FIRE RESISTANCE

OF

PRESTRESSED
CONCRETE

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SUMMARY

In summary some general comments can be made about prestressed concrete in fire. The main problem is the amount of cover required to protect the steel from reaching excessive temperatures. This is also the main problem for reinforced concrete. The other problem is restraint. It could not really be said that prestressed concrete behaves better than reinforced concrete. The prestressing strand however is more susceptible to temperature rise due to its properties. It is surprising that no where in the references used was there anything to indicate the importance of temperature effects on bond. However mention was made about the research required to enable better understanding of this. In general the research done has proved that fire resistance of prestress concrete is excellent.

INTRODUCTION

Fire resistance of prestressed concrete is a subject few people understand. The fact that prestressed concrete contains high strength active reinforcement would appear to confuse the subject even more. Due to this, prestressed concrete is commonly and wrongly treated on construction sites as reinforced concrete. This relates especially to methods of restraining and attaching members to buildings.

Information within this report explains how steel and concrete behave in fire and the critical temperatures at which their properties are altered. The methods of testing and its restrictions are outlined in brief form. The structural behaviour as a result of the changes in concrete and steel properties and type of restraint are then explained, and the mechanisms by which they work. Finally methods of post-fire evaluation are outlined and how structural assessment is made. This gives the reader a good overall understanding of the way prestressed concrete behaves in fire and should enable comparison to be made with other structural materials.

SECTION 1.   HISTORICAL BACKGROUND

In the 1960's fire tests of floor assemblies were simple. The floor system
was placed over a pit (16 square metres with no side less than 3.7 metres for
an ASTM E119 test) and concreted in place. Super imposed loads were applied
and the fire test was conducted. The fire burned at a controlled rate
following the predetermined Time - Temperature graph (Figure A1 in Appendix
A) until either:

   a). The floor collapsed
   b). The fire burned a hole through
   c). The top surface got hot enough to ignite some highly
       combustible material laying on it

Prestressed concrete suffered no irreparable damage when the fire resistance
rating was reached, small deflections, cracking and spalling were the main
problems which are very small in comparison to structural failure (ref. 11).

In the late 1960's however, fire tests of composite floors with steel joists
supporting concrete slabs on metal trays showed up an interesting structural
phemonmenon. Part way through the fire test the steel support joists were
observed to melt and sag away from the slab. The lightly reinforced slab,
which was restrained by thermal expansion by the sides of the test assembly,
was prestressing itself and was able to support the full live load. The floor
assembly had clearly failed but had met the requirements of the standard fire
test. From this it was established that the fire test had to be changed. The
arching, or prestressing effect of the thermal restraint had also affected
other tests, namely those of prestressed concrete (ref. 11).

In the early 1970's the concept of restrained and unrestrained ratings for
fire tests was introduced. Restraint in this case means thermal restraint.
However a designer incorporates restraint for structural reasons and this
would in effect act as thermal restraint and enhance the fire resistance of
the member (ref. 11).

Research over the past decade has resulted in major advances in fire
engineering of concrete building structures. The advances which have occurred
allow a more realistic assessment of the structural components. Existing
practices used may be conservative and not take into account benefits of
continuity and restraint. These factors are currently not adequately
assessed, but research is improving the understanding of them.

SECTION 2.   MATERIALS PROPERTIES AT HIGH TEMPERATURES

2.1 PRESTRESSING STEEL

The most commonly used prestressing steel in New Zealand is hard drawn, stress relieved strands of high carbon content. This type of steel in fire conditions undergoes a decrease in tensile strength similar to the decrease of yield strength in mild steels.

A reduction in the modulus of elasticity is also apparent as we look at the standard stress-strain relationship for prestressing steel at various temperatures. (Figure 1)
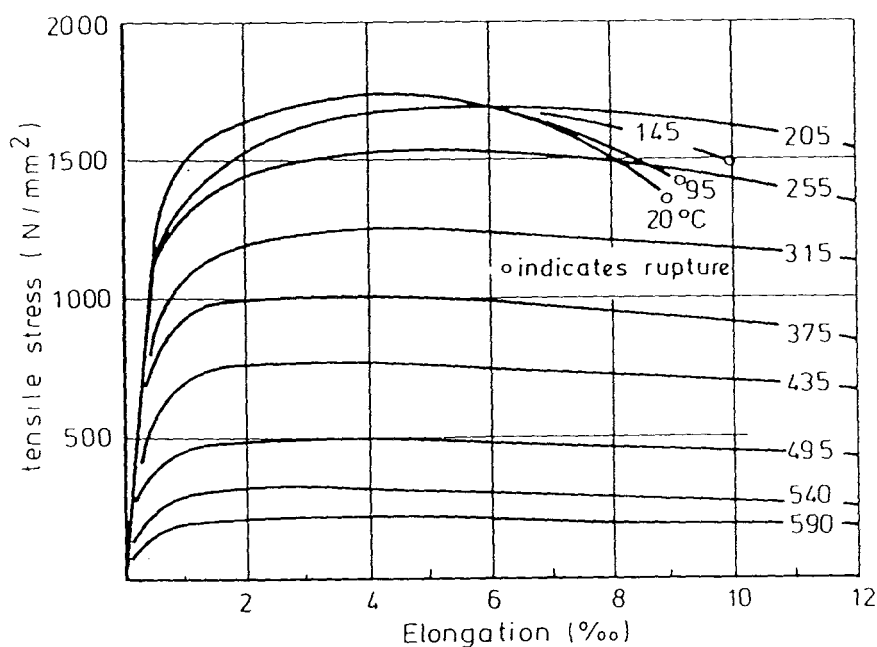

Figure 1   Stress - strain relationship of cold drawn, stress relieved prestressing wire under elevated temperatures (ref #2).

For rational design, it is important to know the "critical" temperature for the steel within the structure. The critical temperature is that temperature at which the tensile strength of the steel is reduced to the "actual" strength required in the steel. This is the point of collapse due to failure of the steel. For prestressing steel, depending on tensile strength, an increase in temperature to 400°C will result in a decrease in strength of approximately 50%. The strength is recoverable up to 300°C and partially up to 400°C (ref. 10). A comparison of losses in strength between structural steel and reinforcing steels can be seen in Figure 2. Here we see the cold drawn steels closely model reinforcing steels but are more susceptible to temperature.

