerned, strength is of primary importance.

Steel strength at high temperatures

Fig. 1 shows typical relations between temperature and strength for hot-rolled steel, i.e., reinforcing bars, cold-drawn prestressing steel, i.e., wire or strand, and high strength alloy steel bars.[5,6,7] Note that one-half of the strengths are retained at about 800 F for cold-drawn steel, 1050 F for alloy steel bars, and 1120 F for hot-rolled steel.

Concrete strength at high temperatures

Fig. 2 shows the temperature-

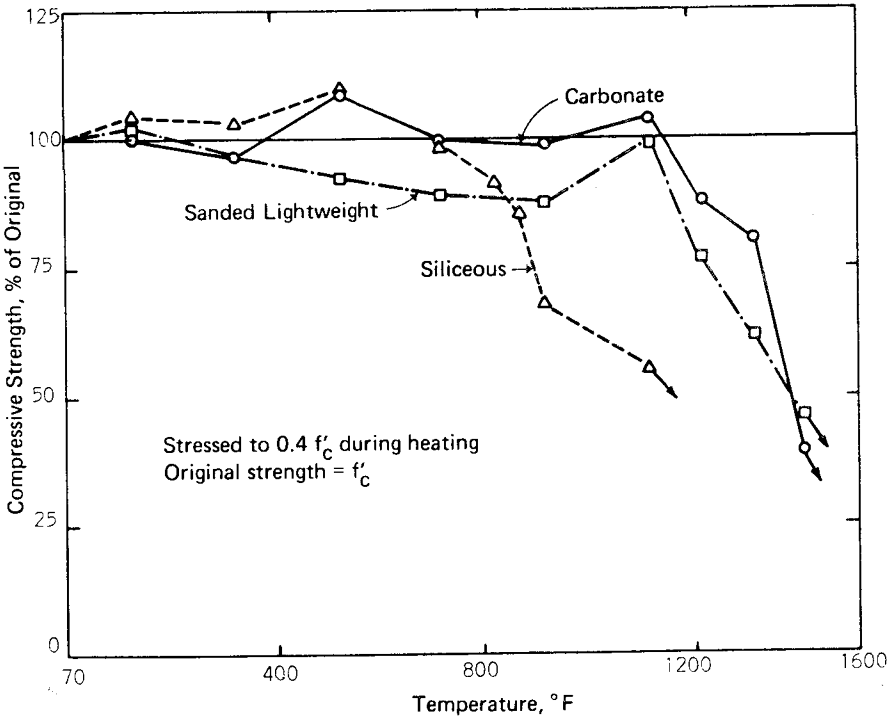


Fig. 2. Compressive strength of concrete at high temperatures (Reference 8)

strength relationships for three kinds of concrete.[8] Carbonate aggregates include limestone and dolomite which undergo a chemical change at temperatures above about 1300 F, i.e., carbon dioxide is given off from the calcium and magnesium carbonates. Heat is used up during the reaction so the temperatures within the concrete remain somewhat lower than for noncarbonate aggregates. Also, the resulting products are better insulators than the original aggregates. Siliceous aggregates include quartzite, granite, and sandstone. The data for sanded lightweight concrete shown in Fig. 2 represent concretes with a unit weight in the range of 105 to 115 lb per cu ft. Note that at 800 F, most concretes retain most of their original strength and at 1200 F carbonate and lightweight concretes have nearly all of their original strengths. Siliceous aggregate concrete retains more than one-half its initial strength at 1200 F.

RESULTS OF 18 STANDARD FIRE TESTS CONDUCTED IN THE UNITED STATES

Data have been published from a total of 18 fire tests conducted in the United States on post-tensioned prestressed concrete slabs and beams. Two fire tests of slabs were conducted by the Fire Prevention Research Institute in Gardena, California.[9,10] Underwriters' Laboratories, Inc., Northbrook, Illinois, conducted three tests of post-tensioned structures, one was a slab[11] and two were inverted tee beams.[12] The Portland Cement Association fire tested seven post-tensioned beams,[7] all of which were modified tee beams spanning 40 ft. Six tests conducted by the National Bureau of Standards in 1953 are of historical interest,[13] and were part of a series sponsored by the British Joint Fire Research Organization and the Building Research Station.

FPRI tests

References 9 and 10 give pertinent data about the Fire Prevention Research Institute tests. Both tests involved normal weight concrete slabs, 6 in. thick, made with siliceous aggregates and post-tensioned unbonded tendons. One of the specimens was an integral beam-and-slab assembly; the other was a flat plate floor. The beams were prestressed longitudinally and the slab was prestressed transversely with moderate longitudinal prestress. The minimum clear cover was 1½ in. for the slab tendons and 2 in. for the beam tendons. In the other specimen, the 6-in. slab was prestressed with post-tensioned tendons in two directions. The minimum cover at midspan was 1½ in.

Both assemblies were mounted in fixed restraining frames during the fire tests. Structural end points were not reached during the tests which lasted more than 4 and 3 hr, respectively. The end point for the first test occurred at 3 hr 51 min when the unexposed surface temperature rose an average of 250 F. Although the second test was stopped before an end point was reached, the heat transmission end point would have been reached at about 3 hr 15 min.

UL tests

Reference 11 gives pertinent details of the fire test of a lightweight concrete post-tensioned flat plate floor conducted by the Underwriters' Laboratories. Duration of the test was 3 hr 45 min with no end point occurring. The specimen had been dried for 7 months at high temperatures prior to the test and the moisture content of the concrete was low. Based on the correction procedure for nonstandard moisture content (Appendix A5 of ASTM E119-71), the heat transmission end point would have occurred at about 4 hr 40 min. No spalling of the specimen occurred.

Reference 12 refers to fire tests of inverted tee beams prestressed with post-tensioned tendons. In one specimen the tendon was bonded while in the other the tendon was unbonded. The superimposed load on the unbonded specimen was substantially lower than the load on the bonded specimen. Both tests were terminated at 4 hr 15 min even though no end point

Table 1. Data from PCA Tests (Reference 7)

Beam No.	Type of Reinforcement	Bonded or Unbonded	Type of Concrete	Superimposed Load, lb. per ft.	Fire Endurance hr.:min.
80	Bars	Unbonded	Normal weight	1040	5:02
82	Bars	Bonded	Normal weight	1535	4:29
83	Bars	Bonded	Lightweight	1680	5:01
76	Wires	Unbonded	Normal weight	1135	3:04
78	Wires	Bonded	Normal weight	1750	3:20
79	Wires	Bonded	Lightweight	1740	4:33
89	Wires	Bonded	Normal weight	1760	3:18

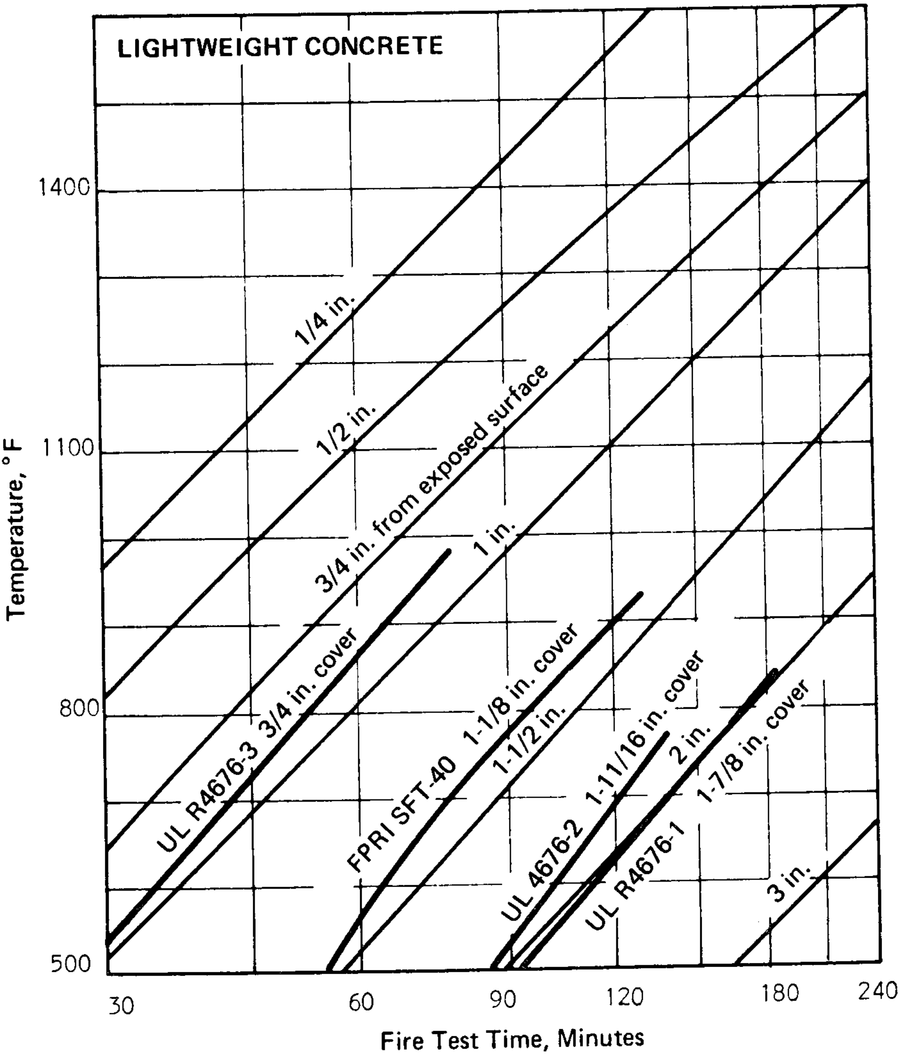


Fig. 3. Temperatures within concrete slabs during fire tests (expanded shale aggregates) showing strand temperatures in hollow-core slabs

was reached. At the ends of the tests the midspan deflections were about 1 in. for the 17 ft 5 in. spans. A companion pretensioned specimen was also fire tested in the same series. The behavior of the pretensioned specimen was similar to that of the post-tensioned companions.

