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Residual Strength of Fire-Exposed Reinforced Concrete Columns

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RÉSUMÉ

On a mené une étude en vue d'évaluer la résistance résiduelle de poteaux de béton armé exposés à un feu normalisé pendant différentes périodes, puis refroidis. Cette étude comportait l'utilisation d'un modèle mathématique, d'une méthode d'essai aux ultra-sons et d'une méthode d'essai de chargement. Les températures et les résistances résiduelles calculées des poteaux testés ont été comparées à celles obtenues par mesure. On a aussi comparé les vitesses des impulsions calculées et mesurées. Les résultats ont révélé que l'emploi du mode de calcul et de la technique de mesure de la vitesse des impulsions décrits dans l'étude permet d'évaluer avec une précision suffisante, à toutes fins pratiques, la résistance résiduelle des poteaux de béton.

Residual Strength of Fire-Exposed Reinforced Concrete Columns

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Synopsis: A study was carried out to assess the residual strength of reinforced concrete columns after exposure to a standard fire for various lengths of time, and cooling. The use of a mathematical model, an ultrasonic pulse test method and a load test method are investigated. Calculated temperatures and residual strengths of test columns were compared with those measured. Comparisons were also made between calculated and measured pulse velocities. The results indicated that using the calculation procedure and the method of measuring pulse velocity described in the study, the residual strength of concrete columns can be assessed with an accuracy sufficient for practical purposes.

Keywords: columns (supports); compressive strength; deformation; fires; fire tests; loads (forces); reinforced concrete; strength; temperature; ultrasonic tests

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INTRODUCTION

The residual strength of fire-damaged concrete structural members is an important factor in determining the feasibility of repair of the structure. The restoration of load-carrying capacity and fire resistance of a fire-damaged structure to an acceptable level is required by relevant building codes. Often, repair is more economical, both in terms of cost and time, than demolishing and rebuilding the structure.

To assess the residual strength of fire-exposed reinforced concrete columns, studies sponsored jointly by the National Research Council of Canada (NRCC) and the Portland Cement Association (PCA) were recently carried out. Both theoretical and experimental studies were performed. These studies include:

- Development of a mathematical model to calculate the temperatures in the columns, and their strength, during and after exposure to fire.
- Subjecting loaded test columns to a standard fire for various lengths of time, and cooling of the columns in an environment of standardized decreasing temperatures; measurement of column temperatures during and after exposure to fire until near ambient temperatures are reached in the column.
- Comparing the measured internal temperatures of the column during and after exposure to fire with calculated temperatures.
- Applying ultrasonic methods of nondestructive testing to monitor changes in concrete material properties of the columns before and after exposure to fire; evaluation of relationships between measured pulse velocities and concrete quality, compressive strength, and modulus of elasticity.
- Determining the ultimate axial strength of the columns after exposure to fire, under conditions of near ambient internal concrete temperatures.

- Comparing measured residual strengths of the columns with calculated values.

In the current studies, two 305 × 305 mm reinforced concrete columns made with siliceous aggregate were investigated. The columns were constructed by the Construction Technology Laboratories of PCA and tested in the laboratories of the Division of Building Research of NRCC. The measured results of these tests and the calculated results are discussed in this paper.

CALCULATION PROCEDURE

The analysis of the performance of a column during and after exposure to fire involves the calculation of temperatures, deformations, and the stresses of the column. The calculation procedure is described in detail in Reference 1 and will not be further discussed here. Information is provided with regard to the environment temperatures and the properties of the concrete and the reinforcing steel during and after exposure to fire.

Environment Temperatures

As in the previous study [1], the temperature course of the environment, i.e. the fire to which the column is exposed, is assumed to follow that of the standard fire described in ASTM E-119 [2]. After exposure to fire the temperature of the environment is assumed to decrease to room temperature according to the relations specified in ISO 834 Standard [3]. Depending on the duration of exposure to the fire, the rate of decrease of temperature is as follows (Fig. 1):

$$dT/dt = 625°C/h \qquad \text{for } t_h < 0.5 \text{ h} \tag{1}$$

$$dT/dt = 250\,(3 - t_h)\ °C/h \qquad \text{for } 0.5 < t_h < 2\text{h} \tag{2}$$

$$dT/dt = 250°C/h \qquad \text{for } t_h > 2 \text{ h} \tag{3}$$

$$dT/dt = 0 \qquad \text{if } T = 20°C \tag{4}$$

where

t = the time in hours,
t_h = the duration of the fire in hours,
T = the fire temperature, at time t, in hours.