PERCENT OF STRENGTH AT 21°C

:00

80

60

40

20

0

HIGH STRENGTH
ALLOY STEEL BARS
(TENSILE STRENGTH)

HOT-ROLLED STEEL
(YIELD STRENGTH)

COLD DRAWN
PRESTRESSING STEEL
1724 OR 1862 MPa

100   200   300   400   500   600   700°C

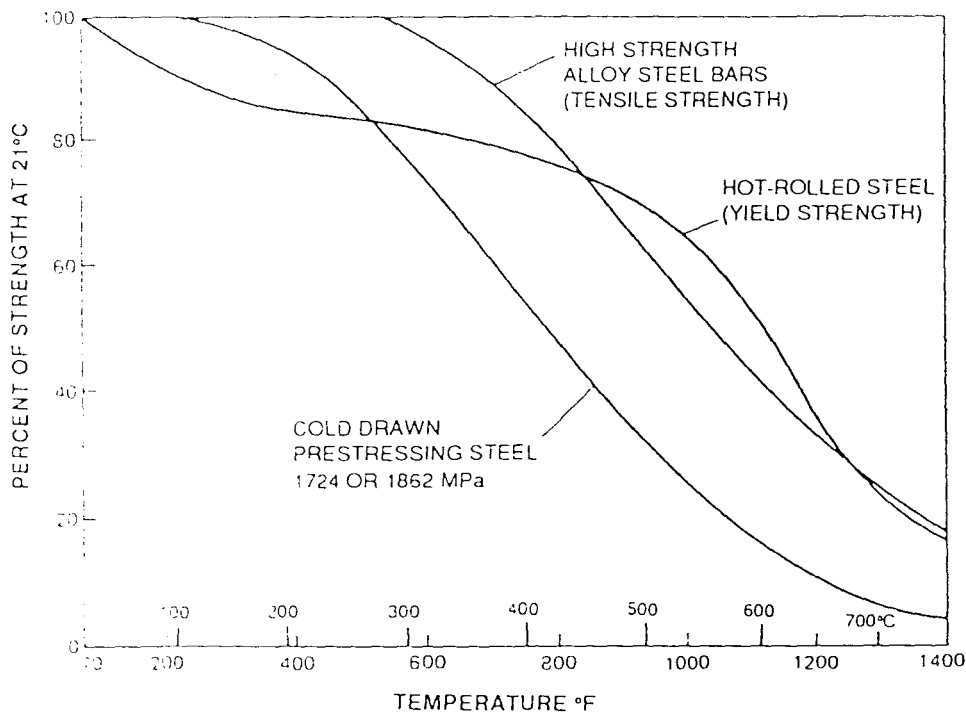0   200   400   600   800   1000   1200   1400

TEMPERATURE °F

Figure 2   Temperature/Strength relationship for hot-rolled and cold drawn steel (ref #2)

Along with tensile strength, prestress tension can be lost within the steel when the modulus of elasticity decreases, (approximately 6% at 200°C to 20% at 300°C). Other losses of tension with increases of temperature arise from relaxation of steel due to creep. Because longitudinal expansion of hard drawn wire is not proportional to temperature, it is not recoverable on cooling from temperatures above 150°C (ref. 10).

When the steel in a loaded prestressed beam is heated to between 200°C and 300°C appreciable deformation occurs and, on cooling, the loss of prestress may be considerable, although the recovery of strength may be complete up to 300°C and considerable up to 400°C. Therefore, the behaviour of a prestressed member which remains intact after fire, is less likely to be severely affected by a fall in tensile strength of the steel than loss of prestress tension.

2.2 CONCRETE

Exposure of concrete to high temperatures can result in significant loss of compressive strength. There is also a drop in the modulus of elasticity at

210°C, this drops to 70% of the value at room temperature, while at 420°C, it drops to 50% and at 650°C only 30%. This causes a reduction in the stiffness of the concrete structural member (ref. 1). The degradation of the concrete properties is influenced mainly by the type of aggregate, due primarily to different rates of heat transmission. Carbonate aggregates which include limestone, limerock and dolomite all consist of calcium and/or magnesium carbonate. These aggregates undergo a chemical change at 680°C during which carbon dioxide is released. This reaction consumes heat and the residual materials tend to retard the flow of heat. Siliceous aggregates are those consisting principally of silicon dioxide. These aggregates do not undergo chemical changes at temperatures encountered in fire tests (Figure 3) (ref. 5).
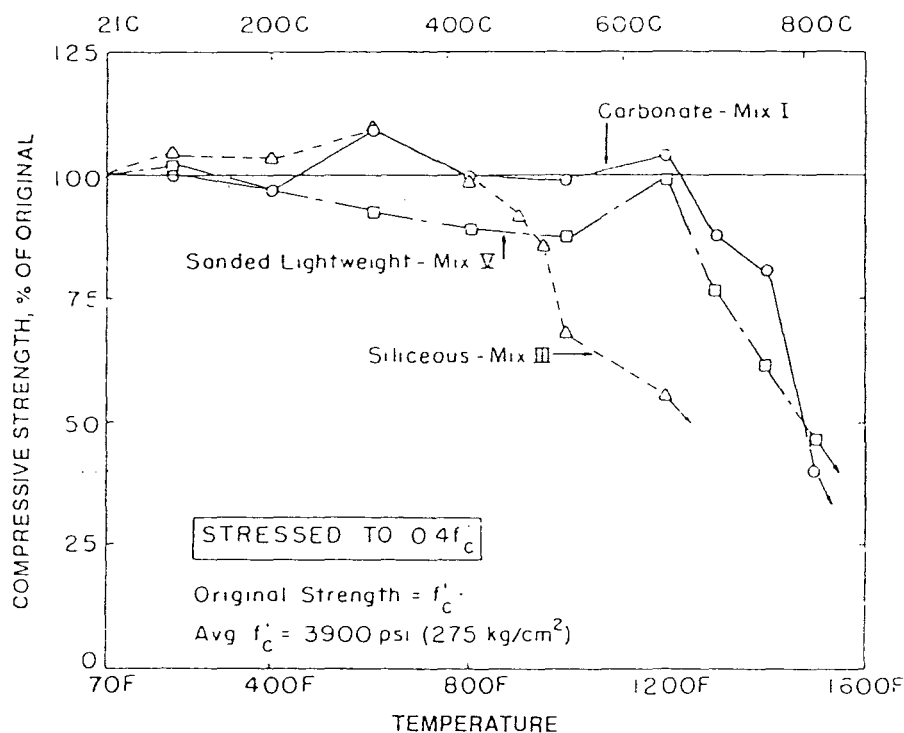