PCA tests

As part of a broad series of fire tests, the Portland Cement Association fire tested seven 40-ft beams in which the reinforcement was post-tensioned.[7] Two types of reinforcement were used, high strength alloy steel bars and cold-drawn wires with button heads. Beams were essentially rectangular, 14 in. wide, 25 in. deep, with 6 x 4-in. flanges. Tendon cover at midspan was 2½ in. Table 1 gives some pertinent data about the specimens and tests.

Beams were simply supported on rocker-roller supports to minimize restraint to thermal expansion. Included in the series of tests were companion specimens reinforced with Grade 40 and Grade 60 bars, and three specimens with pretensioned seven-wire strand. Among the conclusions reached from the Portland Cement Association series of tests were:

1. Prestressed beams of lightweight concrete had longer fire endurances than their normal weight companions, and

2. Beams with unbonded post-tensioned reinforcement had about the same fire endurances as their counterparts with bonded reinforcement.

NBS tests

As noted above, the six tests con-

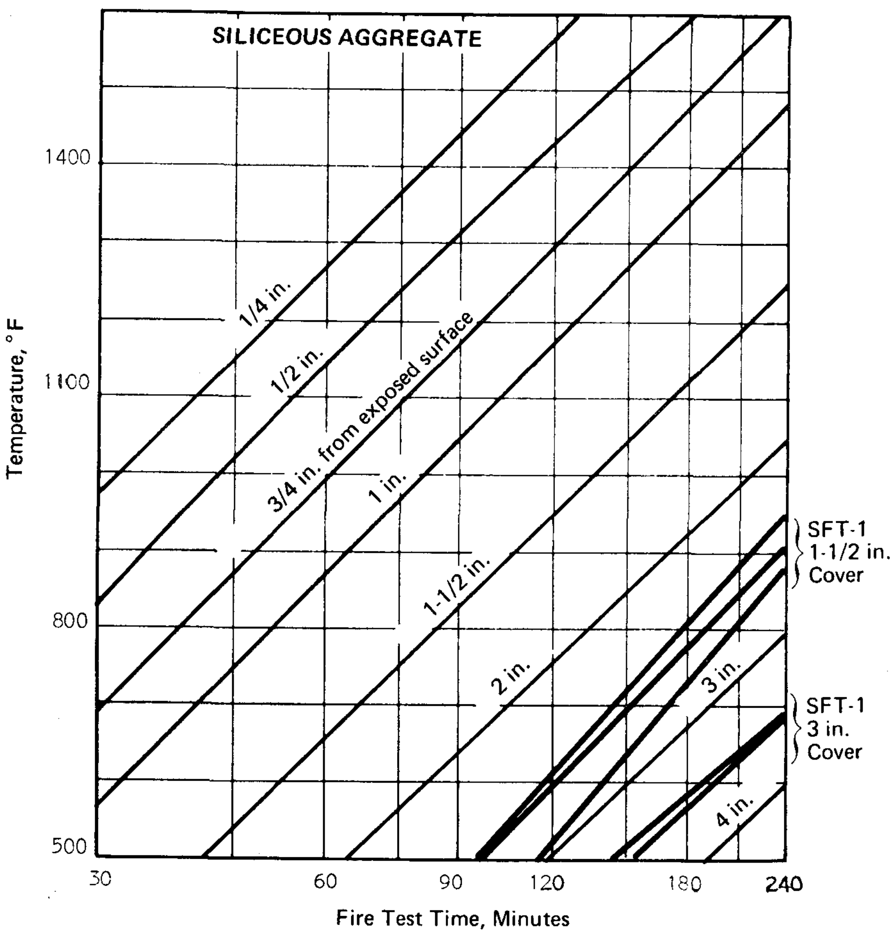


Fig. 4. Temperatures within concrete slabs during fire tests (siliceous aggregates) showing slab tendon temperatures in FPRI-SFT-1

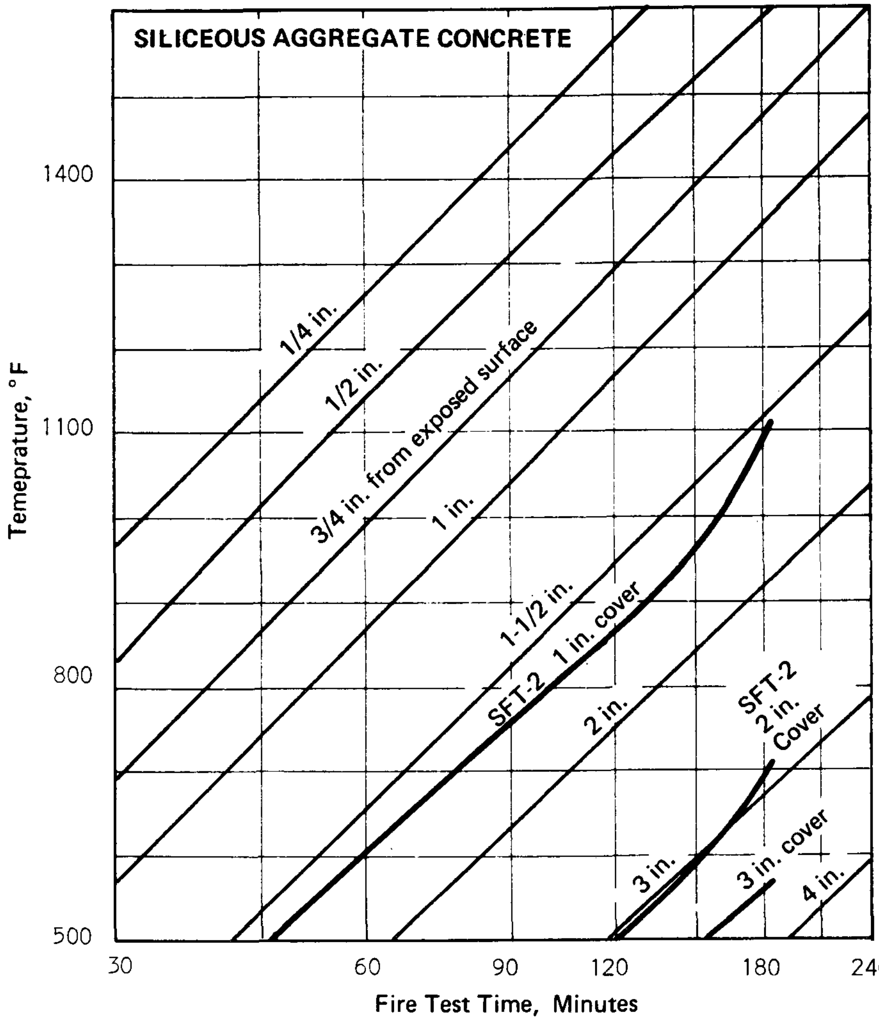


Fig. 5. Temperatures within concrete slabs during fire tests (siliceous aggregate) showing tendon temperatures in FPRI-SFT-2. (Adjusted for furnace temperature lag.)

ducted at the National Bureau of Standards in 1953 involved beams manufactured in England and fire tested in accordance with the 1932 edition of British Standard 476. The test procedure of BS476-32 is similar to that of ASTM E119 except for one major difference. The loading requirement of BS476-32 called for a superimposed load of one and one-half times the design live load rather than one times the live load. Recent editions of BS476 have revised that requirement to one times the live load. This stipulation is the same as ASTM E119. Thus, the fire endurances of the six National Bureau of Standards tests were probably significantly shorter than might be expected if the normal loading had been applied. The beams were rectangular with or without a composite slab. The steel consisted of wires, 0.1 or 0.2-in. diameter, post-tensioned and grouted. The span was either 10 ft or 16 ft and beams were simply supported. Two beams were coated with 1 in. thick vermiculite concrete. Fire endurances (not adjusted for loading) ranged between about 1½ and 6 hr.

