EFFECTS OF HEAT AND THERMAL CYCLING
ON CONCRETE FOR REACTOR SHIELDS

by

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INTRODUCTION

Nuclear energy has been regarded in recent years as a vast new energy source destined to compete with and eventually replace the rapidly dwindling fossil fuels as the source of electrical power. The arrival of this new source of power has brought with it many problems and there remains much engineering development to be done before nuclear power will take its place alongside fossil fuels in normal competitive commercial generation of electrical power. One area where much study and development is needed is that of shield design. The large-scale generation of penetrating radiations and radioactive materials as a result of the use of nuclear reactors creates the need for shields to protect operating personnel and equipment from the harmful effects of these radiations.

The penetrating radiations of concern in reactor shielding are gamma rays and neutrons. All other radiations emitted within the reactor core will be reduced to insignificant levels as a consequence of proper neutron and gamma ray shield design. The biological hazards of radiation stem from the manner in which these radiations interact with human tissue, energize and ionize cells, and upset the chemical balance of these individual cells causing cell death. Considerable cell death within a confined area of the human body will cause extensive tissue damage and even death. Radiations further will alter the electronic and magnetic properties of many materials used in reactor control and operation and render electronic control devices unreliable. The problem of radiation shielding resolves itself then into the problem of attenuation of gamma rays and neutrons to an extent that exposure will not result in damage to exposed personnel and material.

Reactor shields of portland cement concrete have been used predominantly and highly successfully for the past fifteen years. Massive amounts of concrete have been employed to attenuate to tolerable levels these penetrating radiations. The concrete called upon to perform this function has been termed the "biological shield". In order to protect this concrete from the harmful effects of intense nuclear radiations and high temperature elaborate and extensive "thermal shields" have been interposed between the reactor core and the concrete shield.

The Materials Testing Reactor at Arco, Idaho, for example, contains a thermal shield consisting of thicknesses of steel each four inches thick, separated by a space of four inches, through which air is passed to serve as a coolant. The Canadian National Research Reactor at Chalk River, Canada, has a thermal shield consisting of four three-fourth inch steel baffles, a four inch thick steel sheet and a four inch steel floor. Other reactors designed to produce appreciable quantities of heat have similar thermal shielding. There is an increasing tendency in recently constructed reactors, however, to reduce the amount of thermal shielding used to protect the concrete. The exact or even approximate amount of thermal shielding necessary has never been determined. There even remains substantial doubt as to whether or not thermal shielding is necessary in high temperature, power reactors. Before nuclear power can become competitive with other power sources economy must be achieved in many areas. One possible area where great economy is probable is in this field of thermal shield design. A realistic appraisal is needed in this area related to material properties of reactor concretes and their most efficient use in shield construction.

The conservatism prevailing in this field at the present time is fully justified. There has been very little information available pertaining to the behavior of concrete subjected to heat and thermal cycling. There are no specifications to guide the designer and as a consequence thermal shields have been designed to keep the temperature rise and the temperature gradient in concrete to very small values. The thermal shielding necessary to reduce temperatures and thermal gradients to these "allowable limits" can be, as seen in the previously cited examples, an extremely costly engineering task. It is the purpose of this thesis to aid in some small degree in putting shield design on a sound, economic engineering basis by providing information as to how much heat and thermal cycling reactor concretes can actually withstand without excessive deterioration and what kind of reinforcing system, reinforced concrete or prestressed concrete, is best suited to aid in resisting deterioration.

In the research described in this thesis concrete cylinders of high strength portland cement concrete were exposed to varying amounts of heat and varying degrees of thermal cycling to determine the basic ability of concrete to resist these thermal stresses. Test beams were prepared of the same type concrete. Two of these beams were prestressed and a third had the same steel content but was left without prestressing. These beams were subjected to thermal loading under suitable conditions of end restraint. The strains and deflections created by the thermal loading were measured and the surface deterioration was observed.

REVIEW OF LITERATURE

The literature pertinent to the study of concrete in reactor shield design can be divided into two areas. The first relates to the effect of radiation on reactor concrete and the second deals with the effect of heat and thermal cycling on concrete. This review will be concerned primarily with the latter area.

The effect of radiation, gamma rays and neutrons primarily, on the structural properties of concrete has been rather extensively investigated with the widespread conclusion that radiation has no noticeable effect on concrete. D. B. Halliday (7) in 1956 in a summary report by the British Atomic Energy Establishment came to the conclusion that in experiments carried out to that date there would be no noticeable damage over a period of ten years of irradiation at a flux of 10^{12} neutrons per second per square centimeter. Similar conclusions have been reached in studies conducted in the United States Atomic Energy Commission. In some investigations there has been damage reported, but in every case there was high temperature and thermal stresses also involved and there remained the question as to whether the radiation or the high temperature was the mechanism that caused the damage.

In the area of the effect of high temperature and thermal cycling on reactor concrete some scattered work commencing in the late 1940's has been done. Several tests designed to determine the effect of heat on high-density concrete were conducted in the United States Atomic Energy Commission in 1949 by C. R. Brinner and associates (3). Cylinders were heated for seven and fourteen days to 110, 350 and 500 degrees C. The results of the tests showed a gradual deterioration in strength from 7500 psi at 110 degrees C to 3000 psi at 500 degrees C. There was no significant change in strength attributed to the different periods of temperature application.

Theodore Rockwell (16) in a 1950 United States Atomic Energy Commission report discussed several aspects of reactor shielding. He indicated that from the data he had collected the subjects of thermal stress, high temperatures and their effects on concrete were in need of intensive study. He cited the "Clinton Pile" as a reactor without thermal shield in which no noticeable damage to concrete had taken place.

Gallaher and Kitzes (6) in a final report of the United States Atomic Energy Commission Reactor Shielding Development Program in 1953 gave data on the thermal conductivity, specific heat and thermal expansion of barite concrete. They found that barite concrete exhibited no essential differences in properties from those exhibited by ordinary concrete. It is noteworthy that in this final summary report of an extensive development program not one part was devoted to answering the question of whether or not thermal shielding was necessary for concrete.

J. O. Henrie (9), working for North American Aviation, Inc. under a U. S. Atomic Energy Commission contract, investigated in 1954 some of the properties of magnetite iron ore concrete. In compressive strength he found that iron ore concrete compared very favorably to regular concrete with the same cement content. He further stated that the contraction and expansion coefficients should be expected to be of the same order of magnitude in both types of concrete. The effects of high temperatures and thermal cycling were not discussed.

The Reactor Handbook (10) published in 1955 by the U. S. Atomic Energy Commission made reference to the heat resistant qualities of various reactor concretes. Essentially, it reported that ordinary portland cement concrete could withstand up to 350 degree F temperature differentials in three inches. No mention was made of the effect of duration of temperature or thermal cycling on the value of the results reported.

H. S. Davis (5) reported in late 1958 that investigations made on concrete with heavy iron ore aggregate from Running Wolf deposits in Montana indicated that exposures to temperatures greater than about seventy-three degrees F have a detrimental effect on the physical and shielding properties of concrete. He found that loss of strength was quite small for constant exposures at temperatures up to 150 or 200 degrees F and was tolerable for temperatures as high as 500 to 600 degrees F depending on the severity of the thermal cycling. He further stated that little information could be found pertaining to the effects of thermal cycling and therefore cautioned reactor designers to use caution in shield design where thermal cycling is involved.