Figure 3 Compressive Strength of Concrete at High Temperatures (Ref #2)

At high temperatures concrete can produce further hydration of the cement resulting in an initial increase in strength. However there is a loosening of interparticle bond which occurs as a result of different coefficients of

expansion. Rapid temperature rise can result in vaporisation of entrapped water. This causes spalling, which is sometimes explosive, but again this is dependant on the types of aggregate. Carbonate and light weight aggregate concretes do not spall while igneous aggregate concretes do. Excessive temperatures can result in the destruction of a concrete structure from severe spalling (ref. 5).

Concrete expands with rising temperatures, but higher temperatures also cause further shrinkage of the hardened concrete paste. These two movements act in opposition forming micro cracks on cooling, complete recovery of deformation is not always observed. This is complicated further when longitudinal expansion is restrained as is often the case in prestressed concrete (ref. 5).

The range between 250°C and 300°C is generally quoted as the cut off figure, thereafter significant strength loss will occur. The processes of heat penetration into a concrete member are therefore extremely important, regarding the depth to which the concrete becomes damaged. The factors influencing the penetration of heat into concrete relate to its properties at the onset of the fire, combined with changes in its physical and chemical composition. The coefficient of thermal conductivity of concrete depends on the conductivity of its constituents, namely the cement paste and the aggregate. Mix proportions and the degree of compaction will therefore influence conductivity to some extent. Concrete conductivity in general is known to decrease with increased temperature through loss of pore water and the dehydration of cement paste. A concrete surface exposed to a high enough temperature will undergo these changes and effectively produce an insulating layer of lower thermal conductivity that acts as a refractory material and reduces the ingress of heat. This is a very important factor which helps to make it clear that within the context of practical fire resistance it can be said that concrete is an excellent material (ref. 17).

SECTION 3.   DESIGN FIRE

For a fire rating to be allocated to a prestressed member it will be exposed
to the standard fire. The time at which failure in either form, capacity,
integrity or insulation occurs will indicate acceptable times of exposure.

The standard fire test is a time-temperature relationship. It is not
necessarily representative of a real or natural fire but it is generally
regarded as being a "severe fire".

3.1 FIRE RATING METHODS

Exposure of a prestressed member to the standard fire is a useful exercise as
it tests the actual construction situation. Its main disadvantages are
furnace size, cost of testing and large numbers of test variations, but it is
the major test method used. Analytical modelling is another method. This
method deals with cooling, restraint and continuity conditions more
realistically. However such methods are not wide spread (ref. 2).

3.2 TEMPERATURE DISTRIBUTIONS

The temperature distribution within concrete can be modelled using the Fourier
equation for non-steady state conditions. This equation however only gives
an approximation for temperature distribution since in addition to the heat
transfer of concrete there is moisture and vapour movements. Methods of
solving the equation include step by step or finite element methods. When
solving, allowance must be made for the changes in thermal conductivity and
specific thermal capacity of the concrete. Measurements in fire tests also
provide valuable correlation (ref. 10).

3.3 SHAPE OF SECTION

Temperature distribution, along with material properties is a function of the
size and shape of a section. For example the temperature rise within a beam
section is more rapid than that in a slab. This is due to the concrete being
heated from all exposed faces.

Many simple temperature rise curves exist for slab temperatures and the major
variation is the type of aggregate involved (Figure B1, B2, B3 in appendix B).
However with beams, temperature distribution is affected by aggregate but more
so by the width and depth of section. The use of isotherm diagrams makes
determination of internal temperatures for various beams a much simpler
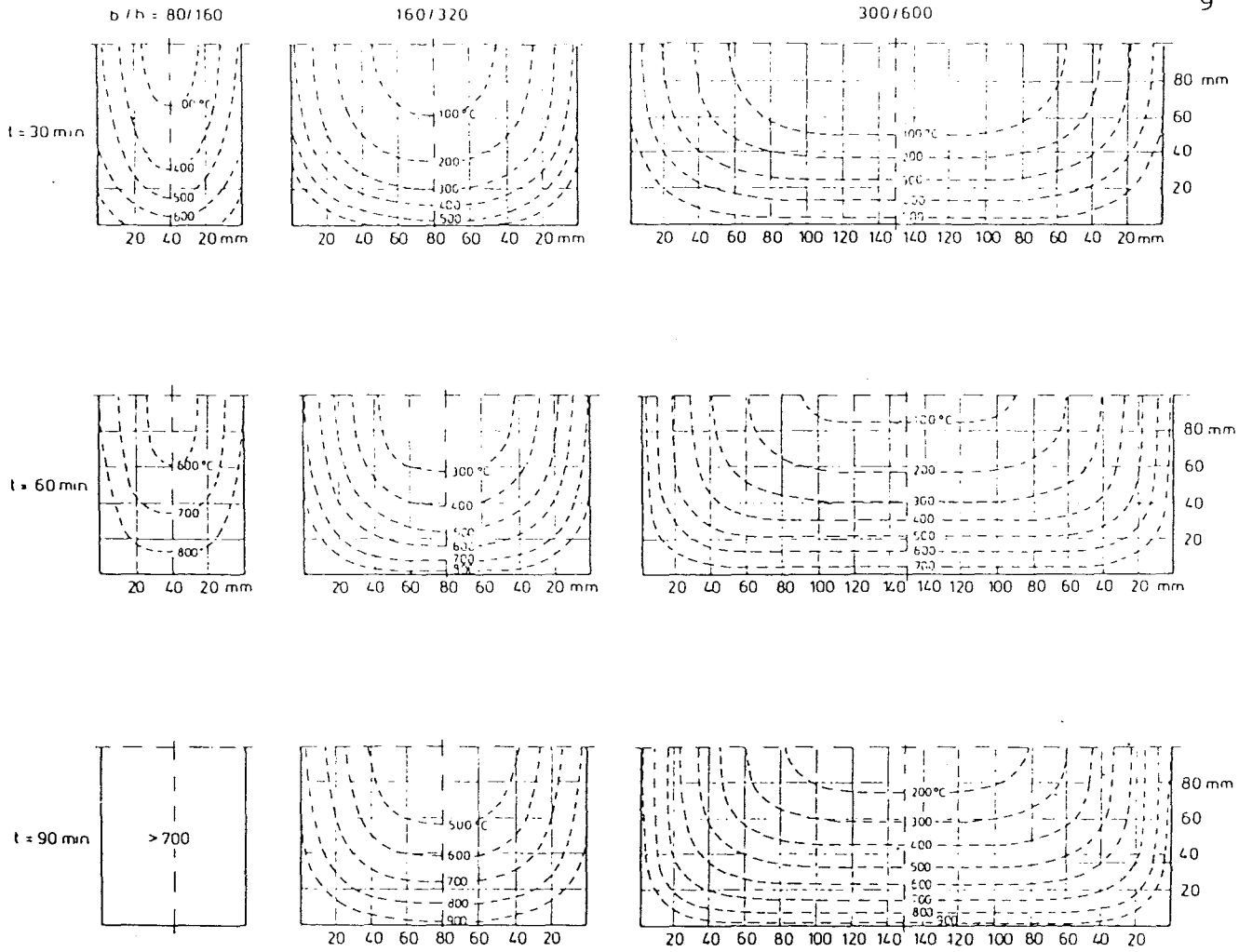process (Figure 4) (ref. 1).