ANALYSIS OF TEST DATA

In 1971, Underwriters' Laboratories developed criteria for fire resistance classifications for precast prestressed concrete units. The classifications were based on results of standard fire tests of hollow-core slabs, various stemmed units, and inverted tee beams. The new criteria are based on the tentative revisions of ASTM E119-71, i.e., for simply supported unrestrained members with cold-drawn prestressing steel the fire endurance is the time required for the steel to reach 800 F during a standard fire test. For restrained beams spaced more than 4 ft on centers, the fire endurance is twice the time required for the steel to reach that temperature. For other restrained units, the steel temperature is disregarded and the structural end point governs.

In the Underwriters' Laboratories studies that led to the classification criteria included in the January 1972 "Fire Resistance Index,"[14] measured steel temperatures were compared with each other and with published data on temperatures within concrete members during fire tests. For example, steel temperatures during full scale fire tests of 14 hollow-core slabs were compared with temperatures measured within plain concrete slabs. Data from four of the tests of lightweight concrete slabs are shown in Fig. 3. The basic slab temperature data on which the charts are based were developed in the test program described in *PCA Research Department Bulletin* 223.[15] Charts similar to Fig. 3 for carbonate and siliceous aggregate concretes were also prepared and analyzed. Note that the measured steel temperature curves are roughly parallel to the slab temperature curves. In each case the steel temperatures are somewhat lower than those estimated from the slab data for the same distance from the exposed surface. Thus, the cover requirements based on the slab concrete temperatures are slightly conservative. Based on Fig. 3, the cover requirements for unrestrained lightweight prestressed concrete slabs are approximately 1 in. for 1 hr, 1⅝ in. for 2 hr, and 2 in. for 3 hr.

A similar, though more complex, procedure was used in analyzing the data for stemmed units and inverted tee beams.

The procedures used by the Underwriters' Laboratories in developing their classification criteria are essentially those used below for analyzing slabs and beams.

Analysis of slab data

Pertinent steel temperature data from the three tests of slabs are shown in Figs. 4, 5, and 6. Fig. 4 shows the temperatures of the tendons in the beam-and-slab assembly (FPRI SFT-1). Note that the temperature of the slab tendons with 1½-in. cover reached an average of 800 F after 3 hr of fire exposure. This temperature is far lower than would be expected for 1½-in. cover based on the concrete slab temperature data. In fact, the beam steel temperatures were lower than the 2-in. line on the plot. The low recorded temperatures might be due to either or both of the following items. First, the recorded temperatures reflect the average temperature of the tendon rather than the maximum temperature that would occur at the bottom of the tendon. Second, the tendons were greased and

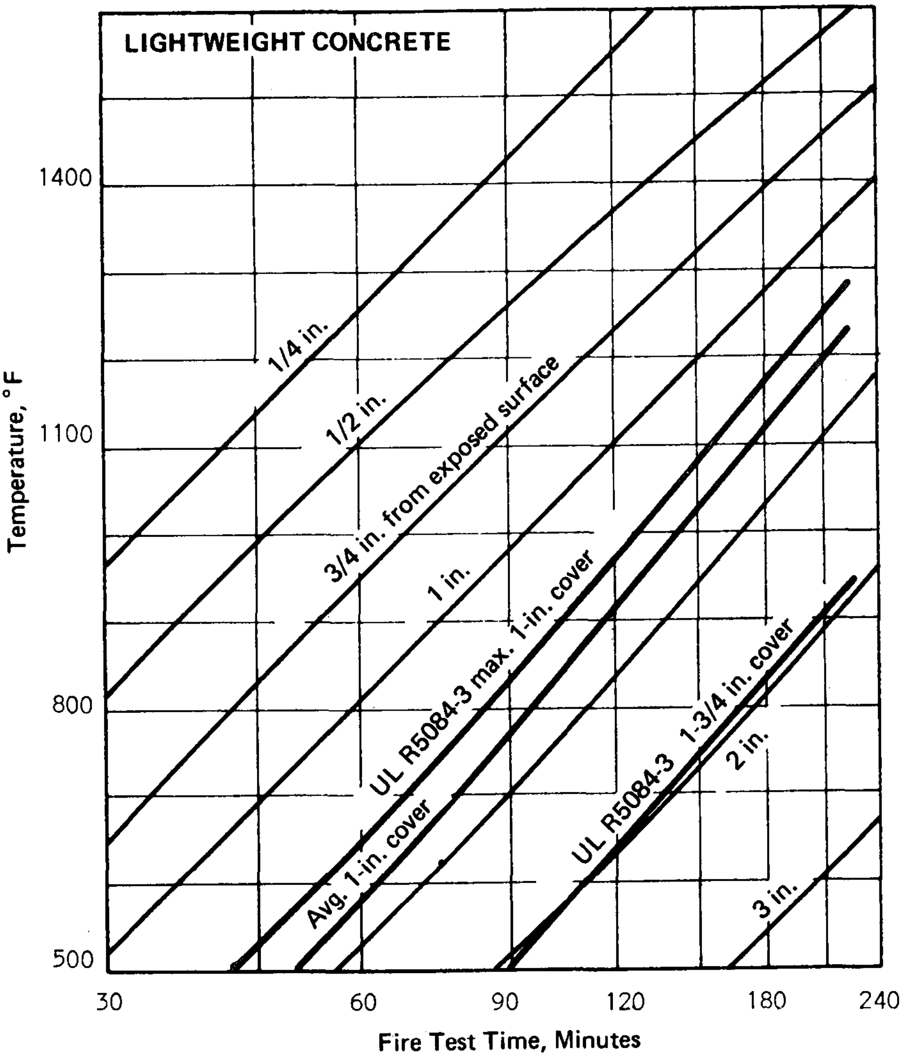


Fig. 6. Temperatures within concrete slabs during fire tests (expanded shale aggregates) showing tendon temperatures in UL 5084-3

wrapped and the lubricant and wrapping materials might have kept the tendons cooler during fire exposure.

Fig. 5 shows the tendon temperatures recorded during the test of the normal weight concrete flat plate floor (FPRI SFT-2). The curvilinear shape of the temperature curves may be due to the furnace atmosphere temperatures which were somewhat low during the first 2½ hr and high thereafter. Again, the recorded steel temperatures were lower than would be expected from the data on which the curves are superimposed.

Fig. 6 shows the tendon temperatures recorded during the test of the lightweight concrete flat plate floor

Table 2. Summary of Data on Slab Tendons

Test	"A" Cover, in.	Test Time to Reach 800F, hr:min	"B" Corresponding Distance from Exposed Surface, in.	"B"–"A" in.
FPRI-1	1-1/2	3:12	2-5/8	1-1/8
FPRI-1	3	5:07*	3-5/8	5/8
FPRI-2	1	1:44	1-3/4	3/4
FPRI-2	2	3:30*	2-5/8	5/8
UL-R5084-5	1	1:31	1-3/8	3/8
UL-R5084-5	1-3/4	2:47	2-1/4	1/2

*Extrapolated

(UL R5084-3) superimposed on a plot of temperatures within lightweight concrete slabs during fire tests. Again the temperatures are lower, but to a lesser extent than those in Figs. 4 and 5, possibly because the specimen was kiln-dried prior to the test.

In an attempt to determine required cover thickness for unrestrained slabs with post-tensioned tendons, an analysis can be made of test times at which the tendons reached 800 F. These values can be compared with corresponding test times at which concrete at various levels reaches 800 F during fire tests. Table 2 provides a basis for comparison.

The values of "B" in Table 2 are the distances from the exposed surface in plain concrete slabs at which the temperature is 800 F at the test time indicated in the third column. The values of "B"-"A" in the last column indicate the magnitude of reduction of cover possible. Note that those values range from 3/8 to 1⅛ in. Thus a reduction of at least 3/8 in. is warranted. Resulting cover thicknesses for simply supported unrestrained slabs with post-tensioned reinforcement, based on temperatures within slabs with a 3/8-in. reduction, are given in Table 3.

Cover requirements shown in Table 3 apply to tendons ½-in. or larger in size. The values for 1 hr for carbonate and lightweight aggregate concretes are governed by minimum cover requirements for slabs (see the provisions in ACI 318-71).