Properties of Steel and Concrete

In the calculations, the same equations were used for the thermal properties and stress-strain relations of concrete and steel as those described in Reference 1. However, the studies described in Reference 1 were concerned only with the period of rising fire temperatures. For periods in which the steel or concrete contracts, as is the case during cooling, other stress-strain relations were assumed, in which the stress reduces with the strain according to a straight line that is parallel to the tangent of the stress-strain curve at the origin (Figs. 3 and 5). The stress-strain curves for the reinforcing steel and the concrete at elevated temperatures are illustrated in Figs. 2-5.

As can be seen in Figs. 2 and 4, steel and concrete lose strength when heated to high temperatures. In the calculations it was assumed that steel retains its original tensile strength after cooling. The compressive strength of concrete after cooling, however, is reduced. The reduction in strength of the concrete depends to a high degree on the temperature to which the concrete has been heated and to some extent also on the load to which the concrete was subjected during heating. In the calculations, approximate values derived from literature [4,5] have been used for estimating residual compressive strength of siliceous aggregate concrete. Because information on the dependance of the concrete strength on load is meagre and there is considerable scatter in the values given in the literature for the residual strength, the influence of load was not considered. This tends to lower the values of the estimated residual strength of the concrete. These values can be given as a function of the maximum temperature attained by the concrete, by the following relations (Fig. 6)

for $0°C < T < 500°C$

$$f_r = (1 - 0.001\ T)\ f_{co} \tag{5}$$

for $500°C < T < 700°C$

$$f_r = (1.375 - 0.00175\ T)\ f_{co} \tag{6}$$

for $T > 700°C$

$$f_r = 0 \tag{7}$$

where

f_r = residual strength of the concrete,
f_{co} = initial strength of the concrete,
T = highest temperature attained by the concrete.

TEST SPECIMENS*

The specimens were square, tied, reinforced concrete columns, made with siliceous aggregate. They were 3810 mm long and had a cross-section size of 305 × 305 mm. Twenty-five-mm diameter longi-

*Detailed information on the test specimens is available.

tudinal reinforcing bars and 10-mm diameter ties were used. The location of the main reinforcing bars, which were welded to steel end plates, and the locations of the ties are shown in Fig. 7.

The yield stress of the main reinforcing bars was 444 MPa and that of the ties was 427 MPa. The ultimate tensile strength was 730 MPa for the main bars and 671 MPa for the ties.

The concrete mix was designed for a compressive strength of approximately 35 MPa. The mix proportions were as follows:

Cement	325 kg/m^3
Water	140 kg/m^3
Sand	874 kg/m^3
Coarse Aggregate	1058 kg/m^3.

The average compressive cylinder strength of the concrete of the two columns tested, measured on the day of the fire test, was 38.9 MPa for Column A and 41.8 MPa for Column B. The moisture condition at the centre of Column A was approximately equivalent to that in equilibrium with air of 87% relative humidity at room temperature, and of Column B, with air of 82% RH.

Chromel-alumel thermocouples, 0.91 mm thick, were installed at mid-height of the columns for measuring concrete temperatures at different locations in the cross-section.

TEST APPARATUS

Fire Tests

The fire tests were carried out by exposing the columns to fire in a furnace specially built for the testing of loaded columns and walls. The test furnace was designed to produce the conditions to which a member might be subjected during a fire, with respect to temperature, structural load, and heat transfer. It consists of a steel framework, supported by four steel columns, and the furnace chamber inside the framework. The characteristics and instrumentation of the furnace are described in detail in Reference 6.

Ultrasonic Tests

Ultrasonic tests to measure the pulse velocity of concrete in the column were made using a commercially available instrument, known as a V-Meter [7]. The test arrangement utilizing this equipment is shown schematically in Fig. 8. The V-Meter supplies a train of sharp, electrical pulses to a transmitting transducer in acoustic contact with the concrete surface. The electrical pulses are converted to low frequency (54 kHz) mechanical pulses by piezoelectric elements in the transducer assembly. The mechanical pulses propagate through the concrete and are sensed by the receiving transducer located at a known distance. The mechanical energy is reconverted to electric signal by the transducer. The electrical signals are subsequently amplified and displayed on an oscilloscope. The V-Meter utilizes a precise calibrated time delay circuitry to calculate and display the transit time of the pulse through the concrete. From the transit time and the thickness of the concrete as measured as the straight line distance between the two transducers, the pulse velocity can be calculated. This velocity is a measure of the quality of the concrete, and is related to compressive strength and dynamic modulus of elasticity.

TEST CONDITIONS AND PROCEDURE

Fire Tests

The columns were installed in the test furnace by bolting their steel end-plates to a loading head at the top and a hydraulic jack at the bottom. Concentric loads were applied to the columns about one hour before the fire tests. The load on Column A was 992 kN and that on Column B, 1022 kN.