b / h = 80/160   160/320   300/600

t = 30 min

0°C
400
500
600

100°C
200
300
400
500

100°C
200
300
400

80 mm
60
40
20

20 40 20 mm   20 40 60 80 60 40 20 mm   20 40 60 80 100 120 140 140 120 100 80 60 40 20 mm

t = 60 min

600°C
700
800

300°C
400
500
600
700
900

200°C
200
300
400
500
600
700

80 mm
60
40
20

20 40 20 mm   20 40 60 80 60 40 20 mm   20 40 60 80 100 120 140 140 120 100 80 60 40 20 mm

t = 90 min

>700

500°C
600
700
800
900

200°C
300
400
500
600
700
800
900

80 mm
60
40
20

20 40 60 80 60 40 20   20 40 60 80 100 120 140 140 120 100 80 60 40 20

Figure 4   Typical Isotherm diagram (Ref #1)

SECTION 4. STRUCTURAL BEHAVIOUR

4.1 SPALLING

The higher quality concrete used in prestressed concrete tends to be less permeable than ordinary concrete. Therefore, unless it is thoroughly dry in fire conditions, spalling of the concrete by steam trapped within the mass may occur and expose the steel tendons to the fire. When an exposed slab face is heated rapidly while the unexposed face is still cool, the thermal stresses become higher than the compressive stresses of the high quality concrete. This may cause spalling of thin slabs, and holes have been produced due to this condition. It has also been found that beams under partial loads, when bottom fibres are in high compression, can also cause spalling when exposed to high temperatures (Figure 5) (ref. 8).
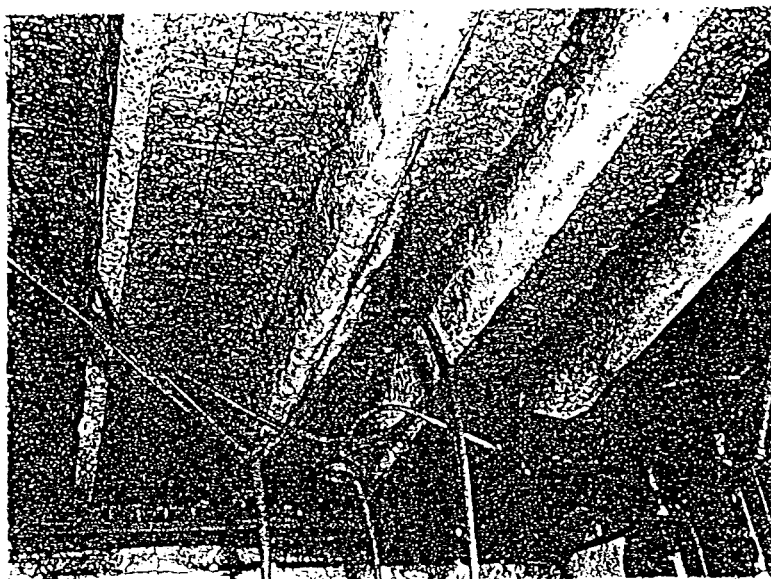

Figure 5   Spalling of roof units after fire (Ref #7)