Analysis of beam data

Fig. 7 shows tendon temperatures at

Table 3. Cover Requirements for Unrestrained Slabs with Post-Tensioned Reinforcement

Aggregate Type	Cover Thickness, in., for Fire Endurance of 1 hr	1-1/2 hr	2 hr	3 hr
Carbonate	3/4	1-1/16	1-3/8	1-7/8
Siliceous	3/4	1-1/4	1-1/2	2-1/8
Lightweight	3/4	1	1-1/4	1-5/8

midspan of the three inverted tee beams fire-tested at Underwriters' Laboratories. It is interesting to note that the temperatures of the pretensioned strand (UL R4123-12) correspond to the temperatures that might be anticipated for strand centered about 2¼ in. above the bottom of a slab even though the cover was only 1¾ in. and the strands were in a beam rather than in a slab. Temperatures of the post-tensioned tendons (UL R4123-12A) were lower yet, possibly because the bonded and unbonded tendons were centered 2¾ and 2½ in., respectively, above the bottoms of the beams. Nevertheless, the

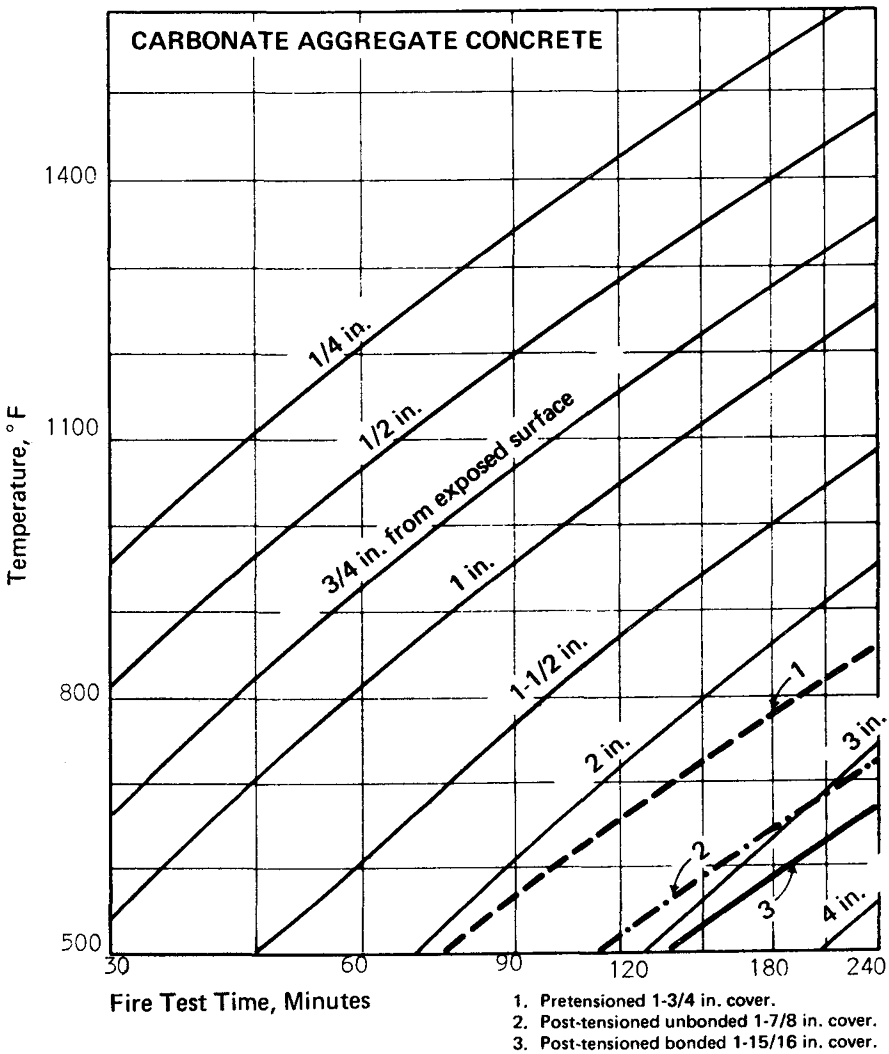


Fig. 7. Temperatures within concrete slabs during fire tests (carbonate aggregate) showing tendon temperatures in inverted T beams (UL 4123-12-12A)

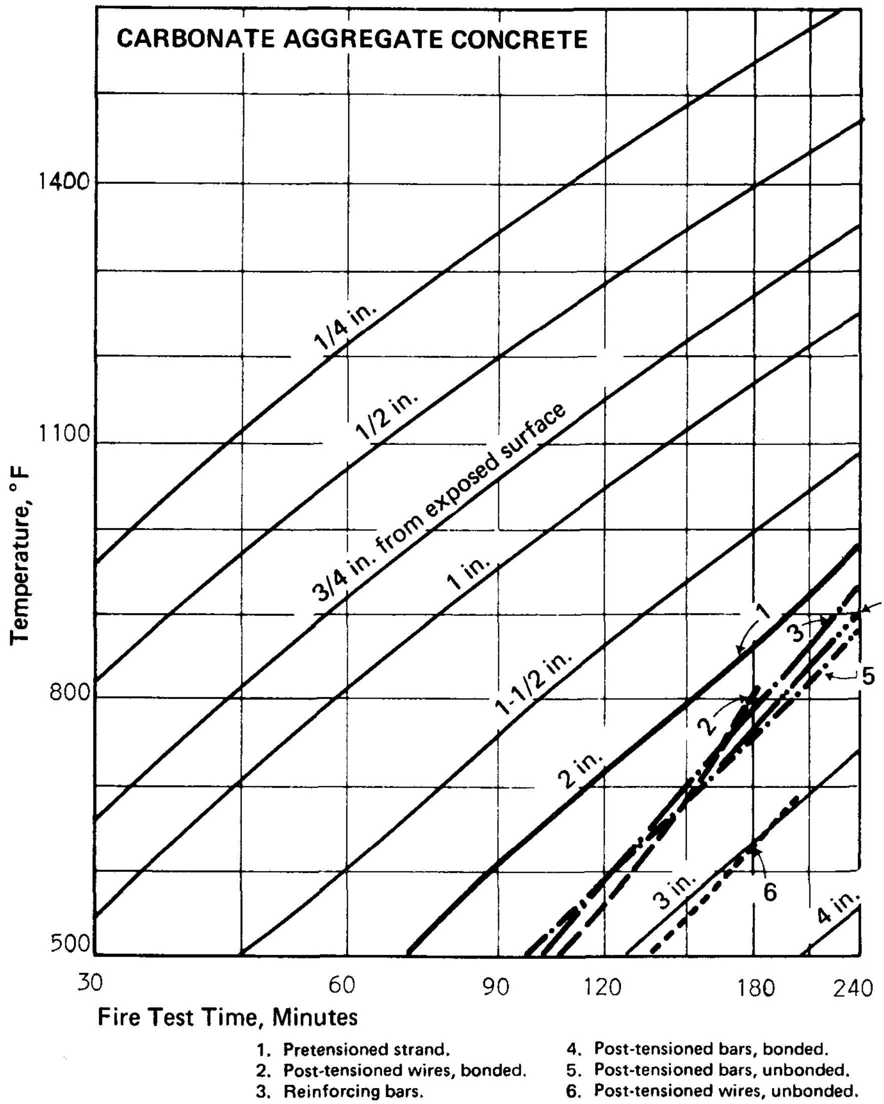


Fig. 8. Temperatures within concrete slabs during fire tests (carbonate aggregates) showing temperatures of corner bars, wires, or strand during PCA tests of 40-ft beams

temperatures correspond to those of about 3¼ and 3 in. above the bottom of a slab. The low temperatures for the bonded tendon might result from the insulation afforded by the high water content of the grout within the duct.

Figs. 8 and 9 show temperatures of the corner bars or strands of the 40-ft beams tested at the Portland Cement Association. The temperatures shown represent the maximum bar or strand temperatures because the corner bars or strands, i.e., those with 2½ in. side and bottom cover, were the hottest in

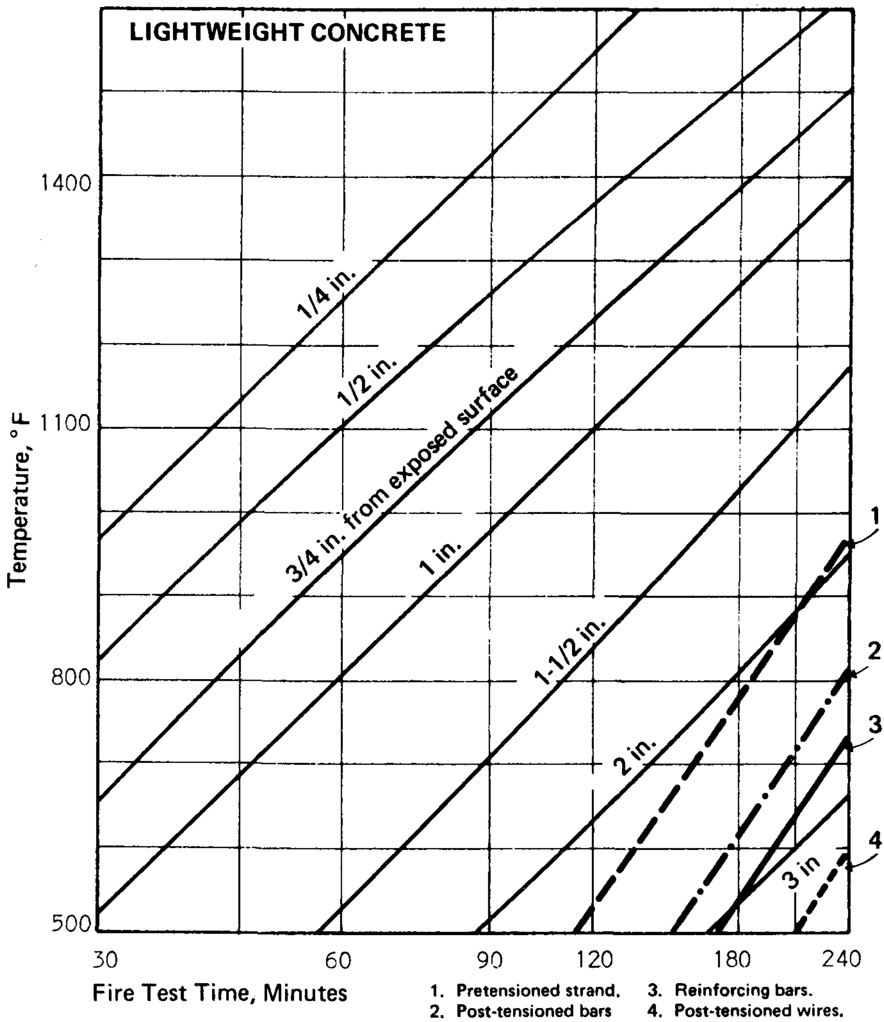


Fig. 9. Temperatures within concrete slabs during fire tests (expanded shale aggregates) showing temperatures of corner bars, or strand during PCA tests of 40-ft beams

each of the tests. Note that in Fig. 8 (normal weight concrete) the time-temperature relations for the post-tensioned bars, bonded or unbonded, the reinforcing bars, and the unbonded post-tensioned wires are grouped closely together. Temperatures of the corner pretensioned strands were higher and those of the bonded post-tensioned bonded wires were lower. The same approximate relationships are also true for the lightweight concrete specimens (see Fig. 9).