In so far as the reactor geometry and its effect in causing thermal stresses are concerned, several standard treatments of the subject are available. Timoshenko and Goodier (19) in their book, "Theory of Elasticity", devote one chapter to the effects of temperature gradients along with varying end restraints. H. S. Davis (5) discusses a simplified approach to the same subject using equivalent thermal moments and ordinary structural analysis. There has, however, been little discussion or investigation of the plastic properties of concrete at high temperatures. At the high temperatures to which reactor concretes may be subjected concrete could have properties that are far from ideally elastic. In addition very little has been written about the use of structural steel reinforcing or prestressing to resist thermal stresses caused by high temperature gradients in conjunction with structural restraints. The first reactor using prestressed concrete was completed in France in 1959 and no information is presently available concerning the thermal stress resistant characteristics of the system used.

In so far as reactor design with reinforced concrete is concerned, H. S. Davis (5) suggested a few relationships to be used in the absence of better information. Considering a concrete wall with top and bottom edges fixed and with a linear temperature drop in degrees F he suggested the following formula for determining the nominal percentage of primary steel near the cold side. For an explanation of the symbols used in the following two formulas see the "Notations" section of this paper.

$$P_n = \frac{F_t}{6F_s}(100)$$

He also mentioned that specifications usually called for 0.2 per cent temperature steel to be used in a direction normal to the primary reinforcement. When the amount of temperature steel is known he stated that the following formula can be used to compute the permissible temperature differential.

$$t = \frac{F_s}{\alpha E_s} \left\{\frac{d}{d - kd}\right\} = \frac{100}{1 - k}$$

If all four walls are considered as fixed he stated that the values of t obtained from the preceding equation should be decreased by ten per cent. He further indicated that greater temperature gradients could be tolerated in reactor shields by using larger amounts of steel than would correspond to a "balanced design". However, a substantial increase in steel would be needed to appreciably raise the temperature gradient allowed. Since large increases in steel content rapidly increase the cost of reinforced concrete and since numerous process pipes and test holes are normally required through reactor shields, this normally would not be a practical solution and it would usually be better to try a different shield design.

Concerning the use of prestressed concrete for concrete shielding, as far as this author can determine, no investigations have been made. However, it would seem logical that any structural system that prevents entirely the formation of tension cracks such as prestressing does, would offer advantages in shield systems. Inherent in any power reactor operation are countless operating cycles in which large temperature changes may be involved. The mechanism of opening and closing of cracks as thermal loads are applied to reinforced concrete is to some undetermined degree irreversible and should eventually result in some deterioration. This effect would not be present in a prestressed concrete system.

From the review of literature presented it will be noted that the area dealing with thermal aspects of reactor shield design has only been investigated slightly and much further investigation needs to be done. Further investigation in this field should result in the establishing of an upper limit to the temperature and thermal gradient that can be tolerated by reactor concrete. Further investigation should also see a substantial reduction or perhaps even elimination of costly thermal shielding and provide structural steel or prestressing in a logical and economical manner. It should also determine the effect that thermal cycling will have on introducing progressive deterioration.

DEFINITION AND NOTATION

Definitions

Thermal shield

The shield interposed between the reactor core and the concrete to protect the concrete from high temperature and radiation.

Biological shield

The massive shield which has as its function the reduction of biologically harmful radiations to safe levels.

High-density concrete

Concrete made of cement and high density aggregate such as iron ore, iron punchings or barite ore.

Reinforced concrete

A combination concrete-structural steel load carrying system designed in such a manner that the structural steel takes the tensile stress.

Prestressed concrete

A combination concrete-steel load carrying system designed in such a manner that the concrete is kept in compression.

<u>Thermocouple</u>

A device used to measure temperature changes by the electrical potential changes that are created at the junction of two dissimilar metals in contact upon temperature change.

Notations

P_n	Percentage of primary steel
F_t	Ultimate tensile stress of concrete in psi
F_s	Allowable steel stress in psi
t	Permissible temperature differential in degrees, Fahrenheit
α	Coefficient of thermal expansion of concrete in inches per inch-degree Fahrenheit
d	Distance from extreme compressive fiber to centroid of the tensile steel in inches
k	Ratio of distance between extreme compressive fiber and neutral axis to distance between extreme compressive fiber and resultant tensile steel
E_s	Modulus of elasticity for steel
F.	Fahrenheit temperature scale
C	Centigrade temperature scale
psi	Pounds per square inch

EXPERIMENTAL INVESTIGATION

General Information

The purpose of this investigation was to determine the effect of high temperature and thermal cycling on reactor concrete and to investigate the relative merits of ordinary reinforced concrete and prestressed concrete in resisting thermal deterioration. The investigation of the effect of high temperature and thermal cycling was confined to the investigation of concrete cylinders made from a high strength concrete with a one-fourth inch maximum size of coarse aggregate. The concrete proportions are shown in the following section on the "Fabrication of Test Specimens". A high strength concrete was chosen so that the same type concrete could be used in prestressed beams constructed in the second phase of the investigation. It was thought that by using the same type concrete in both phases that the concrete deterioration due to high temperature and thermal cycling might be isolated from that which might be due to the nature of the end restraints imposed on the concrete beams during heating. A better comparison could then be obtained of the relative merits of prestressing in comparison with ordinary reinforcing. A small size coarse aggregate was used because of the small size of the cylinders and beams that were made and because it was readily available.

After all forty-five cylinders were cast the cylinders were subjected to varying degrees of high temperature and thermal cycling. The deterioration of the cylinders was determined by measuring the compressive modulus of elasticity and the compressive strength of the cylinders. The cylinders were also checked visually for deterioration. A chart displaying the temperature changes to which the various cylinders were subjected to is shown as Figure one.

The relative merits of ordinary reinforced concrete and prestressed concrete were investigated by fabrication and thermal testing of three beams, two prestressed and one reinforced. All beams were made of the same high strength concrete, had the same cross section and contained the same amount of steel. These beams were held in a condition of restraint intended to simulate reactor shield conditions and then subjected to repeated thermal loading. The strains, deformations and temperature gradients in the beams were measured and visual indications of deterioration were noted.

Fabrication of Test Specimens

Test cylinders

A total of forty-five test cylinders three inches in diameter and six inches in height were cast in galvanized steel molds. The aggregate and cement proportions used were designed to produce a concrete which had a minimum compressive strength of 4,000 psi at seven days after casting. Batch quantities of the concrete for a one cubic foot batch were as follows:

13.3 lb of water

31.8 lb of cement

44.2 lb of coarse aggregate

61.6 lb of fine aggregate

All cylinders were prepared in three layers. All three layers were rodded twenty-five times with a three-eighth inch diameter steel rod. The cylinders were allowed to moist cure for seven days at eighty degrees F in their molds. Burlap soaked with water at regular intervals was used to maintain a moist atmosphere. After seven days curing the cylinders were stripped of their molds and placed in an oven maintained at sixty degrees C. The cylinders were kept in this sixty degree C atmosphere for two days to remove any unattached moisture and then thermal cycled. Pictures of preparation of cylinders and oven arrangement are shown as Figures two and three.

Test beams

The test beams were cast in a prestressing bed located in the Iowa Engineering Experiment Station laboratory. Figures four and five show the prestressing bed and end block arrangement. The initial step in the fabrication procedure was to construct the proper side forms modified for the insertion of gage plugs. Seven strand wire cables were then cut to twenty-four foot lengths for use in the beams. The steel base sheet of the prestressing bed was placed and leveled at the proper height and wooden end plates and four and one-half inch wooden side plates were installed. The previously cut wire strands were threaded through the wooden end plates which formed the ends of the prestressing bed. The anchorage at one end was stationary while the plate at the other end was movable. Next load cells to measure the prestressing force in the cables were slipped over the ends of the strands at the fixed plate end and butted against the anchorage plate. These were followed by steel-cased "Strandvise" grips which were installed adjacent to the load cells and also adjacent to the movable plate. Each cable was individually tensioned to a load of 1,000 pounds with the use of a ratchet hoist mechanism and spring scale to set the grips and ensure that all cables would have essentially the same load. The gage plugs were then inserted in the side and base forms with the use of insert screws. A hydraulic jack was used to push the movable head to the position required to tension the cables the desired amount. The proper tensioning was accomplished through the measurement of the strain in the load cells. The load cell lead wires were wired into a switching circuit and from there into an SR-4 strain indicator. Three six foot long beams could be fabricated in the prestressing bed at one time.