4.2 UNRESTRAINED BEHAVIOUR

Consider a simply supported prestressed concrete member exposed to fire from below, with ends free to rotate and tendons close to the soffit. As the underside heats up expansion occurs at a greater rate than the topside and the member deflects downwards. Numerous cracks propagate from bottom fibres due to this behaviour (Figure 6). As this occurs the tensile strength of the steel is diminishing and when the steels critical temperature is reached flexural collapse will occur (ref. 1).
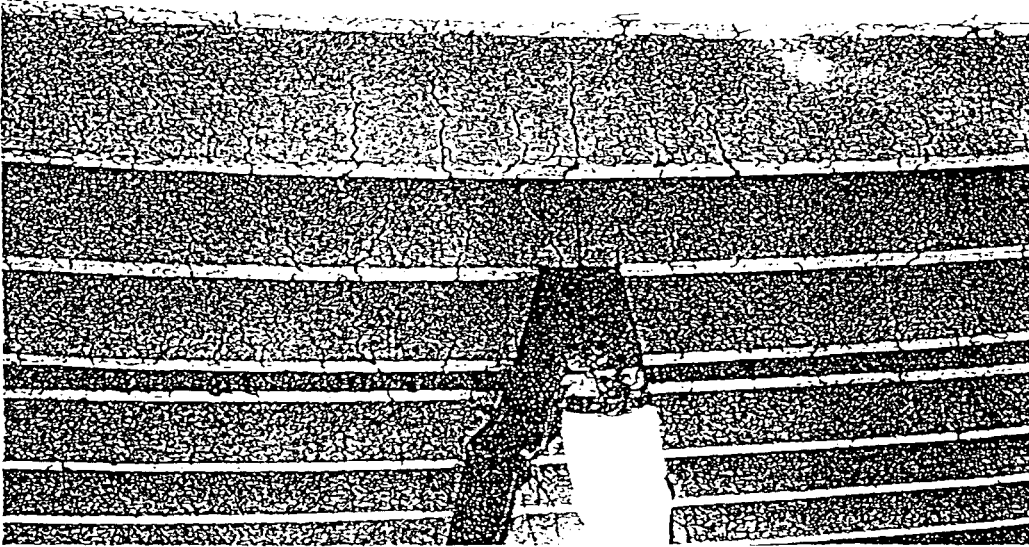
Figure 6 Cracking of roof units after fire (Ref #7)

4.3 RESTRAINED BEHAVIOUR

Figure 6 Member expansion

There are two mechanisms that supply restraints, they are restraint due to moment continuity and restraint due to resistance of the surrounding cooler structure.

i) For the case of members having continuity:

During the fire the positive capacity is reduced due to heating but the negative moment reduces at a slower rate as it tends to remain cooler. Because of this fact there is a redistribution of moments, allowing greater capacity and therefore out performing a simply supported member.

ii) The other method involves the restraint of horizontal expansion:

The surrounding cooler structure resists a force known as thermal thrust. Provided that the line of action of the thrust falls below the resultant of the compressive stress block of the element, the thrust will increase the moment capacity and fire resistance of the member.

Restraint can easily be provided in the interior bays of a multi-bay structure as resistance is provided by diaphragm action in outer bays. However in the exterior bays of a structure the problem of providing thermal restraint is more difficult as thrust forces are enormous. These forces therefore have to be carried by spandrel beams and columns. To over come this problem reduction of floor spans in exterior bays as to allow easy design for unrestrained ratings is a good option (ref. 1).

SECTION 5.   POST FIRE EVALUATION

In order for reinstatement to be effective and economic, it is important that the assessment of damage to the structure is quickly and reliably carried out. This can be done by visual inspection or examination of the structure by other simple methods.

5.1 VISUAL INSPECTION

Visual Inspection will enable evaluation of spalling or cracking. If cracking is extensive and if spalling is severe enough to have resulted in obvious damage of the prestressing tendons, the affected unit will probably have to be removed and replaced. If the spalling or cracking is minor and there is no other evidence of significant structural damage, repairs are generally limited to restoring the unit to its original dimensions and appearance.

Another important visual inspection is to determine change in camber or deflection. This can be done by comparison with undamaged units or by measuring and comparing with calculated values. Generally a severely fire damaged prestressed member will have considerable deflection because of the strength reduction of the prestressing steel and the reduction of the modulus of elasticity of the concrete (ref. 16).

5.2 OTHER METHODS OF ASSESSMENT

As prestressed concrete is effectively a composite member, the relationship of the temperature between steel and adjacent concrete is important. To find this information an examination of available fire debris should be carried out. This includes sounding and observation of colour change (ref. 16).

5.3 CONCRETE SOUNDING

It is common to use a Shcmidt hammer to deliver a controlled blow to the concrete surface. The rebound distance is measured and expressed as a number. The higher the number the better the residual concrete condition. However the rebound values can be seriously affected by the surface condition and the presence of hard aggregate particles near the surface (ref. 16).

5.4 COLOUR CHANGES

Concretes produced with certain aggregates show distinct colour changes when heated. These changes normally occur between the temperatures of 300°C and 600°C. The colour changes are mostly evident in siliceous river gravels, sandstones and the like. The most common alteration is the development of a pink colour due to a change in the hydration states of iron oxides and other salts within the concrete aggregates.

Not all aggregates exhibit colour changes. For example this method may not

be applicable to concretes produced with igneous aggregates, such as Dolerite, Basalt or Granite. If no colour changes are observed then either the concrete is undamaged or the necessary iron salts are not present (ref. 16).

5.5 TEST METHODS

If further assessment is required as may be the case in the event of complete failure the following tests could be used:

i).   Ultrasonic testing

ii). Core testing

iii). Thermoluminescence

These tests provide detailed information about the changes in the micro-structure of the concrete due to high temperatures (Figure 8). However this depth of detail will generally not be required for evaluation (ref. 16).
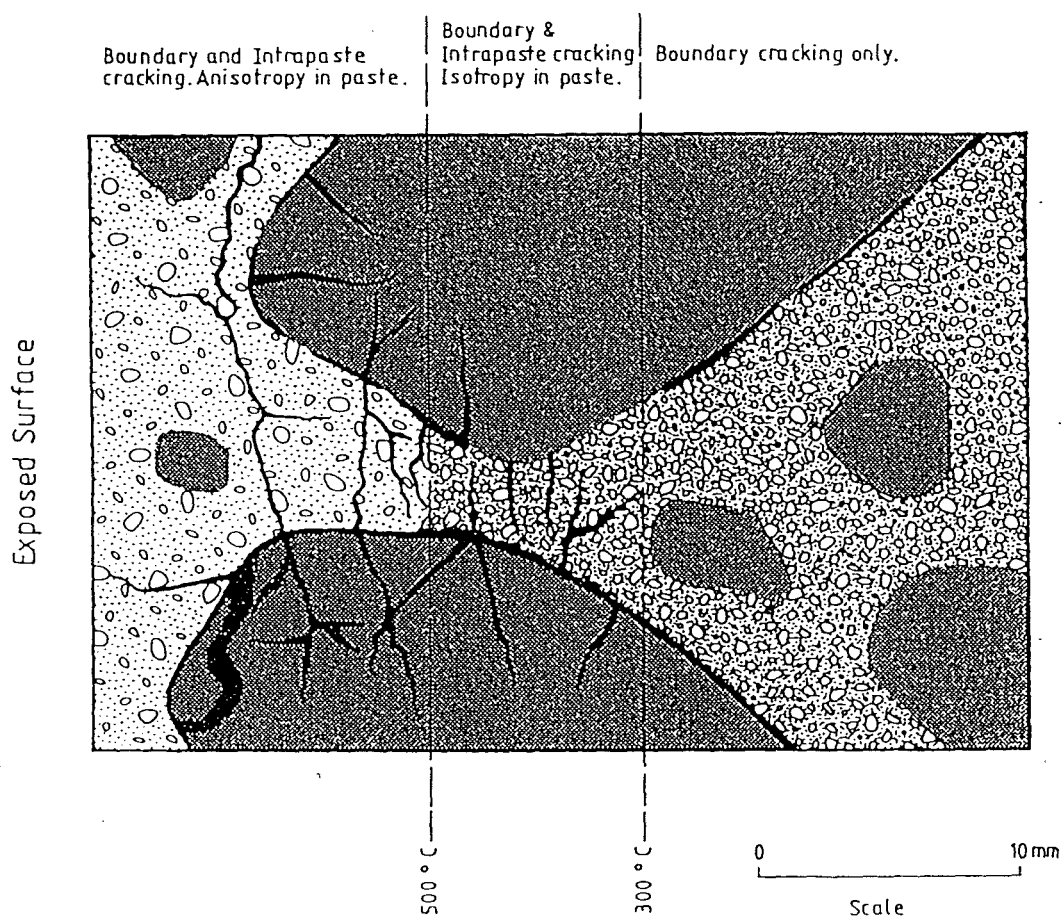

Figure 8 Typical detailed section of fire damage concrete (Ref #16)

5.6 LOAD TESTING

Load tests of units are sometimes warranted to aid the assessment of fire damaged units. This is clearly a simple way of determining the effects of fire on the prestressed components in a building. Although this procedure can be time consuming and expensive, it is a very reliable way of determining the post fire behaviour of the unit under load.

CONCLUSIONS

It can be concluded that prestress concrete structures can be designed to resist fires lasting up to four hours. The principal factor affecting its resistance is the thickness of the protective cover on the steel tendons. However, a secondary condition is the application of axial restraint which increases fire resistance. Continuity over points of support will also increase fire resistance in the same manner as for conventional reinforced concrete. Also of importance is the size of the element; the larger it is, the greater its heat capacity and fire resistance.

The required cover for slabs is less than for beams because the heat of the fire penetrates slabs from the bottom only, whereas in beams it penetrates from the sides as well as from the bottom.

For unrestrained beams, failure of the steel will occur when it reaches approximately 400°C. However, for axially restrained beams, a beam might safely carry its design load even at appreciably higher temperatures, but the steel will then have lost most of its strength. In this case, the prestress in the concrete is maintained by the restraint at the supports. Although there are some unique characteristics to be considered in prestressed concrete design, generally it can be considered a very efficient structural member in the context of fire resistance.

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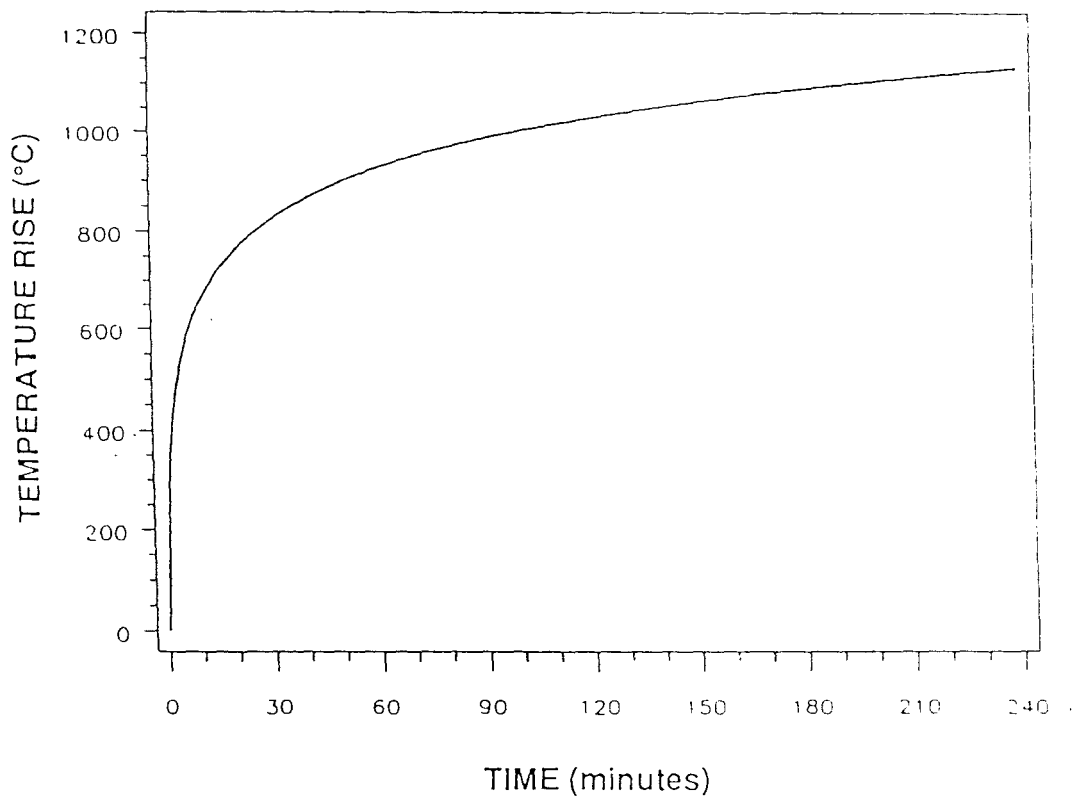*Very Good List*