From Figs. 8 and 9 it can be seen that the corner bar temperatures of the post-tensioned units are essentially the same as (or lower than) slab temperatures at a distance of about 2½ in. from

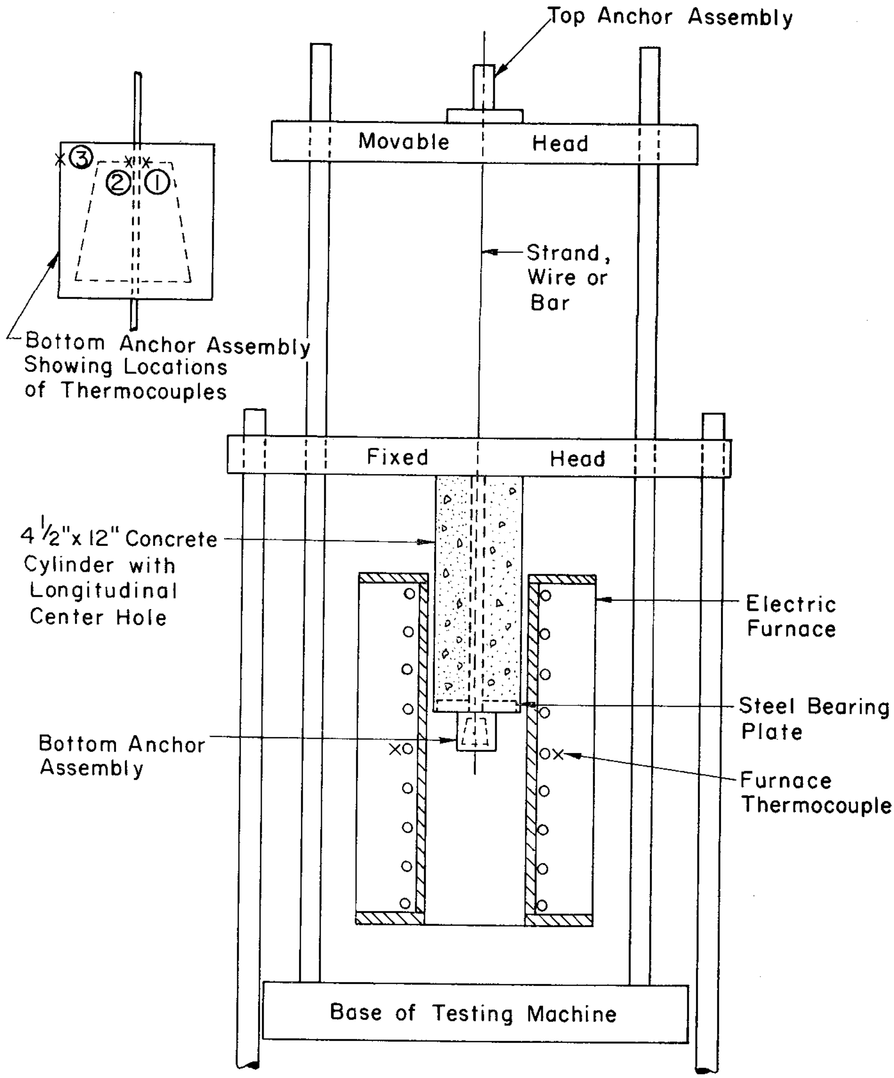


Fig. 10. Arrangement for high temperature tests of tendon-anchor assemblies

the exposed surface. Since that was the cover of the corner bars, the cover requirements for slabs, as determined from the Portland Cement Association concrete slab data, should be adequate for beams with dimensions roughly comparable to those tested. On this basis, for beams with post-tensioned reinforcement wider than about 12 in. the cover requirements for unrestrained classifications would be those shown in Table 4.

The above tabulation assumes that the minimum cover would be 1½ in. for all beams, and that tendons are ½-in. or larger in size. For narrower beams the cover would have to be somewhat greater in some cases. For

Table 4. Cover Requirements for Unrestrained Beams at Least 12-in. Wide and Prestressed with Post-Tensioned Reinforcement

Steel Type	Concrete Type*	For Beams at Least 12-in. Wide, Cover Thickness, in., for Fire Endurance			
		1 hr	2 hr	3 hr	4 hr
Cold-Drawn	NW	1-1/2	2	2-1/2	3
Cold-Drawn	LW	1-1/2	1-3/4	2	2-1/2
H.S.A. Bars	NW	1-1/2	1-1/2	1-1/2	2
H.S.A. Bars	LW	1-1/2	1-1/2	1-1/2	2

*NW = normal weight; LW = lightweight

beams 8 in. wide, comparable cover requirements could be those shown in Table 5. For beams with widths between 8 and 12 in., cover requirements can be obtained by direct interpolation. For example, for a 10-in. wide beam of lightweight concrete with cold-drawn steel, the cover for 3 hr would have to be 2⅝ in.

The values for 8-in. wide beams were derived from the relationships of cover, beam width, and temperature based on results of tests at the Portland Cement Association and the Underwriters' Laboratories, some of which have not yet been published. For cold-drawn steel, a temperature limit of 800 F was used. For high strength alloy steel bars, a temperature limit of 1000 F was used, making the results somewhat conservative.

Protective coatings

A 1972 report[16] gives analyses of fire tests of slabs, beams, and joists and concludes with recommended thicknesses of sprayed insulation for prestressed units. The thicknesses of sprayed mineral fiber, vermiculite Type MK, or intumescent mastic are given for slabs and beams of various widths

Table 5. Cover Requirements for Unrestrained 8-in. Wide Beams Prestressed with Post-Tensioned Reinforcement

Steel Type	Concrete Type	For Beams 8-in. Wide, Cover Thickness, in., for Fire Endurance			
		1 hr	1-1/2 hr	2 hr	3 hr
Cold-Drawn	NW	1-3/4	2	2-1/2	4-1/2*
Cold-Drawn	LW	1-1/2	1-3/4	2	3-1/4
H.S.A. Bars	NW	1-1/2	1-1/2	1-1/2	2-1/2
H.S.A. Bars	LW	1-1/2	1-1/2	1-1/2	2-1/4

*Not practical but shown for interpolation purposes.

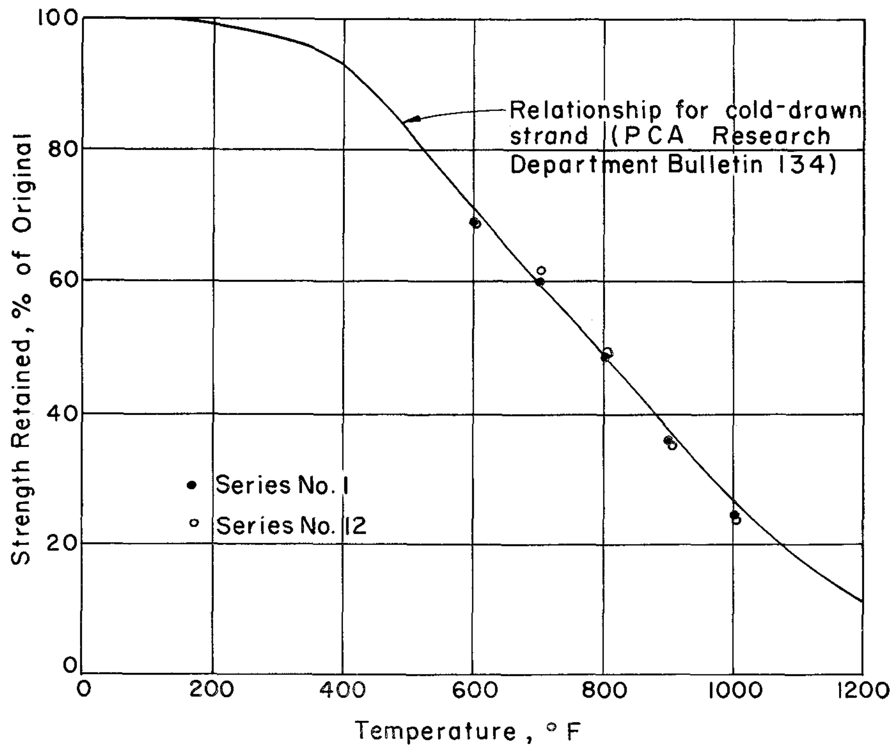


Fig. 11. Relation between temperature and tensile strength of cold-drawn strand (from Abrams and Cruz) together with test results of tendon-anchor assemblies

and concrete cover thicknesses.