The concrete was mixed in a motor driven concrete mixer and placed from the mixer into the forms with shovels. The same mix was used in the beam construction as in making the cylinders previously described. Once the concrete was placed in the forms (only two beams were cast at this time) it was consolidated with an internal vibrator and hand finished on top by troweling. Wet burlap was then placed in contact with the exposed concrete surfaces and kept wet until the time of release of the prestressed cables.

During the concrete placing samples of the concrete used were made into six four and one-half by nine inch compression cylinders. Two of these cylinders were tested for compressive strength at two and four days after casting. The concrete was found to have 5,000 psi at the end of four days permitting the cables to be released.

In preparation for cable release all plug screws were taken out. The prestressed cables were then released slowly by the use of a hydraulic jack. The remaining concrete cylinders were also taken out of their forms at the same time. After release of cables, the cables were retensioned to 500 psi and a third beam was cast. It will be noted that this third beam was not prestressed. Subsequent to casting and curing of this third beam all end cables were cut, forms stripped and beams stored to await testing.

Experimental Equipment

Testing apparatus

Compression testing apparatus   The testing machine used to measure the compressive strength of the cylinders was a Southwark-Emery compression testing machine of 60,000 pound capacity.

Beam-loading frame   The beam-loading frame for the beam thermal cycling tests was fabricated and set up in the laboratory. The loading frame applied the elements of basic beam symmetry to simulate a condition of fixity at the center. Figure eight shows a schematic diagram of the apparatus complete with heating elements and deflection measuring devices. Figures nine and ten show beams being tested in the frame.

Strain measuring equipment

Load cells   Devices called "load cells" were used to measure the strains created in the seven strand high strength steel wires by prestressing. These load cells were steel cylinders to which were affixed on the outside Baldwin SR-4 strain gages on opposite sides. The steel cylinders were drilled with holes so that they fit snugly around the three-eighth inch diameter cables used in prestressing. In jacking strain was introduced in the load cells due to the compressive force exerted by the Strandvise grips wedged directly over the cells. Figure five shows the entire arrangement in detail. The strains created in these cells were measured as changes in electrical resistance in the strain gages on the sides of the cylinders. An amplifying and switching circuit converted this information into direct strain readings from which cable stresses were calculated.

<u>Strain indicator and switching unit</u> In order to convert the SR-4 gage changes in resistance into strain readings a Baldwin SR-4 Type M strain indicator was used. Interposed between the indicator and the load cells was a switching unit to make possible the reading of all cells without changing wires at the indicator.

<u>Mechanical gages</u> Mechanical gaging was used to measure the thermal strain developed in the test beams upon heating. Mechanical measurement was selected over electrical measurement because it was felt that electrical measurement, though more sensitive, would give erratic results at high temperatures. The mechanical gage used was a Whittemore type linear gage manufactured by the Soiltest Company of Chicago, Illinois. The gage length selected was six inches and the gage was read to the nearest ten-thousandth of an inch. Gage plugs were employed to obtain an accurate gage length.

<u>Compressometer</u> A laboratory compressometer with a gage length of four inches and a lever arm magnification ratio of two was used to obtain strain readings during compression testing of the cylinders. The strain data obtained by this

method during testing of the cylinders was plotted against applied load and a curve was fitted to it. A straight line was drawn tangent to the initial third of this curve. By calculating the slope of this initial tangent line, a relative modulus of elasticity in compression for the concrete was obtained. Figure six shows a typical stress-strain curve with initial tangent drawn in as a solid line.

Heat source equipment

Thermocouples Thermocouples were employed at the quarter points of the beams one and one-half inches in from the top and bottom concrete surfaces along the centerlines of the top and bottom beam faces. Figure seven shows the location of the thermocouples and the previously mentioned gage plugs. The purpose of these thermocouples was to determine the temperature gradient across the cross sections of the beams at these locations. Additional thermocouples were placed in the center of each beam and three inches from each end. These thermocouples were employed to obtain an idea of the general temperature distribution along the beams while subjected to thermal loads.

Additional thermocouples were also inserted into the center of one concrete cylinder in each oven used in cycling concrete cylinders. These thermocouples, connected into an automatic recording device, were used to give a minute to minute account of the temperature inside the cylinders in the ovens and thereby give an accurate picture of the temperature cycling applied to the cylinders.

The thermocouples used were made of twenty-four gage copper-constantan wire with fiber glass insulation to resist high temperature deterioration.

Thermocouple recording equipment High speed automatic recording equipment was used to obtain a continuous record of temperature change in all the ovens. Since it was necessary to record several temperature measurements simultaneously a multichannel recorder was used. It was a Speedomax Type G Model S 60,000 potentiometer manufactured by the Leeds and Northrup Company of Philadelphia, Pennsylvania. The recorder had a standard potentiometer type measuring circuit and could record eight separate thermocouple channels.

High temperature ovens The ovens used for thermal cycling with the cylinders were, with the exception of the oven used for the highest temperature cycle, laboratory ovens with 260 degree C temperature ranges. They were made by the Precision Scientific Company and had internal heater ratings of 600 watts. Four ovens of this type were used. Figure three shows the ovens. The same figure also shows the thermocouple equipment wired into the recorder.

The fifth oven employed was a glow bar heated ceramic kiln with a temperature range of 2,400 degrees F. This kiln was equipped with a Leeds and Northrup automatic control and recording device. This kiln was used for the 750 degree F thermal cycling because such temperatures were beyond the range of the ordinary laboratory ovens available.

Beam heating elements The beam heating elements used were laboratory electrical resistance plate type elements with plate dimensions of one by two and one-half feet. They had ranges of 350 degrees F for one element and 900 degrees F for the other element. They were placed directly under and in direct surface contact with the beams subjected to thermal loading. A schematic diagram of the entire arrangement is shown as Figure eight.

Experimental Procedure

Cylinder tests

Forty-five concrete cylinders were cast for thermal cycling. After curing and drying the cylinders were placed in previously calibrated ovens. Five ovens were used and set at temperatures of 200, 300, 400, 500 and 750 degrees F. Eight cylinders were placed in each oven and five were left at eighty degrees F to serve as a control group. While in the ovens the cylinders were subjected to two thermal cycles daily. Two cylinders were taken out after ten, twenty, thirty and forty cycles. All cylinders were tested for compressive strength and compressive modulus of elasticity after thirty-one days. The procedure used in thermal cycling was initially to close the oven doors, turn on the ovens and allow the temperature to build up for five hours. The oven doors were then opened, the ovens turned off and the cylinders allowed to cool for another seven hours. This procedure was then repeated until forty cycles had been reached. By thermocouples positioned in the middle of representative cylinders in each oven the actual variation of the temperature in the cylinders was determined. Figure one shows the temperature variation that occurred in the centers of the selected cylinders. The air quench afforded a fairly rapid method of bringing down the temperature although as the cylinder temperatures approached the ambient air temperature the change became rather slow. This was to be expected.

Beam tests

After the beams were cast they were moist cured seven days, air dried one day and then thermal cycled. The initial step in beam thermal loading was to place the beam in the load frame. The Ames dials for measuring deflections were put in place and adjusted. The thermocouple wires were then attached into the thermocouple recorder. Following this procedure, initial readings were taken of all strain gage lengths and deflection gages and recorded. The thermocouple recorder and heating elements were turned on to start the thermal loading. During thermal loading strain measurements were taken at the start and three and six hours after the start during the first cycle. After twelve hours of heating strain gage measurements were again taken, the heating elements turned off and the beam allowed to cool a further twelve hours. Then the entire cycle was repeated with the exception that no further readings were taken three and six hours after the start of thermal loading during the second, third, fourth and fifth cycles.