During the tests the heat input into the furnace was controlled so that the average temperature followed as closely as possible the standard temperature-time relations described earlier in the present paper. The accuracy of control during both tests was such that the duration of the average furnace temperature from that given by the standard temperature-time relations was less than 10°C, except in the first 3-5 minutes of the test and after the furnace had cooled down to below 100°C. Column A was exposed to fire for one hour and Column B for two hours before the start of the cooling period. Measurements were made of the furnace temperatures during the fire exposure and cooling periods until the average furnace temperature reached near ambient temperature. Measurements were also made of the concrete temperatures at various locations during the fire exposure and cooling periods. During these periods the load was kept constant and the axial deformation of the column was measured until the temperatures in the column reached values close to ambient temperature. At that stage, which was reached about one day after the cooling period started, the load on the column was increased at a rate of 12.5 kN per minute until the column failed.

Ultrasonic Tests

Prior to the start of the fire test, over 90 pulse velocity measurements were made through the cross-section at various locations along a vertical profile of the column. A layout of locations of the pulse velocity measurements is presented in Fig. 9.

Pulse velocity measurements were also made on companion 152 × 305 mm concrete cylinders of Column A and Column B. The cylinders were fabricated at the time the columns were cast, and then maintained at curing and storage conditions identical to those of the columns. The pulse velocity measurements were made along both the transverse and the longitudinal axis of the cylinders. The cylinders were subsequently tested for compressive strength on the day of the fire test.

Approximately 20 hours after the start of the fire test, pulse velocity measurements were attempted at each of the test grid locations identified in Fig. 9. At approximately 25 hours after the start of the test, a region near mid-height of the column, between Elevations F and G, was chipped to a depth of 1 to 3 in. (25 to 75 mm) to remove loose, delaminated material. Chipping was performed with hand tools. The resulting exposed shape of the column was nearly round, as shown in Fig. 10. Pulse velocity measurements were also made through the column in the region.

RESULTS AND DISCUSSION

Column Temperatures

Previous studies [1], in which several columns were tested, showed that the mathematical model used for the calculation of temperatures in siliceous concrete columns during fire exposure gives reasonably accurate predictions. In general, predicted temperatures were slightly higher than those measured, probably because of conservative assumptions for the mechanism of heat transfer from the furnace to the column.

The columns tested in the present study were made from the same siliceous aggregate concrete. Using the same procedure and thermal properties described in Reference 1, the temperature history of the columns was calculated. There was again good agreement between the calculated and the measured temperature curves. Figures 11 and 12 present measured and calculated temperatures of the concrete as a function of time for various depths along the centerline of the column.

Ultrasonic Test Method

The use of ultrasonic test methods to evaluate material properties and defects in concrete is widespread [8]. These techniques are based on detecting changes in amplitude, phase, and direction of mechanical waves as they propagate through a concrete member. Changes in wave characteristics generally indicate a corresponding change in internal structural make-up of the concrete.

Pulse velocity measurements made prior to exposure to fire ranged from 4360 to 4665 m/s over the height of Columns A and B. The average measured reading was 4510 and 4560 m/s for Columns A and B, respectively.

Results of pulse velocity tests and corresponding compressive strengths for 152 × 305 mm concrete control cylinders are presented in Table 1.

TABLE 1 Average measured pulse velocity and compressive strength for 152 × 305 mm companion cylinders

Column	Cylinder No.	Average Pulse Velocity (m/s)	Compressive Strength (MPa)
A	1	4570	37.7
A	2	4600	39.0
A	3	4615	40.0
B	1	4665	42.1

Relationships between pulse velocity and compressive strength as determined by tests of cylindrical specimens have been compiled by many investigators. It is important to realize, however, that these relationships can vary significantly between concrete mixes, due to differences in aggregates, water/cement ratios, air content, moisture content, and density. Plots of pulse velocity versus compressive strength from references 9 and 10 are presented as Figs. 13 and 14, respectively. Figure 14 additionally presents a curve identifying "fire damaged" concrete. Due to dehydration, matrix restructuring, and micro cracking, there is a significant decrease in measured pulse velocities for a given compressive strength for "fire damaged" concrete compared to "undamaged" material.

The relationship between residual compressive strength and temperature exposure for siliceous concretes is presented in Fig. 15. This plot, from Reference 4, indicates an approximate residual strength of 70, 50, and 20% of original for concrete exposed to 300, 500, and 700°C, respectively.

Calculations were performed to predict pulse velocity values for Columns A and B exposed to a one-hour and a two-hour fire, respectively. The following sequence was employed:

1. Maximum internal temperatures at various distances from the exposed face were derived from recorded values.
2. Measured temperatures were related to anticipated strength for each zone of temperature exposure within the column, based on data presented in Fig. 15.
3. Strength values for each zone of temperature exposure within the column were related to anticipated pulse velocity values based on data presented in Figs. 13 and 14.
4. Utilizing the "total transit time" computational approach, individual pulse velocity values were combined for each temperature zone to calculate total, through-section pulse velocities.