Even though none of the tests analyzed in that report were of members with post-tensioned reinforcement, the data should be directly applicable to any member with cold-drawn prestressing steel. It should be noted that two of the National Bureau of Standards tests were of specimens coated with vermiculite concrete. In each case the fire endurance of the coated specimen was more than double that of its uncoated counterpart. The data are not directly applicable for beams or slabs with high strength alloy steel bars, but would be conservative if applied directly.

ANALYSES OF RESULTS OF TESTS OF TENDON-ANCHOR ASSEMBLIES AT HIGH TEMPERATURES

Several tests have been performed to determine if anchors commonly used in North America for post-tensioning continue to function at temperatures that occur during fires. Reports of these tests are not readily available, so much of the pertinent data is included here.

Tensile tests of tendon-anchor assemblies

Three series of tests were conducted at the Portland Cement Association

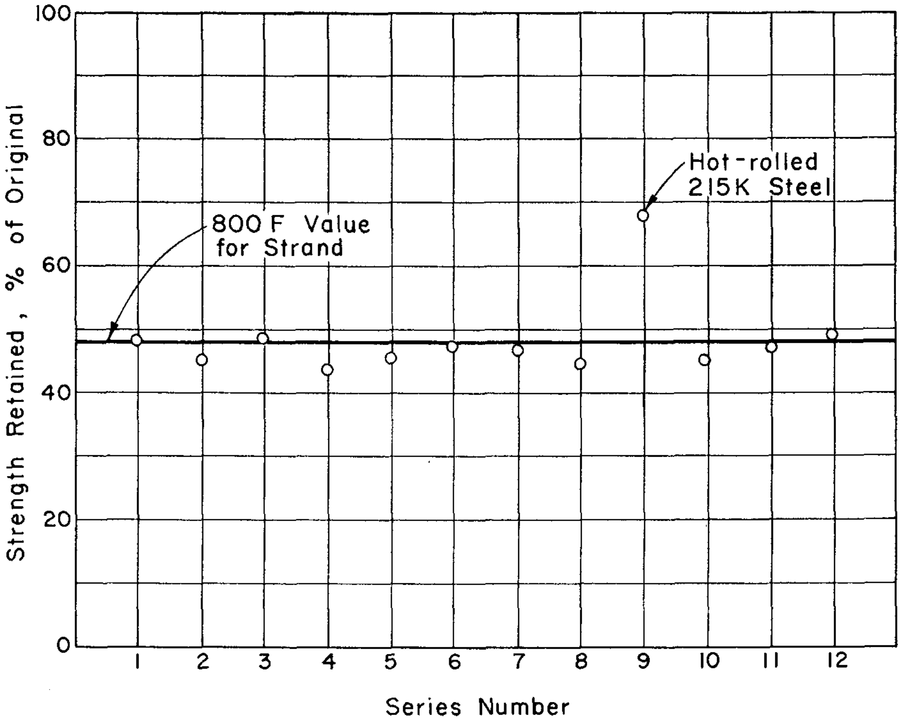
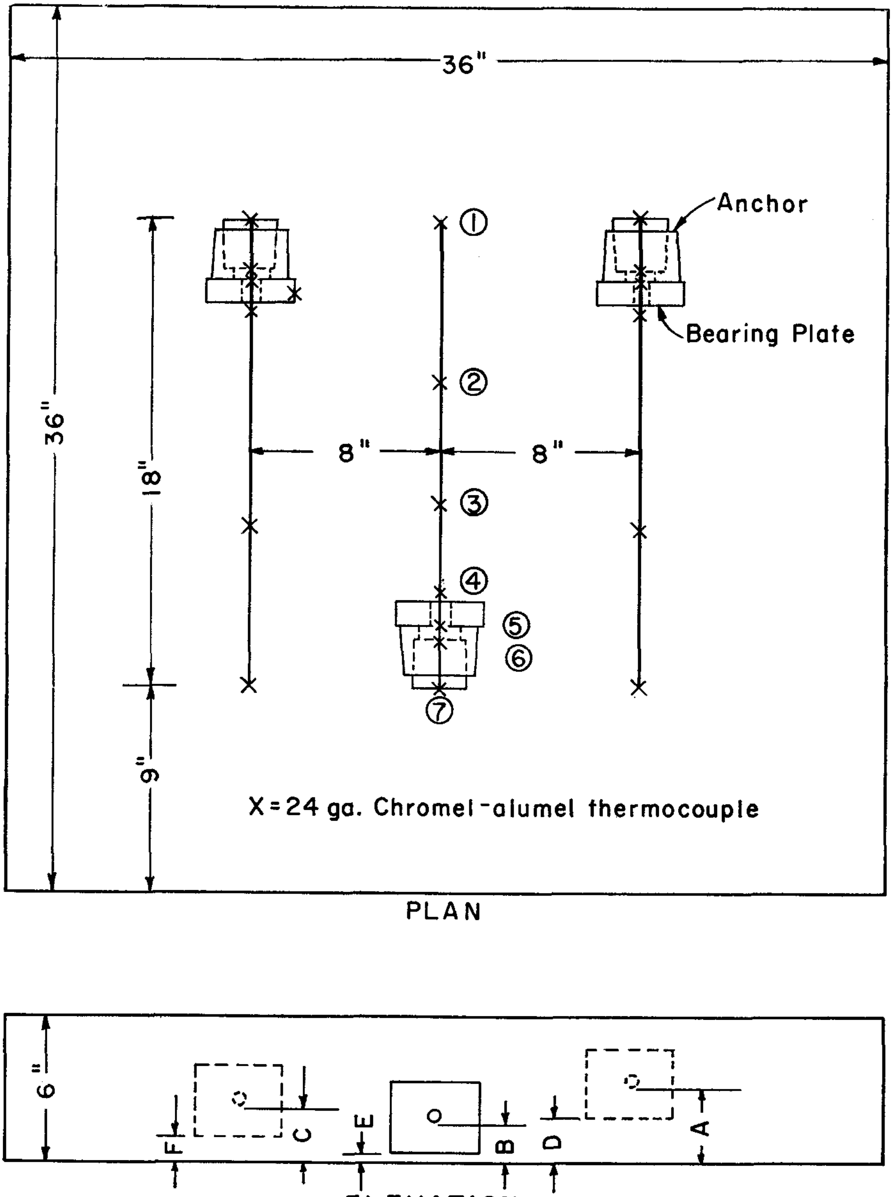


Fig. 12. Results of tests made at 800 F compared with value obtained for cold-drawn strand at same temperature

Laboratory. In two of the series, tests were performed at various temperatures between 600 F and 1000 F, and at 70 F. In the other series, 12 types of tendon-anchor assemblies were tested at 800 F and at 70 F. Results of these tests were compared with results of tensile tests of tendons in which the anchors were not heated.

Fig. 10 shows the test setup. Note that the bottom anchor assembly was centered within the electric furnace. The concrete cylinder was used to provide uniform bearing for the anchor. In fact, several of the types of bearing plates must be cast into the concrete. The cylinder also served to locate the anchor within the furnace. Some of the cylinders were jacketed with steel pipes. In such cases, the bearing plates were machined to a maximum diameter of 3⅞ in. to ensure that the plates did not bear on the steel jacket. A load of about 1000 lb was applied to the specimen at the start of the test and maintained during the heating period. A period of 2 to 3½ hr was required to heat the specimen to the desired test temperature. When thermocouples 1, 2, and 3 (Fig. 10), located on the anchor housing and on the tendon, reached the test temperature with a variation of 15 F or less, the tensile load was increased at a rate of about 8000 lb per min until failure occurred.

Results of the two series of tests conducted at 70 F, 700 F, 800 F, 900 F, and 1000 F, are shown in Fig. 11.



Dimensions in inches

Dimension	A	B	C	D	E	F	Strand Size
Specimen A	3	1 1/2	2 1/4	1 7/8	3/8	1 1/8	0.6 in.
Specimen B	1 1/2	2 1/2	3	3/4	1	1	1/2 in.