The procedure described above was followed for all three beams. The order of testing was first the first prestressed beam designated as beam one. The second beam tested, designated as beam two, was the structural steel reinforced beam and the third, designated as beam three, was the second prestressed beam. The one variation of thermal loading that was introduced was in the temperature of the heating elements and, consequently, the beams. Beam one was kept at a 350 degree F heating plate temperature. Beams two and three were kept at the maximum temperatures allowed by each of the heating elements. After five cycles of heating the beams were examined for signs of deterioration and pictures were taken of the beams to illustrate the deterioration that had taken place. These pictures are shown in the "Results" section of this thesis.

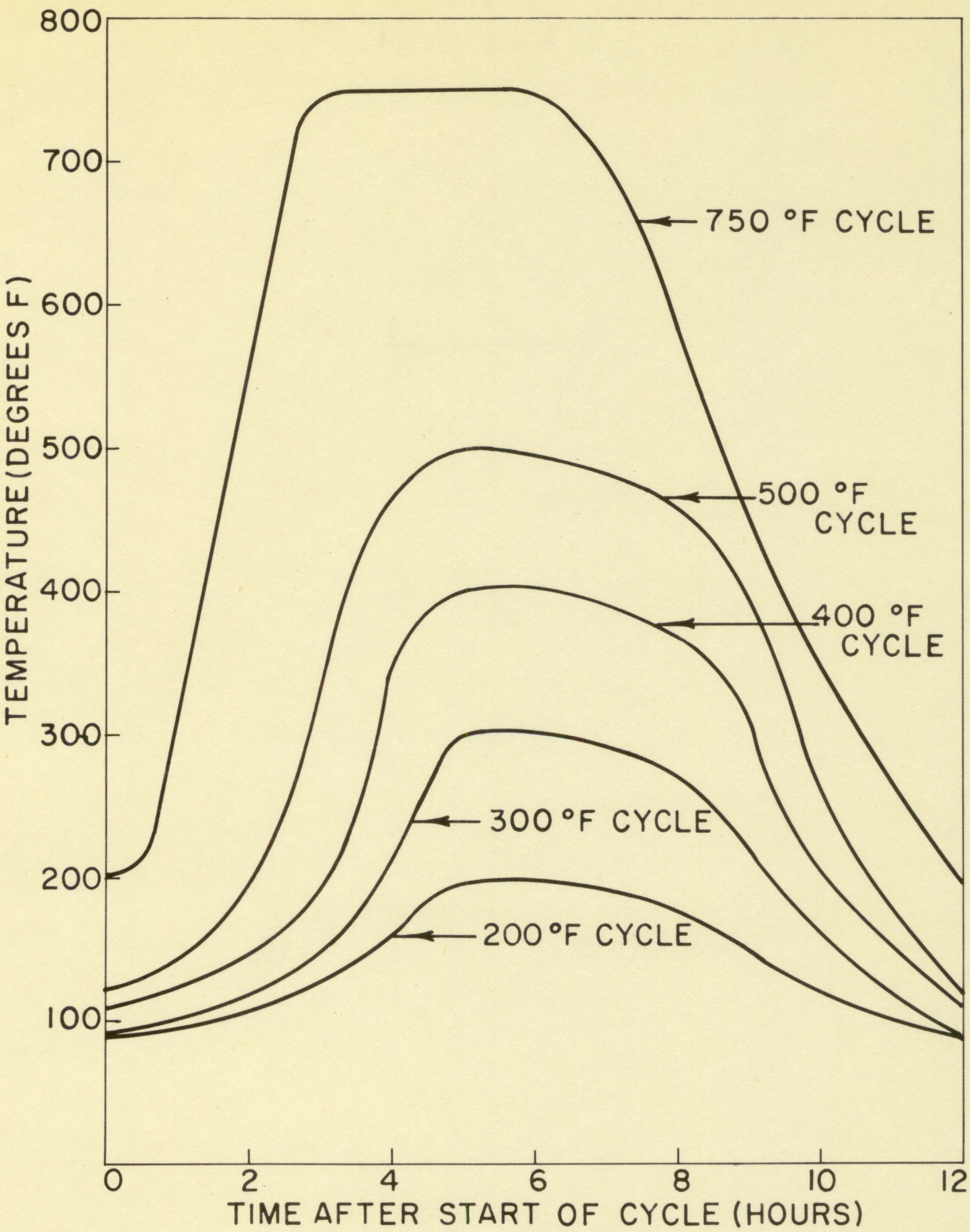


Figure 1. Variation in temperature of concrete cylinders during one typical thermal cycle (for 750 degree F cycle, air temperature shown; for all others, temperature at center of cylinder shown)