APPENDIX "A"

Figure A1 ISO 834 Standard time - temperature curve
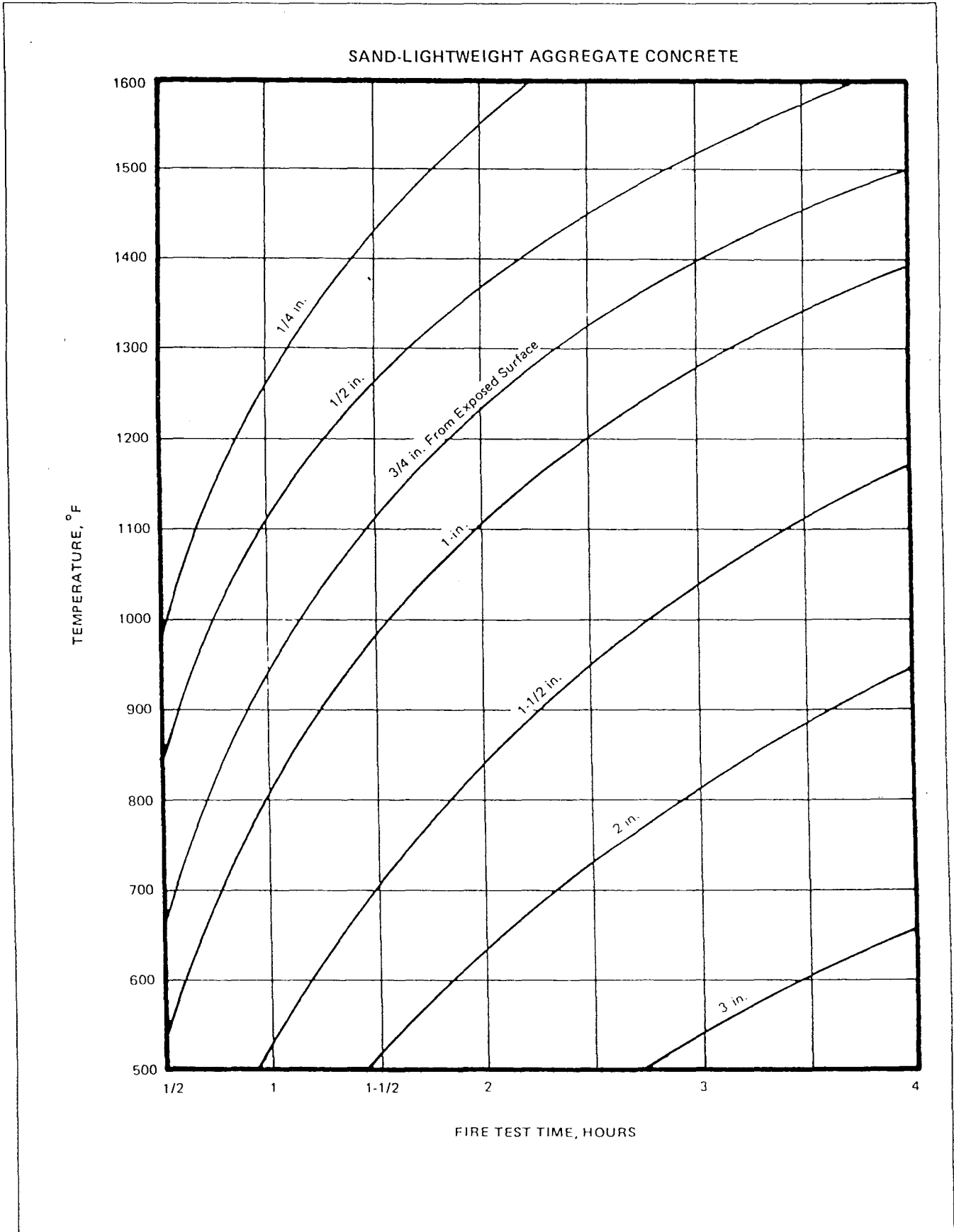
APPENDIX B"

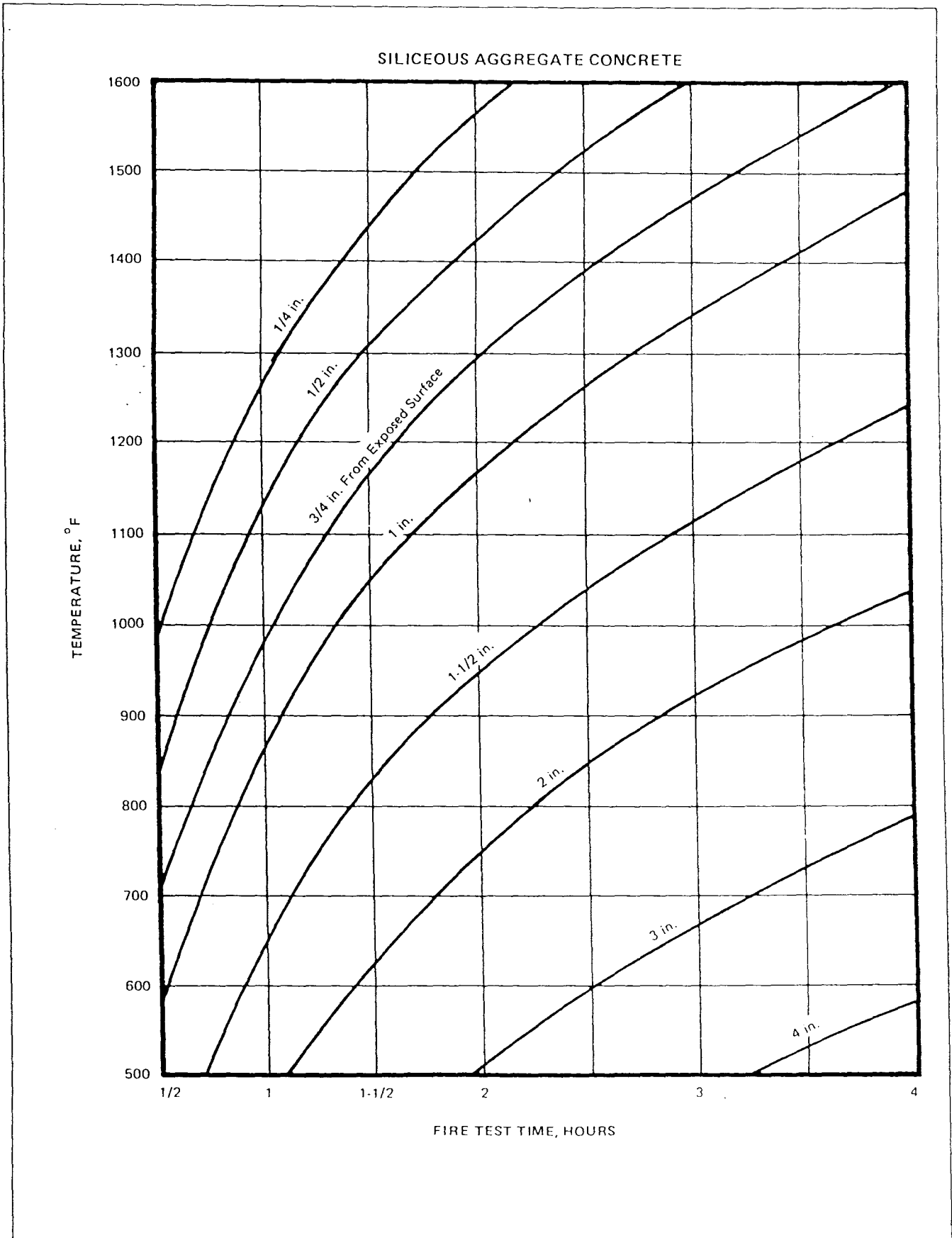Fig. B1   Temperatures within solid or hollow-core concrete slabs during fire tests — SAND-LIGHTWEIGHT AGGREGATE.