Fig. 13. Details of strand-anchor assemblies embedded in slab

Series No. 1 consisted of 0.6-in. diameter strand and a rather massive anchor-bearing plate assembly. Both the strand size and the anchor are among the largest in use in North America. They were selected for this series of tests because the investigators felt that large tendon anchor assemblies might be more vulnerable to heat than smaller ones. Series 12 consisted of ½-in. strand and small anchor-bearing plate assemblies. Duplicate tests were conducted at 70 F, 700 F, and 900 F and triplicate tests at 800 F. Relatively small differences in the breaking loads occurred for duplicate and triplicate tests at a specific temperature. Fig. 11 shows the results of these tests compared with the tensile strength-temperature relation of cold-drawn steel strand determined by Abrams and Cruz[6] from tests in which the anchors were not heated. It can be noted that the test results compare favorably with those for strand, differing by three percentage points or less in all cases. Thus it appears reasonable to assume that the temperature-strength relationships of tendon-anchor assemblies are about the same as those for the tendon alone.

In the third series of tests, 12 types of tendon-anchor assemblies were tested at temperatures of 800 F and 70 F. Eight assemblies made use of ½-in. diameter seven-wire strand, two used 0.6 in. strand, one used ¼-in. diameter button-headed wire, and one a ⅝-in. diameter deformed bar. Participants who supplied tendons and anchors were:

- Atlas Prestressing Corp.
- Dyckerhoff and Widmann, Inc.
- Freyssinet Company, Inc.
- Inland-Ryerson Construction Products Co.
- Prescon Corp.
- Stressteel Corp.
- Stresstek Corp.
- VSL Corp.
- Western Concrete Structures, Inc.

Fig. 12 shows the results of the 12 tests (at 800 F) compared with the comparable value reported for Abrams and Cruz[6] for strand at 800 F. The value shows for Series No. 9 is not directly comparable to the others because the tendon was a ⅝-in. diameter hot-rolled bar having a tensile strength of 215 ksi. The other tendons were 270 ksi cold-drawn strand or 240 ksi wire. Note that the average variation from that for strand was less than two percentage points and the maximum variation was about four percentage points.

As noted above, the results of Series No. 9 are not directly comparable to the others since the temperature-strength relation for 215 ksi hot-rolled steel is probably different than that of cold-drawn strand. From Fig. 1 it can be seen that high strength alloy steel bars (145 ksi) have about 80 percent of their 70 F strength at 800 F. Even though the value of 68 percent for Series No. 9 is lower than that for 145 ksi bars, it is considerably higher than the value for 270 ksi cold-drawn steel, and thus seems to be reasonable.

From these tests it appears that the anchor does not influence the temperature-strength relation significantly. It should be noted that the tendon-anchor assemblies represented a wide spectrum of those in use in North America. It does not appear that the mass of the anchor has a significant influence on the behavior at high temperatures.

Fire tests to study the effects of cover on tendons and anchors

In most post-tensioned structures, exposure to fire is likely to be less severe at the anchor than at other locations in the beam or slab. However, in some cases, the anchors are situated in vulnerable locations. Because the anchors represent concentrations of metal, it is likely that the strand temperature at the

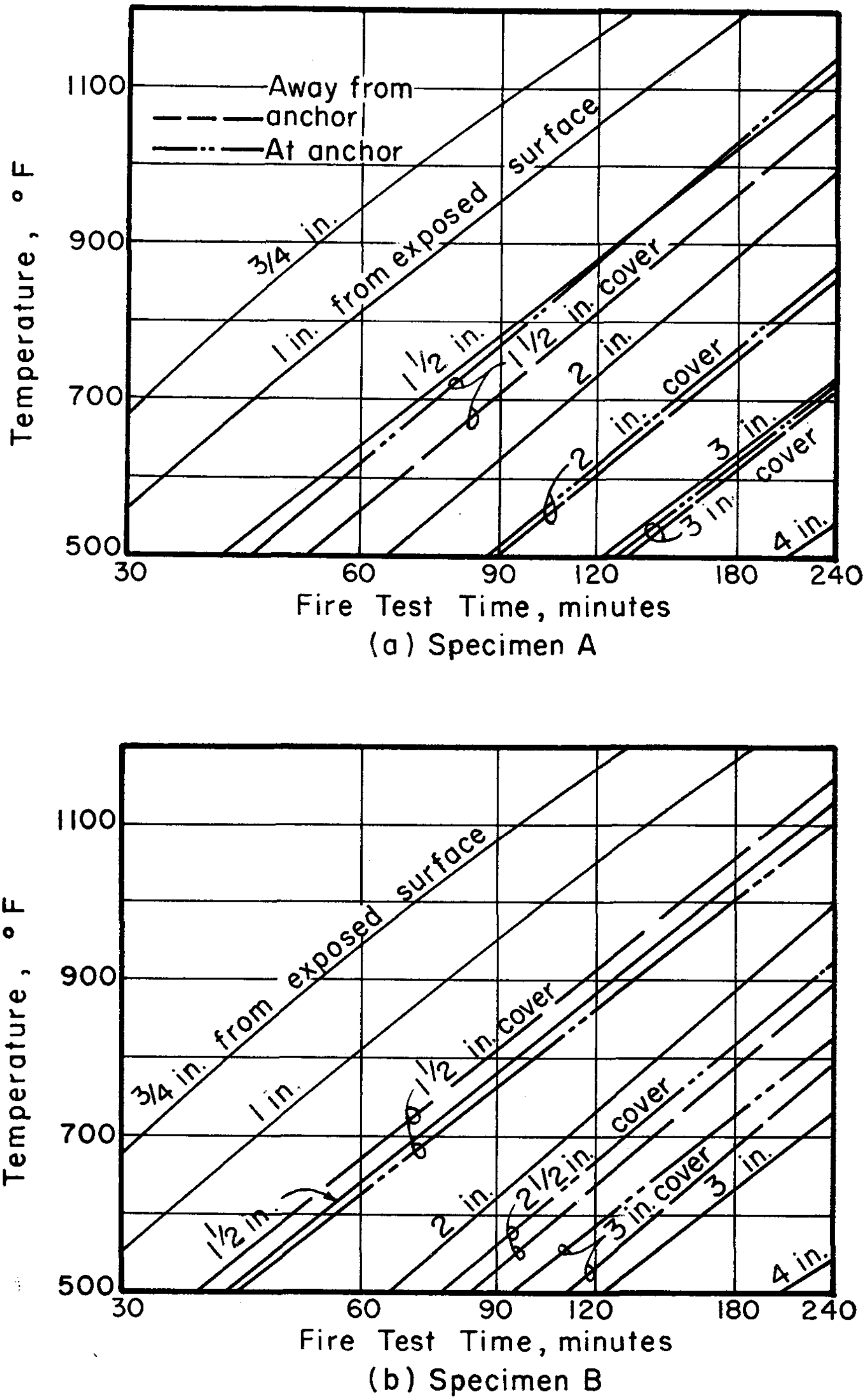


Fig. 14. Temperatures within concrete during fire tests compared with strand temperatures at and away from anchors

Table 6. Suggested Concrete Slab Thickness Requirements for Various Fire Endurances

Aggregate Type	Slab Thickness, in., for Fire Endurance Indicated				
	1 hr	1-1/2 hr	2 hr	3 hr	4 hr
Carbonate	3-1/4	4-1/8	4-5/8	5-3/4	6-5/8
Siliceous	3-1/2	4-1/4	5	6-1/4	7
Lightweight	2-5/8	3-1/4	3-3/4	4-5/8	5-1/4

anchor can be different from that away from the anchor.

To study the magnitude of the temperature difference, fire tests were performed on two slabs in which three tendon-anchor assemblies were embedded. Slab specimens were 3 x 3 ft in plan and 6 in. thick. Strands in the tendon-anchor assemblies were horizontal throughout as shown in Fig. 13.

Fig. 14 shows the results of the tests. Temperatures of the tendons at the anchors were higher than away from the anchor for three of the tendons. The difference was insignificant for two tendons, and for one tendon, the temperature of the tendon at the anchor was cooler than away from the anchor. Disregarding the tendon that was cooler at the anchor, the tendons were up to about 70 F warmer at the anchor than away from the anchor. To compensate for the higher temperature at the anchor, the cover to the tendon can be increased by about ¼ in.

Fire tests to study the effects of different sheathing materials for unbonding

In North American practice, unbonded post-tensioned tendons are generally greased and sheathed with either kraft paper or plastic. Paper sheathing is generally spirally wrapped while plastic sheathing is usually in the form of a continuous tube. A fire test was conducted at the Portland Cement Association Laboratory to determine if the sheathing material affects the tendon temperature during exposure to fire.

The fire test specimen consisted of a concrete slab in which some strands were sheathed with paper and some with plastic. The 4-in. thick slab specimen, which was 3 x 3 ft in plan, contained two layers of sheathed strand. Four strands in the east-west direction had 1-in. cover and four in the north-south direction had 2-in. cover. At each level, the first and third strands were sheathed with paper and the other two with plastic. The 4-in. thick slab specipositioned on each strand, one at midspan and the other 12 in. away. The thermocouples were located on the strand within the sheaths.