Figure 2. Capping of compression cylinders in preparation for testing

Figure 3. Oven arrangement for thermal cycling of compression cylinders

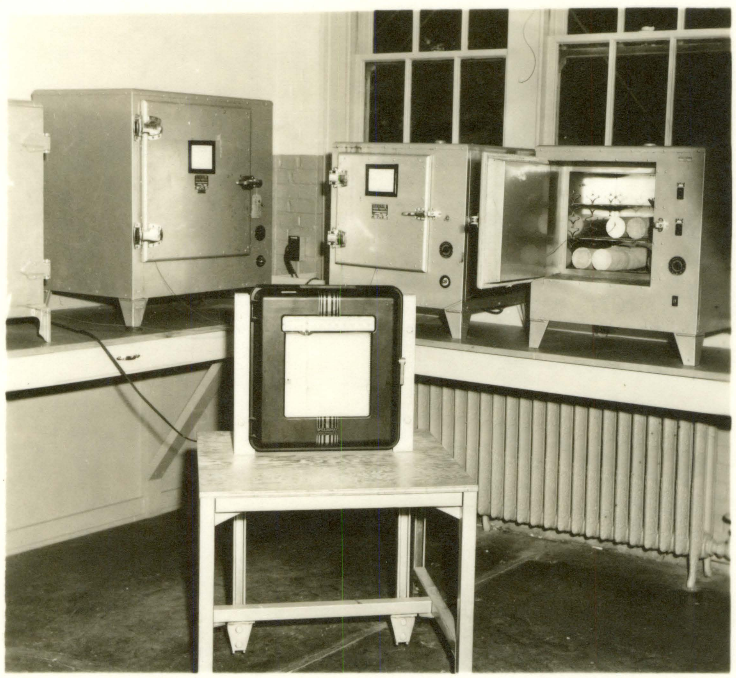
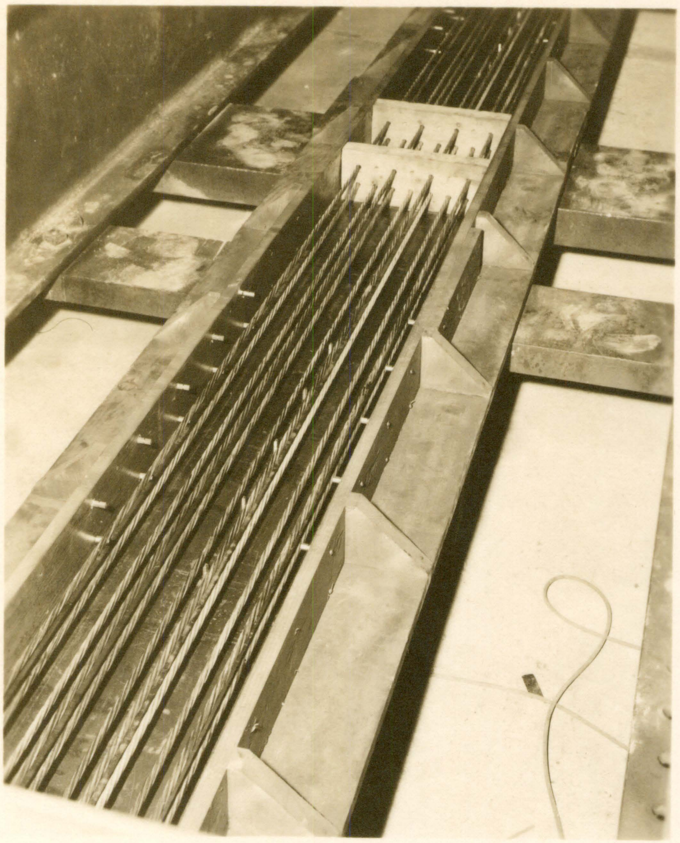
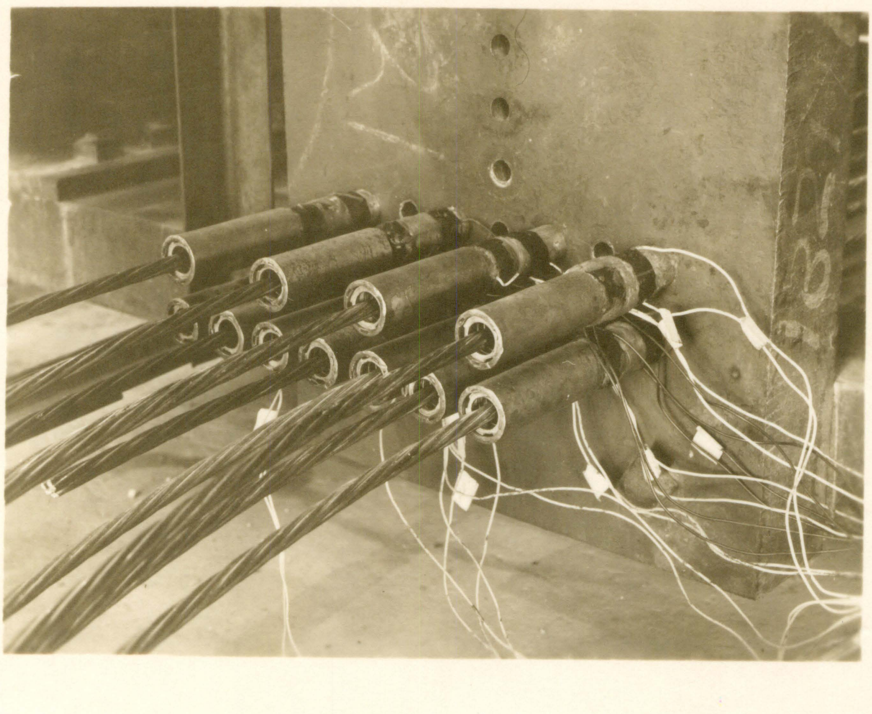


Figure 4. Prestressing bed with side forms and cables in place

Figure 5. End block arrangement showing load cells, cable grips and ends of cables





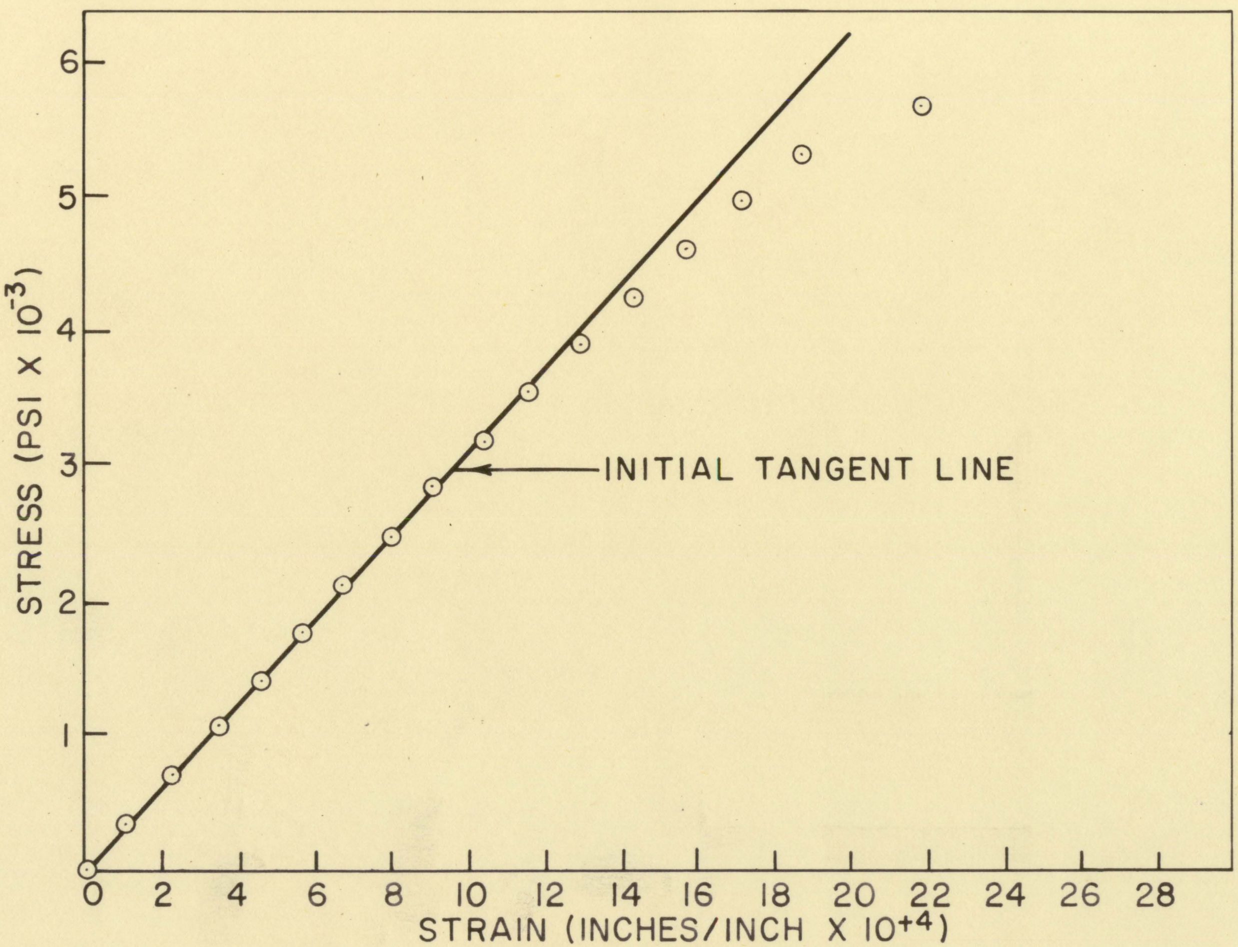


Figure 6. A typical stress-strain diagram showing plotted points and initial tangent line