Post-fire velocity values calculated utilizing the above approach ranged from 1500-2000 m/s for Column A and from 1100-1500 m/s for Column B. These results compare favourably to actual measured values of 1200-2000 m/s for Column A and 1000-1600 m/s for Column B.

Axial Deformation and Strength

Deformation The axial deformations measured during the tests and the calculated deformations are shown for Column A in Fig. 16 and for Column B in Fig. 17. Up to a test time of four hours, there is good agreement between calculated and measured deformations. After this time, the differences between calculated and measured deformations become considerable.

A good agreement between calculated and measured deformations was also found in earlier tests on reinforced concrete columns, which were exposed to fire until the column failed [1]. These tests generally lasted from two to five hours and did not include a cooling period. The good agreement for tests without a cooling period and for the present tests in the first few hours may be attributed to the fact that, for rising temperatures and strains of the concrete and steel, the effect of creep is included in the stress-strain relations used in the mathematical model. Because of insufficient data and complexity of the model, the effect of creep is not included in the stress-strain relations for decreasing temperatures and strains of the concrete and steel. It is likely that this, and shrinkage of the concrete, are the main reasons for the considerable difference between calculated and measured deformations after about four hours.

Strength The calculated residual axial strength of Column A was 1725 kN and of Column B, 2470 kN. The strengths measured after cooling of the columns to near ambient temperatures, were 1987 kN for Column A and 2671 kN for Column B. Calculated strengths are about ten percent lower than those measured.

Factors that may have contributed to the differences include the somewhat conservative prediction of column temperatures and neglect of creep in the cooling period. Implied in this relation is the assumption that the residual strength of the concrete is only a function of the maximum temperature attained by the concrete. It is known that the residual strength is to some extent affected by the load on the concrete during heating. Studies [4,11,12] showed that the residual strength of concrete specimens heated under stress was higher than that of specimens heated unstressed. Above a critical stress of about one-third of the compressive strength of the concrete, however, the residual strength was no longer affected by the stress.

The neglect of the influence of creep and load in the calculations is expected to result in a lower calculated residual strength. It appears, however, that the neglect of the influence of these factors does not introduce large errors. Thus the results suggest that it is jústified to consider the residual strength of a concrete column that has been exposed to fire as principally dependent on the maximum temperatures attained in the concrete.

In Figs. 18 and 19, calculated maximum temperatures reached at various depths in the column are shown, as well as the residual strength of the concrete at various depths after cooling. Figure 18 shows the temperature and strength distribution for Column A, which was exposed for one hour to fire, and Fig. 19 for Column B, which was exposed to fire for two hours.

In these figures also the profiles of the sections are shown after further chipping off of the weaker concrete with handtools a few weeks after the test. The profiles are rather irregular, with most of the concrete coming off at the corners. The concrete at the corners separated from the column along vertical cracks and came off rather easily in essentially triangular-shaped pieces. The cracks were formed after the exposure to fire, and can be attributed to the fact that the inner part of the column was still increasing in temperature, whereas the outer part was cooling substantially.

A comparison between the profiles of the remaining concrete sections and the calculated isotherms indicates that a calculated maximum concrete temperature of about 550°C is a reasonable limit at which the concrete possesses significant residual strength. Because calculated temperatures are about ten percent conservative, the actual temperature at the locations where this limiting temperature is reached, is about 500°C. At this temperature the residual strength of the concrete, immediately after cooling down is still about fifty percent of the original strength. After this, the strength reduces somewhat with time for several days [4,13-15] but the concrete will later rehydrate and gradually regain most of its original strength (Fig. 20).

CONCLUSIONS

Various methods can be used to assess the residual strength of fire-exposed reinforced concrete columns. In this study the use of a mathematical model, an ultrasonic pulse test method and a load test method were investigated. The results of the study indicate that:

1. The column temperatures predicted by the model are in good agreement with those measured, although the predicted temperatures are somewhat conservative.
2. The axial deformations predicted by the model are fairly accurate for the first few hours, but deviate from those measured in the cooling period. The deviation can be attributed to creep, which is taken into account in the model only for the period of rising column temperatures. The creep

did not significantly affect the residual strength of the column.

3. Pulse velocity measured by the ultrasonic test method correlated well with the residual strength and quality of the fire-damaged concrete.
4. Calculated residual strengths of the columns are about ten percent lower than those measured. In the light of the complexity of residual strength evaluation, the agreement between the calculated and measured strengths can be regarded as satisfactory.

In summary, using the calculation procedure and the method of measuring pulse velocity described in this study, the residual strength of concrete columns can be assessed with an accuracy that is sufficient for practical purposes.