The slab specimen was exposed to a standard (ASTM E119) fire exposure for 2½ hr. During the test, thermocouple readings were monitored and compared. With 1-in. cover, strands with paper sheathing were about 15 F to 30 F cooler than those with plastic sheaths. With 2-in. cover, strands with paper sheaths were 10 F cooler to 15 F warmer than those with plastic sheaths. These differences are not considered to be significant because of the usual variations in temperature readings of embedded metal in concrete. Thus it appears that the type of sheathing material (paper or plastic) has only a minor influence on the strand temperature and does not affect the concrete cover requirements significantly.

RECOMMENDATIONS FOR MINIMUM DIMENSIONS FOR VARIOUS FIRE RESISTIVE CLASSIFICATIONS

Slabs

For heat transmission, i.e., temperature rise of 250 F of the unexposed surface, the thickness requirements for concrete slabs should be the same whether the concrete is plain, reinforced, or prestressed. Table 6 gives slab thicknesses suggested in *PCA Research Department Bulletin* 223.[15]

Cover thicknesses for post-tensioned tendons in unrestrained slabs are determined by the elapsed time during a fire test until the tendons reach a critical temperature. For cold-drawn prestressing steel that temperature is 800 F. For restrained slabs there are no temperature limitations. Fire tests of restrained slabs indicate that slabs with post-tensioned reinforcement behave about the same as reinforced concrete slabs of the same dimensions. Accordingly, the cover for post-tensioned tendons in slabs should be the same as the cover for reinforcing steel in slabs. Applying these criteria to slabs with post-tensioned tendons made of cold-drawn steel, cover thicknesses are suggested in Table 7.

Beams

Minimum dimensions for beams with post-tensioned reinforcement for various fire endurances are functions of the types of steel and concrete, beam width, and cover. For very wide beams, the cover requirements should be about the same as those for slabs.

For restrained beams spaced more than 4 ft on centers, the fire endurance is twice the elapsed time during a fire test at which the steel reaches the critical temperature. The suggested cover thicknesses in Table 8 are based on these criteria.

For beams or joists less than 8 in. wide, the Underwriters' Laboratories requirements for pretensioned stemmed members can be used for members with post-tensioned cold-drawn steel. Beams or joists with post-tensioned high strength alloy steel bars and narrower

Table 7. Suggested Concrete Cover Thicknesses for Slabs Prestressed with Post-Tensioned Reinforcement

Restrained or Unrestrained	Aggregate Type	Cover Thickness, in., for Fire Endurance of				
		1 hr	1-1/2 hr	2 hr	3 hr	4 hr
Unrestrained	Carbonate	3/4	1-1/16	1-3/8	1-7/8	– – –
Unrestrained	Siliceous	3/4	1-1/4	1-1/2	2-1/8	– – –
Unrestrained	Lightweight	3/4	1	1-1/4	1-5/8	– – –
Restrained	Carbonate	3/4	3/4	3/4	1	1-1/4
Restrained	Siliceous	3/4	3/4	3/4	1	1-1/4
Restrained	Lightweight	3/4	3/4	3/4	3/4	1

Table 8. Suggested Cover Thickness for Beams Prestressed with Post-Tensioned Reinforcement

Restrained or Unrestrained	Steel Type	Concrete Type*	Beam Width,** in.	Cover Thickness, in., for Fire Endurance of				
				1hr	1-1/2	2 hr	3 hr	4 hr
Unrestrained	Cold-drawn	NW	8	1-3/4	2	2-1/2	4-1/2***	---
Unrestrained	Cold-drawn	LW	8	1-1/2	1-3/4	2	3-1/4	---
Unrestrained	H.S.A. Bars	NW	8	1-1/2	1-1/2	1-1/2	2-1/2	---
Unrestrained	H.S.A. Bars	LW	8	1-1/2	1-1/2	1-1/2	2-1/4	---
Restrained	Cold-drawn	NW	8	1-1/2	1-1/2	1-3/4	2	2-1/2
Restrained	Cold-drawn	LW	8	1-1/2	1-1/2	1-1/2	1-3/4	2
Restrained	H.S.A. Bars	NW	8	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2
Restrained	H.S.A. Bars	LW	8	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2
Unrestrained	Cold-drawn	NW	>12	1-1/2	1-/34	2	2-1/2	3
Unrestrained	Cold-drawn	LW	>12	1-1/2	1-1/2	1-3/4	2	2-1/2
Unrestrained	H.S.A. Bars	NW	>12	1-1/2	1-1/2	1-1/2	1-1/2	2
Unrestrained	H.S.A. Bars	LW	>12	1-1/2	1-1/2	1-1/2	1-1/2	2
Restrained	Cold-drawn	NW	>12	1-1/2	1-1/2	1-1/2	1-3/4	2
Restrained	Cold-drawn	LW	>12	1-1/2	1-1/2	1-1/2	1-1/2	1-3/4
Restrained	H.S.A. Bars	NW	>12	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2
Restrained	H.S.A. Bars	LW	>12	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2

* NW = normal weight; LW = lightweight
** For beams with widths between 8 and 12 in., cover thickness can be determined by interpolation
***Not practical for 8-in. wide beam but shown for purposes of interpolation

than 8 in. should have the same cover as joists of the same size and fire endurance.

Anchor protection

The cover to the prestressing steel at the anchor should be at least ¼ in. greater than that required away from the anchor. Minimum cover to the steel bearing plate should be at least 1 in. in beams and ¾ in. in slabs.

REFERENCES

1. ASTM Designation: E119-71, "Standard Methods of Fire Tests of Building Construction and Materials," Part 14, *ASTM Book of Standards,* American Society for Testing and Materials.
2. Gustaferro, A. H., and Selvaggio, S. L., "Fire Endurance of Simply Supported Prestressed Concrete Slabs," *PCI JOURNAL,* Vol. 12, No. 1, February, 1967, pp. 37-52. *PCA Research Department Bulletin* 212.
3. Gustaferro, A. H., "Temperature Criteria at Failure," *Fire Test Performance,* ASTM STP 464, American Society for Testing and Materials, 1970, pp. 68-84.
4. Selvaggio, S. L., and Carlson, C. C., "Effect of Restraint on Fire Resistance of Prestressed Concrete," *Fire Test Methods, ASTM STP* No. *344,* American Society for Testing and Materials, 1962. *PCA Research Department Bulletin* 164.
5. Brockenbrough, R. L., and Johnston, B. G., *Steel Design Manual,* U.S. Steel Corp., Pittsburgh, Pennsylvania, 1968, 246 pp.
6. Abrams, M. S., and Cruz, C. R., "The Behavior at High Temperature of Steel Strand for Prestressed Concrete," *Journal of the PCA Research and Development Laboratories,* Vol. 3, No. 3, September, 1961, pp. 8-19; *PCA Research Department Bulletin* 134.
7. Gustaferro, A. H., *et al.,* "Fire Resistance of Prestressed Concrete Beams. Study C: Structural Behav-

ior During Fire Tests," *PCA Research and Development Bulletin (RD 009B)*, Portland Cement Association, 1971.

8. Abrams, M. S., "Compressive Strength of Concrete at Temperatures to 1600 F," *Symposium on Effect of Temperature on Concrete*, American Concrete Institute Publication SP-25, Detroit, Michigan, 1971.
9. Troxell, G. E., "Fire Test of Prestressed Concrete Floor Panel No. 1, *"Fire Prevention Research Institute*, SFT-1, Gardena, California.
10. Troxell, G. E., "Fire Test of 6-in. Hardrock Concrete, Post-Tensioned, Prestressed Flat Slab", *Fire Prevention Research Institute*, SFT-2, Gardena, California.
11. "Report on Unbonded Post-Tensioned Prestressed, Reinforced Concrete Flat Plate Floor with Expanded Shale Aggregate," *Underwriters' Laboratories, Inc.*, R5084-3. Reprinted in *PCI JOURNAL*, Vol. 13, No. 2, April 1968, pp. 45-56.
12. "Report on Prestressed Pretensioned Concrete Inverted Tee Beams and Report on Prestressed Concrete Inverted Tee Beams Post-Tensioned," *Underwriters' Laboratories, Inc.*, R4123-12-12A. *PCI Publication R-119-66.*
13. Ashton, L. A., and Bate, S. C. C., "The Fire Resistance of Prestressed Concrete Beams," *ACI Journal*, Vol. 32, May 1961, pp. 1417-1440.
14. "Fire Resistance Index,'" Underwriters' Laboratories, Inc., Northbrook, Illinois, January, 1972.
15. Abrams, M. S., and Gustaferro, A. H., "Fire Endurance of Concrete Slabs as Influenced by Thickness, Aggregate Type, and Moisture," *Journal of the PCA Research and Development Laboratories*, V. 10, No. 2, May 1968, pp. 9-24. *PCA Research Department Bulletin 223.*
16. Abrams, M. S., and Gustaferro, A. H., "Fire Endurance of Prestressed Concrete Units Coated with Spray-Applied Insulation," *PCI JOURNAL*, Vol. 17, No. 1, January-February, 1972, pp.82-103.

Discussion of this paper is invited. Please forward your discussion to PCI Headquarters by August 1, 1973, to permit publication in the September-October 1973 issue of the PCI JOURNAL.