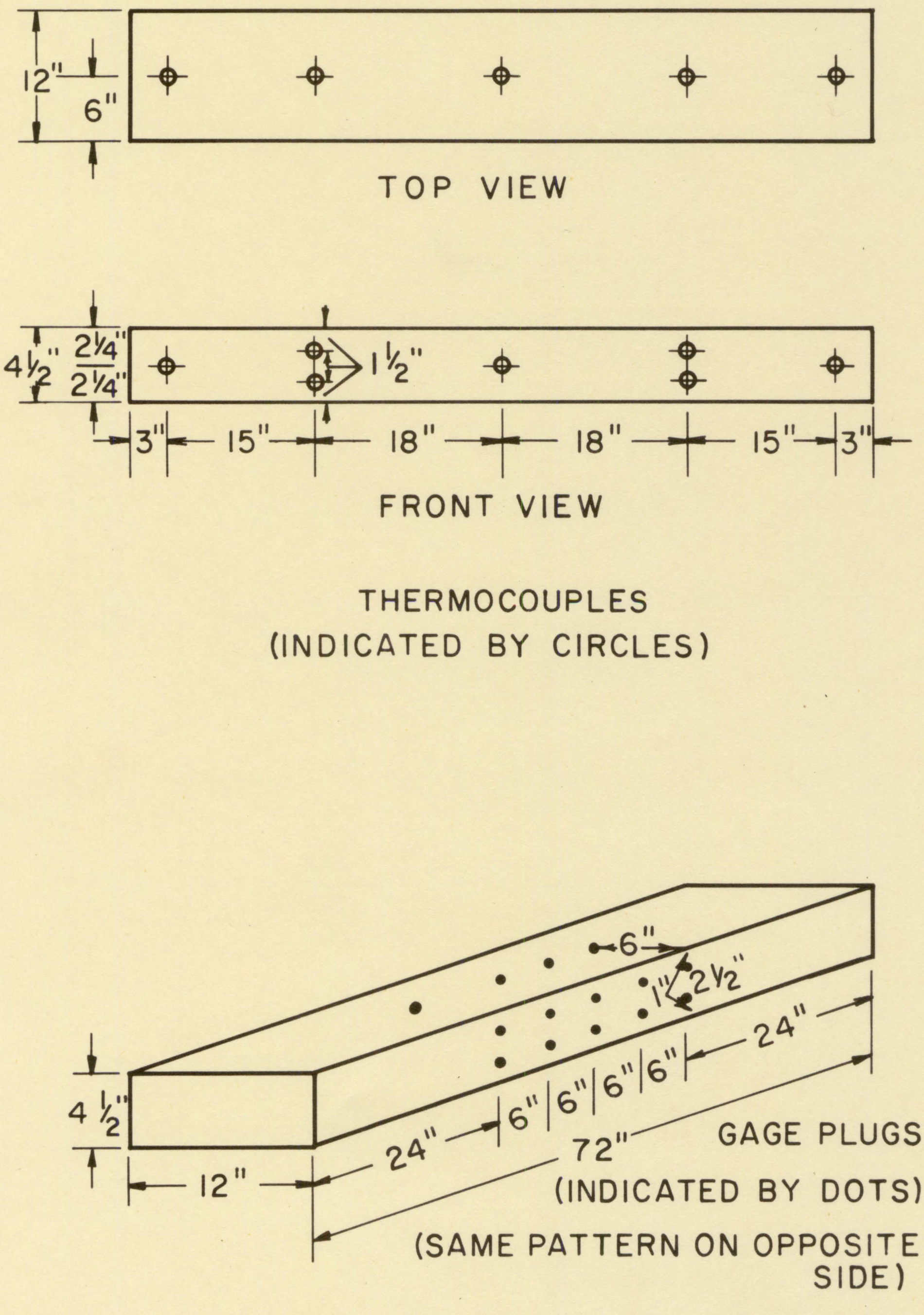


Figure 7. Location of gage plugs and thermocouples on test beams

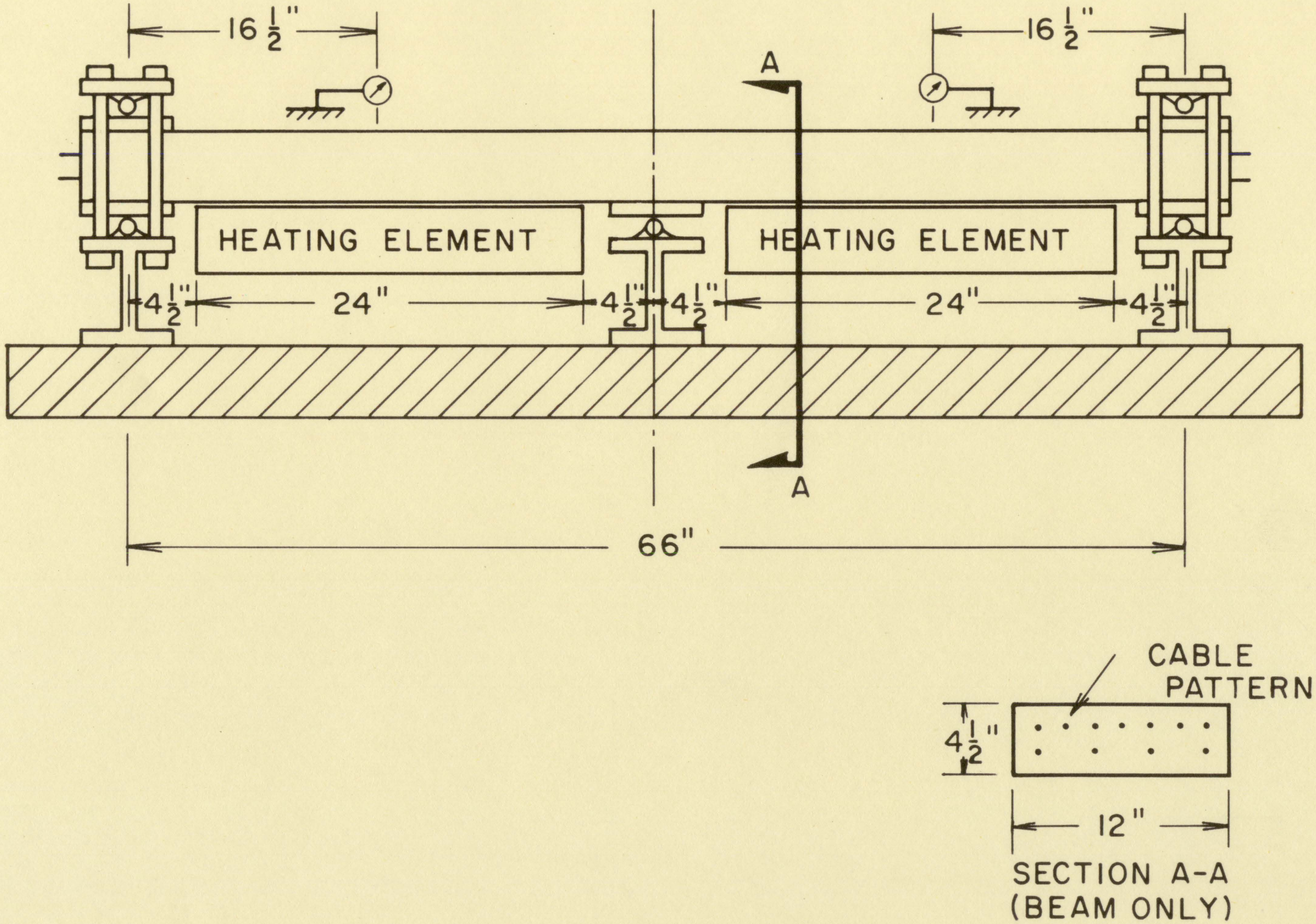
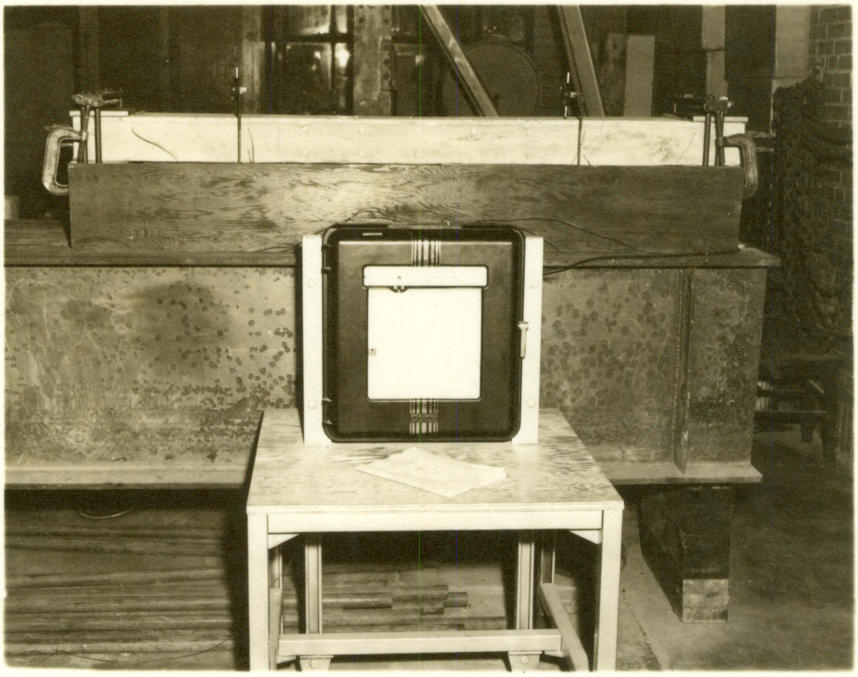
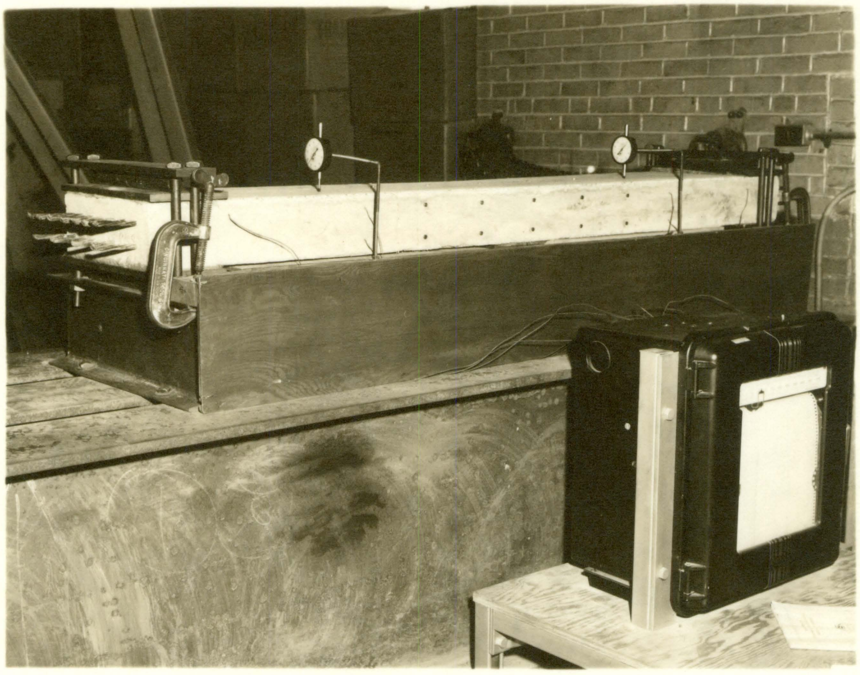