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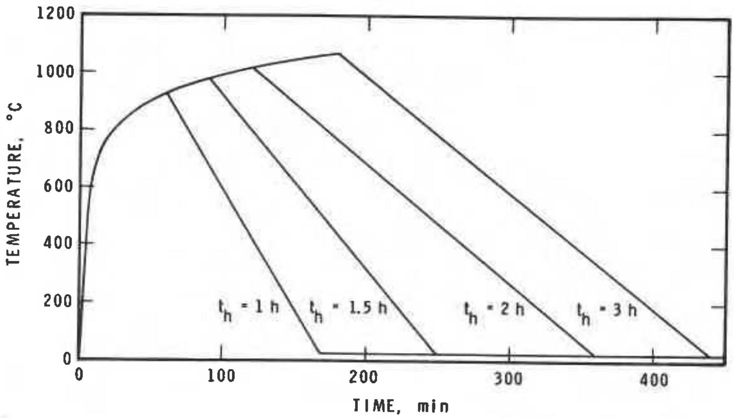


Fig. 1—Temperature of environment as a function of time during and after fire for various durations (t_h) of the heating period

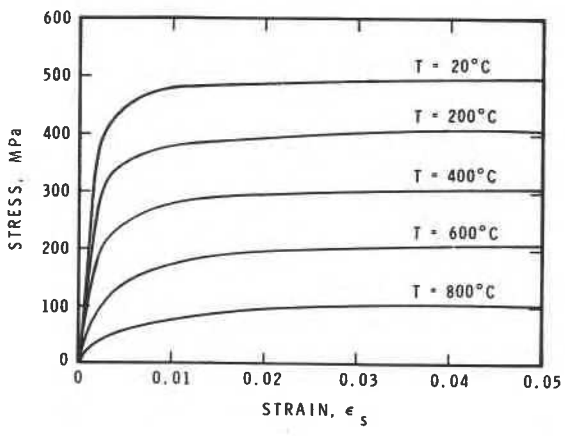


Fig. 2—Stress-strain curves for the reinforcing steel at various temperatures (yield strength = 443 MPa)

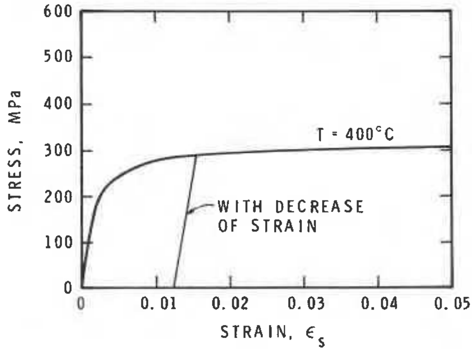


Fig. 3—Stress-strain relationship for steel after decrease of strain

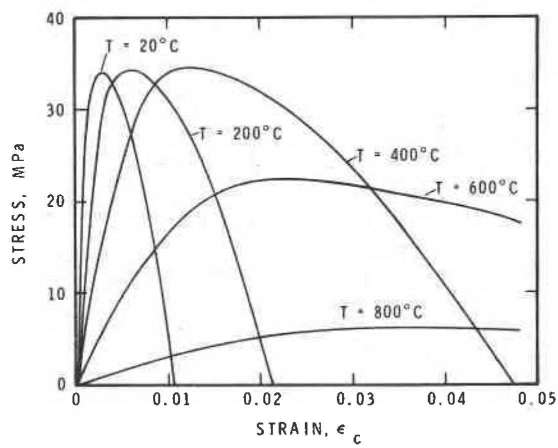


Fig. 4—Stress-strain curves for concrete at various temperatures (compressive strength = 35 MPa)

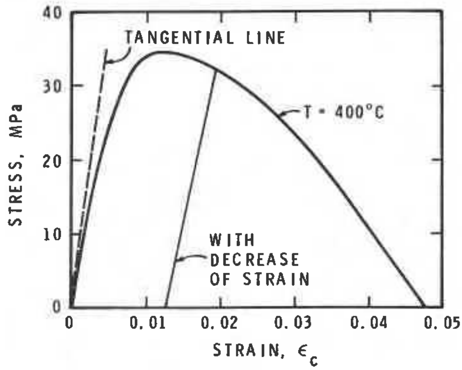


Fig. 5—Stress-strain relationship for concrete after decrease of strain

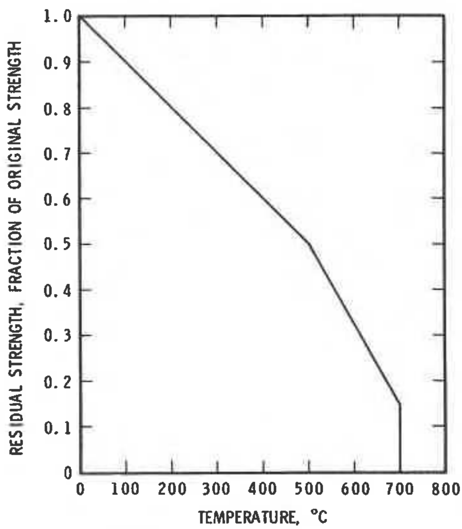


Fig. 6—Residual strength of siliceous concrete after cooling as a function of the temperature attained by the concrete [4,5]