Fig. B 2   Temperatures within solid or hollow-core concrete slabs during fire tests — SILICEOUS AGGREGATE.

CARBONATE AGGREGATE CONCRETE
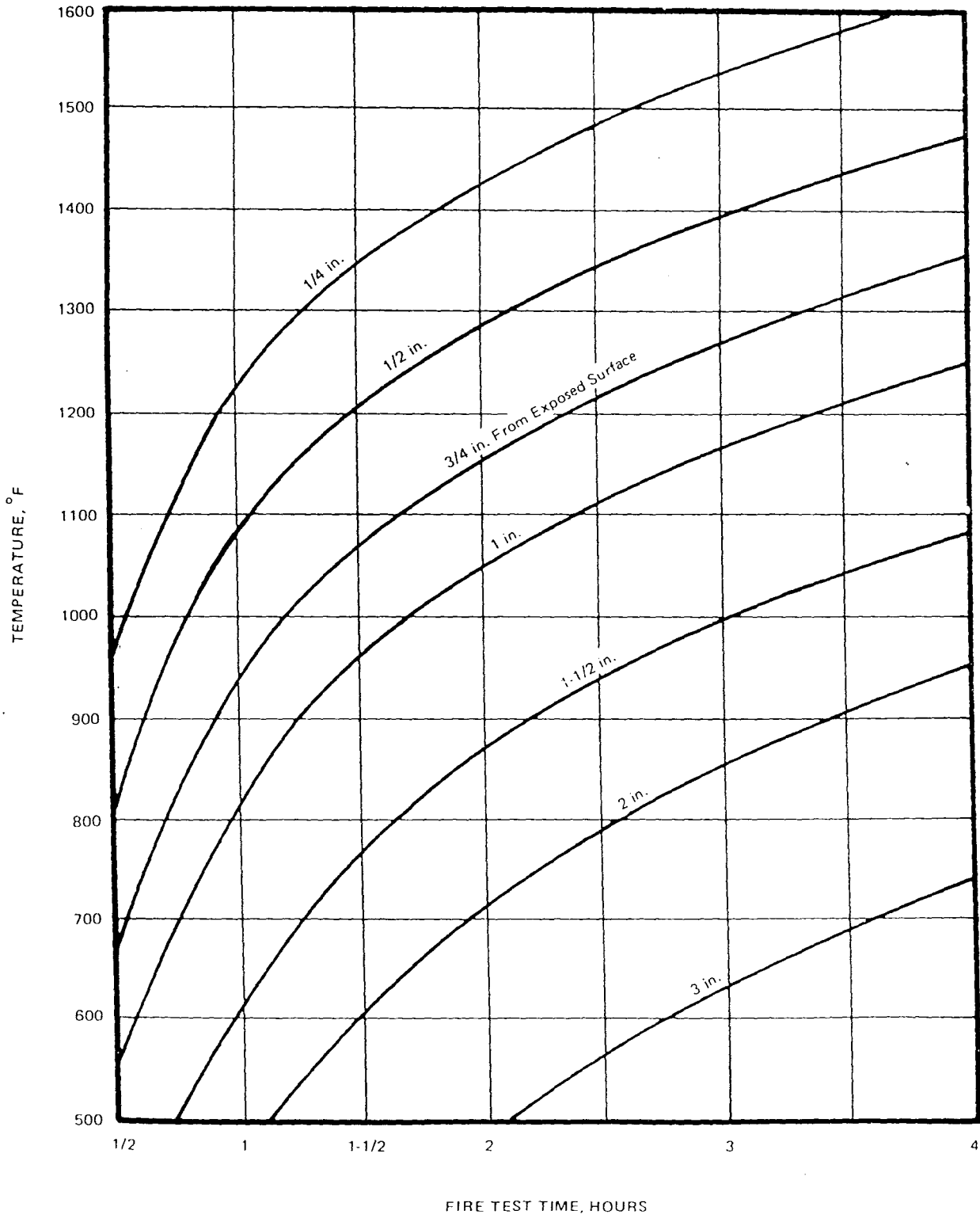

FIRE TEST TIME, HOURS

Fig. B3   Temperatures within solid or hollow-core concrete slabs during fire tests — CARBONATE AGGREGATE.