Figure 8. Thermal loading and deflection measuring arrangement for tést beams

Figure 9. Front view of beam, load frame and potentiometer

Figure 10. Side view of beam, load frame and potentiometer





RESULTS AND DISCUSSION

Cylinder Tests

The results of the cylinder testing are illustrated in Figures eleven through seventeen of this section. These results merit some discussion. It will be noted first of all that the actual data shown by plotted points and straight vertical lines on the figures show that there is considerable dispersion in the ultimate compressive strength results. This dispersion is not wholely reflected in the five reference cylinders tested without thermal cycling. In the case of the five reference cylinders all compressive strengths were found within the region of 6,400 psi to 6,600 psi, a considerably narrower range than those shown after thermal cycling. It must be concluded from these results that heating introduces a certain additional degree of randomness in the concrete compressive strength property.

An indication of the consistency or lack of consistency of data can be obtained from a comparison of the modulus of elasticity and ultimate compressive strength data shown for each temperature. In comparing this data by inspection of Figures eleven through seventeen it is seen that in all cases a finer, more consistent set of plotted values is obtained by using the modulus of elasticity as a criterion of deterioration rather than ultimate compressive strength. For example, in Figure eleven showing the results of thermal cycling at 200 degrees F, there is a slight deterioration evidenced by the modulus of elasticity data. The same trend but with considerably more scatter and uncertainty is also evident in the compressive strength curve. This is even more clearly illustrated in Figure twelve showing the variation in compressive strength and modulus of elasticity with thermal cycling at 300 degrees F. In this case a downward trend is evident in the compressive strength data but there is too much scatter to obtain any reasonable indication of the type of downward trend (or effect of repeated cycling on deterioration). With the modulus of elasticity curve this is not the case. The curve illustrating the change in modulus of elasticity with thermal cycling shows a definite drop or increase in deterioration with increase in thermal cycling up to a point and then shows a subsequent leveling off. This trend of obtaining consistent data and a better indication of the effect of deterioration by the use of the modulus of elasticity is shown in all cases.

Another item also of interest is shown in Figure eleven. At 200 degrees F there appears that there might actually be a slight initial increase in strength with thermal cycling rather than a decrease as would be expected. A possible explanation for this unusual behavior is that the beneficial effects attributed to accelerated hydration and the driving off of remaining unattached moisture caused by slight elevations of temperature override the initial deterioration resulting from such temperature increases.

The most significant result of the thermal cycling with concrete cylinders is in answering the question of what effect, if any, thermal cycling has on the deterioration of concrete. The answer to this question can be obtained through an examination of Figures eleven through fifteen. Figure eleven displays the change in compressive strength and modulus of elasticity at 200 degrees F. This figure shows that with the cylinders tested at this temperature there was possibly a slight increase in compressive strength up to twenty cycles and then a slight tapering off to forty cycles which was the extent of thermal cycling. The modulus of elasticity curve at the same temperature remains essentially constant out to about twenty cycles and then also tapers off gradually. Both curves indicate that some small deterioration is taking place with increased cycling. However no real conclusion can be drawn from this series of cylinders at 200 degrees since the cycling was only carried out to forty cycles. One can see from this test that at low temperatures (around 200 degrees F) thermal cycling would have a very small deterimental effect out to forty cycles.

In Figure twelve showing the results of testing at 300 degrees F more meaningful results were obtained. Here it is seen that in the modulus of elasticity data a significant trend is indicated. With thermal cycling there was a progressive and fairly uniform increase in deterioration out to twenty cycles. After this amount of cycling the modulus of elasticity values remained essentially constant indicating no noticeable further deterioration with increased cycling. At 400 degrees F the final stable values of compressive strength and modulus of elasticity were both considerably lower than at 300 degrees F. At 500 degrees F a very pronounced trend had developed. As seen in Figure fourteen the initial slopes of the modulus of elasticity and compressive strength curves are both very steep indicating rapid initial deterioration. However, at forty thermal cycles the modulus of elasticity data indicated that ultimate deterioration had been reached. At 750 degrees F, as shown in Figure fifteen, virtually all deterioration had taken place inside of ten cycles. The final values of both the modulus of elasticity and compressive strength were also the lowest of any series of cylinders tested. Subsequent heating does not indicate any further noticeable damage past ten thermal cycles.