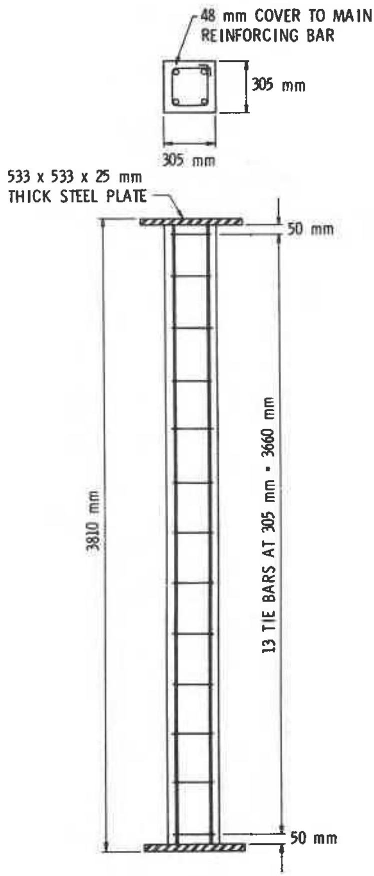


Fig. 7—Test column and location of reinforcing bars

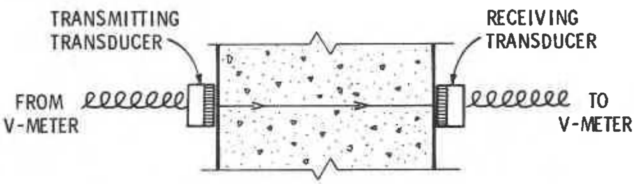


Fig. 8—Schematic of ultrasonic test arrangement

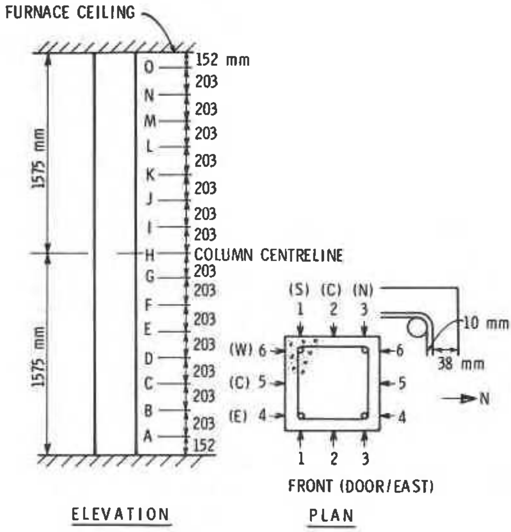
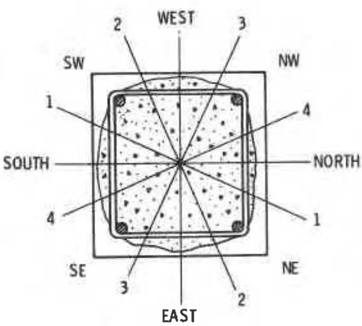


Fig. 9—Location of pulse velocity measurements



DIRECTION	THICKNESS OF CONCRETE, mm
E - W	292
N - S	279
SE - NW	276
NE - SW	287
1 - 1	307
2 - 2	286
3 - 3	285
4 - 4	286
SE BAR - NE BAR	200
SW BAR - SE BAR	201
NW BAR - SW BAR	199
NE BAR - NW BAR	192

Fig. 10—Cross section of column after exploratory chipping

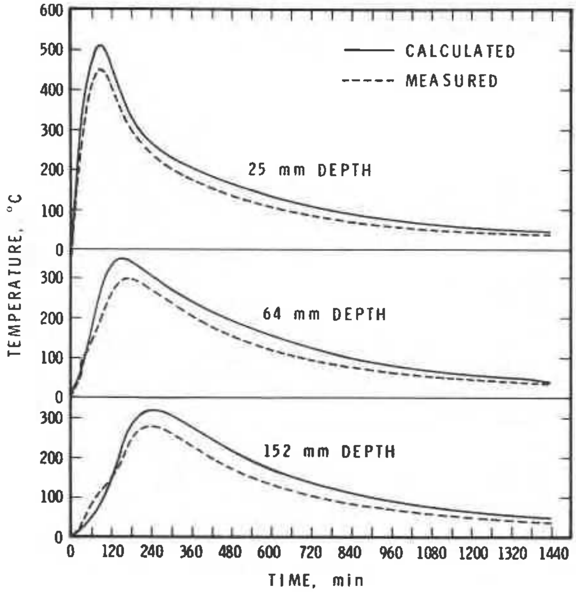


Fig. 11—Temperature of concrete as a function of time for various depths along center line of column A (fire exposure time: 1 hour)

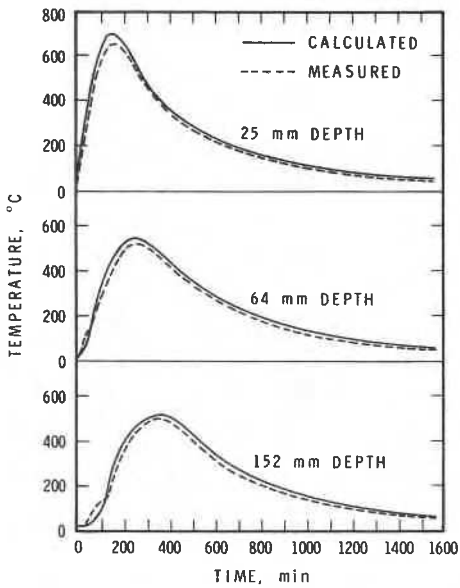


Fig. 12—Temperature of concrete as a function of time for various depths along center line of Column B (fire exposure time: 2 hours)

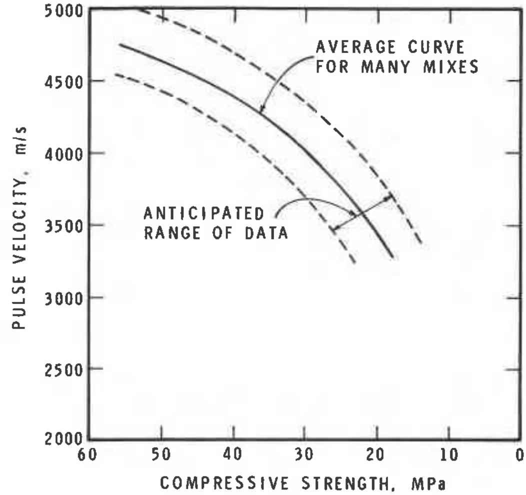


Fig. 13—Pulse velocity-compressive strength relationship [8]

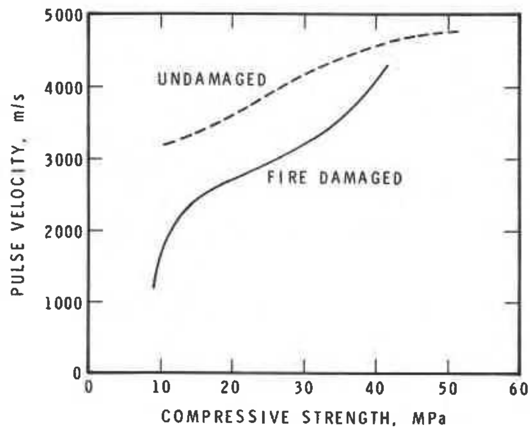


Fig. 14—Pulse velocity-compressive strength relationship [9]

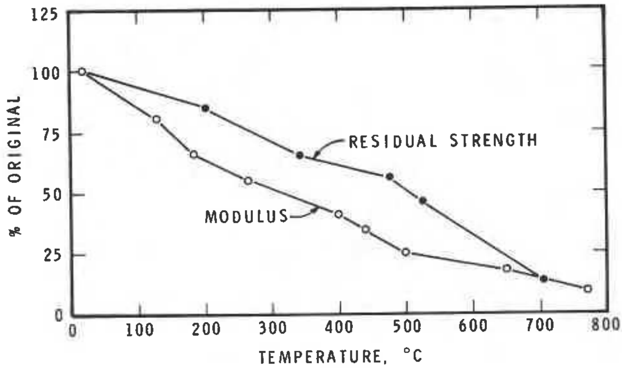


Fig. 15—Residual strength and modulus relation [4]

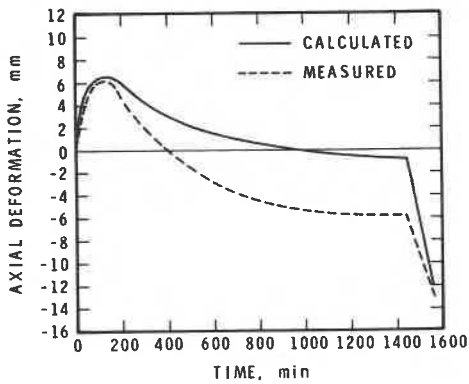


Fig. 16—Axial deformation as a function of time (fire exposure time: 1 hour, Column A)

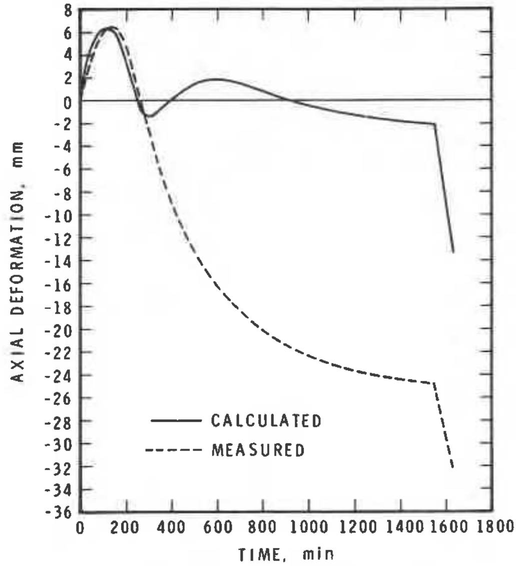


Fig. 17—Axial deformation as a function of time (fire exposure time: 2 hours, Column B)

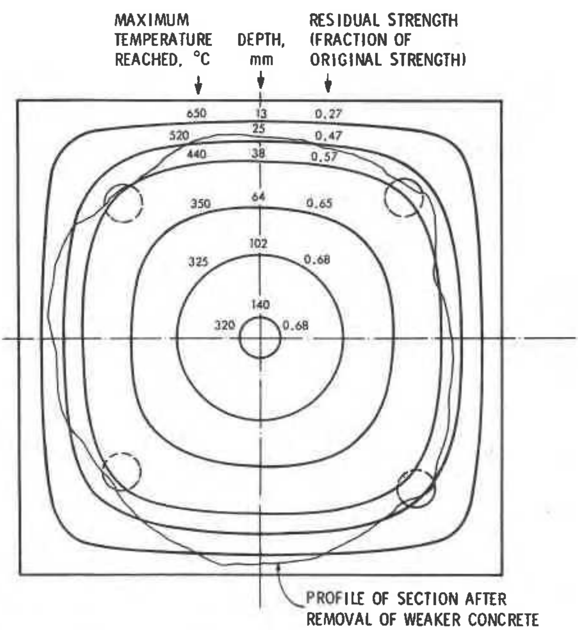


Fig. 18—Calculated maximum temperatures reached at various depths during heating and cooling period and residual strength at various depths after cooling; profile of section after weaker concrete has been removed (fire exposure time: 1 hour, Column A)

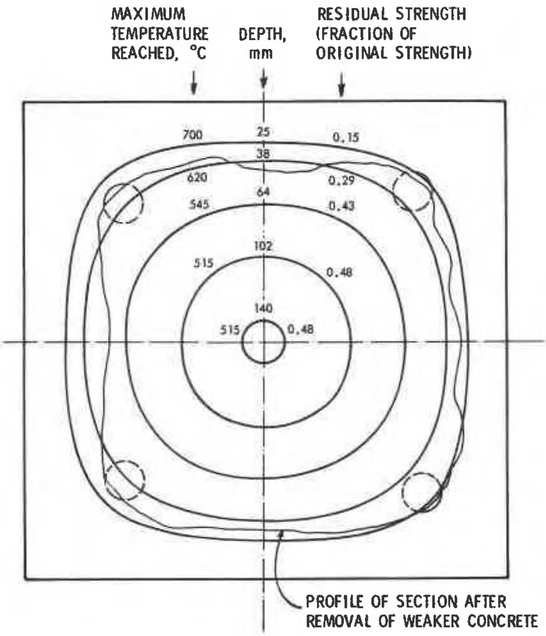


Fig. 19—Calculated maximum temperatures reached at various depths during heating and cooling period and residual strength at various depths after cooling; profile of section after weaker concrete has been removed (fire exposure time: 2 hours, Column B)

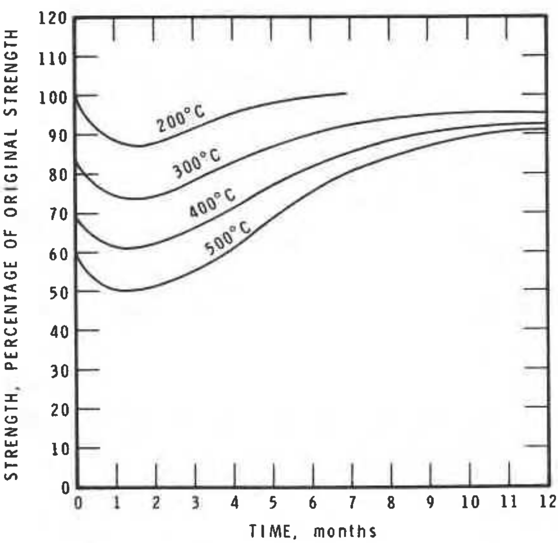


Fig. 20—Natural recovery of compressive strength of concrete, heated at various temperatures [12]