Lastly, it would also be of interest to look at how the compressive strength and compressive modulus of elasticity vary as a function of the temperature. In the previous discussion it was strongly indicated that at high temperatures, beyond a very limited range of cycling, thermal cycling had virtually no effect on the final values of either the compressive strength or the modulus of elasticity. Figure sixteen shows a plot of the variation in compressive strength and modulus of elasticity with temperature after ten thermal cycles. Figure seventeen shows the same type of plot after the cylinders had been exposed to forty thermal cycles. The plot after ten thermal cycles shows a slight humping at low temperatures whereas the plot of compressive strength and modulus of elasticity vs. temperature after forty shows an almost straight line relationship in both compressive strength and modulus of elasticity. This "humping" at low temperatures can be attributed to the fact that at low temperatures after ten thermal cycles the final compressive strength and modulus of elasticity values had not been reached.

An extrapolation to the horizontal axis of the modulus of elasticity vs. temperature line at forty cycles results in a zero value of modulus of elasticity at around 900 degrees. Similarly, an extension of the compressive strength line to the horizontal axis results in a zero compressive strength at around 1,300 degrees F. The validity of such extensions are questionable and would have to be borne out by experimentation. However, it does give an indication that general deterioration due to heat is expected in the region of 900 to 1,300 degrees F. Concrete can be expected to reach its absolute limit of usefulness in reactor shielding in this region.

All of these results point to the following conclusions.

First, deterioration increases only up to a certain limit with thermal cycling. Secondly, with increase in temperature the number of cycles required to reach this limit becomes less and less. By 750 degrees F limited cycling is required to attain maximum deterioration. Thirdly, the rate of deterioration until an ultimate value is reached becomes progressively greater and greater with higher temperatures. This can be seen from the increasingly negative slopes on the modulus of elasticity curves with increased temperature. Lastly, it should be noted that nothing has been mentioned about the surface condition of any of the cylinders cycled. The tests indicate that thermal loading would impair the load carrying capacity of the concrete. However, even at 750 degrees F little visual evidence of progressive surface damage could be found. For 750 degrees F in a non-load-carrying role such as might be the case in certain reactor shields, there is no evidence that concrete would need protection from heat. Note however that the comment on tolerances to heat with no load-carrying role can be a deceptive one. Concrete, if not allowed to expand freely (subjected to restraint), is subject to loading that could result in deterioration.

It will also be seen that in all previous discussions it was tacitly assumed that thermal cycling was the cause of deterioration. This might not necessarily be true. There remains, of course, uninvestigated the possibility that the same duration of heating alone could cause an equal deterioration. In order to investigate this possibility it would be necessary to conduct the same tests again with identical temperature conditions for the same periods of time without thermal cycling. This was not possible in this experimentation due to the limited time that heating equipment was available. However, considering the results obtained in this thesis, it would in a great many situations be immaterial as to whether the thermal cycling or the high temperature caused the reduction in concrete strength. Any design incorporating into the shield concrete at high temperatures will generally also have the small amount of cycling associated with it that was shown in this paper to result in ultimate deterioration.

Beam Tests

In addition to the concrete cylinders thermal cycled, three concrete beams were subjected to thermal loading. Two of the beams, as mentioned previously, were prestressed with seven strand wire cable reinforcement and the third was reinforced with the same type cable but left unprestressed. The intent was to determine the effect of thermal cycling on prestressed and reinforced concrete subjected to end restraint. In so doing it was felt that it might be possible to determine an upper limit to the temperature such a configuration could withstand and also determine the steel reinforcing system better adapted to resist thermal cycling and thermal stress. Since only three beams were thermal loaded, no conclusive results could be expected and definitely none was obtained. Several results indicated however where further study might be warranted.

The first and perhaps most important result was in the area of general surface deterioration by thermal cycling. The first beam (prestressed) was subjected to a moderate thermal load with a 325 degree F heating element temperature. This temperature was chosen because it was felt desirable to have both heating elements at the same temperature and this temperature was at the maximum capacity of one of the elements. After five thermal cycles no deterioration was observed. Further, no crack pattern or excessive deflection was observed. The second beam (unprestressed) and the third beam (prestressed) were thermal cycled at the maximum capacity of both heating elements. That was 325 degrees F for the previously mentioned element and 900 degrees F for the other element. This thermal cycling at maximum heating element capacity gave a maximum temperature in the beams of 640 degrees F and a maximum temperature gradient across the beams of 420 degrees F after twelve hours of continuous heating. Both these maximums occurred approximately in the center of the span subjected to the 900 degree F plate temperature. Figure eighteen shows this variation in temperature at the center of the beam top and bottom faces caused by the maximum heating. These results shown in this figure were obtained by linearily extrapolating out to the beam faces the results obtained at thermocouple recording points. These thermocouples were actually located one and one-half inches in from each beam face as shown in Figure seven.

After five thermal cycles with the second beam some differences in surface condition from that of the first beam were observed. Small hairline cracks had formed in the beam. Figure twenty shows the crack pattern observed. The crack pattern almost invariably went through the gage plug locations. In the top of the beam the crack pattern extended well into the center of the beam from the top face. In the lower part of the beam the crack pattern was localized around the lower gage plugs. This concentration of cracks through gage plug locations may be explained by the stress concentration that will be created around such discontinuities in the concrete. Unequal thermal expansion by the concrete and brass plugs during thermal cycling and the strain imposed on the plug sockets by use of a mechanical gage also should have aided in the formation of cracks through these locations.

These effects alone explain the localized cracks formed around the bottom gage plugs. The cracks through the upper side of the beam can be attributed to tension in the top of the beam cross section resulting from the thermal load and conditions of end restraint imposed. The crack pattern formed was not unlike that formed in ordinary reinforced concrete beams loaded in the usual manner. It was further not open or pronounced enough to justify any conclusion that further thermal cycling would have resulted in an extension of the pattern or deterioration. The third beam tested was the second prestressed beam. This beam was also loaded with a maximum thermal load. Under these conditions no noticeable cracking or other deterioration occurred in this beam with five thermal cycles.

In summary it will be concluded that under the maximum thermal loading at the center of the hotter span of 640 degrees F, a maximum temperature differential of 420 degrees F in four and one-half inches and subjected to thermal cycling five times, no noticeable deterioration occurred.

A second result of general interest is the inelastic deflection created in the beams by thermal cycling. This inelastic deflection was measured by taking the difference between dial readings at the start and end of each complete cycle (one half cycle of heating and one half cycle of cooling). The dials read were located in such a manner as to measure the deflection at the center of each span. In all three cases large amounts of inelastic deflection were introduced by heating. In all three cases this occurred as deflection directed away from the hotter face of the beam. Figure twenty-one shows the deflections created in beams two and three. Several experimental difficulties encountered with beam one during thermal loading resulted in measurements of little value and hence the data for this beam is not shown. It is interesting that the inelastic deflection occurred in the direction opposite that of the elastic deflection and that almost all of the inelastic deflection occurred during the first thermal cycle.

The strains created in the beams due to thermal loading were also measured. Figure twenty-two shows the maximum strains induced in the second and third beams at four selected cross sections due to the first, third and fifth thermal cycles. These strains were computed by taking the difference between initial gage length readings taken before the start of a thermal cycle and the readings taken just before the start of the cooling half of the same cycle. The readings shown were obtained by averaging the results and extrapolating to the beam faces from the gage points. In spite of the end restraints imposed by the load frame, much of the strain created in the beams was due to the thermal expansion of the concrete rather than the bending moment created in the beams due to the nature of the end restraints.

It should be noticed that the maximum total strains were recorded at section D - D. This was to be expected since section D - D represented a gage in the hottest part of the beam.

It should also be noticed by reference to Figure twenty-two that a great deal less strain due to bending was introduced in the beams during the third and fifth thermal cycles. It must be concluded from this that the beams just did not behave elastically after initial heating. A possible explanation of this behavior is that with initial heating there was enough structural breakdown within the concrete to prevent it from acting subsequently as a solid mass.

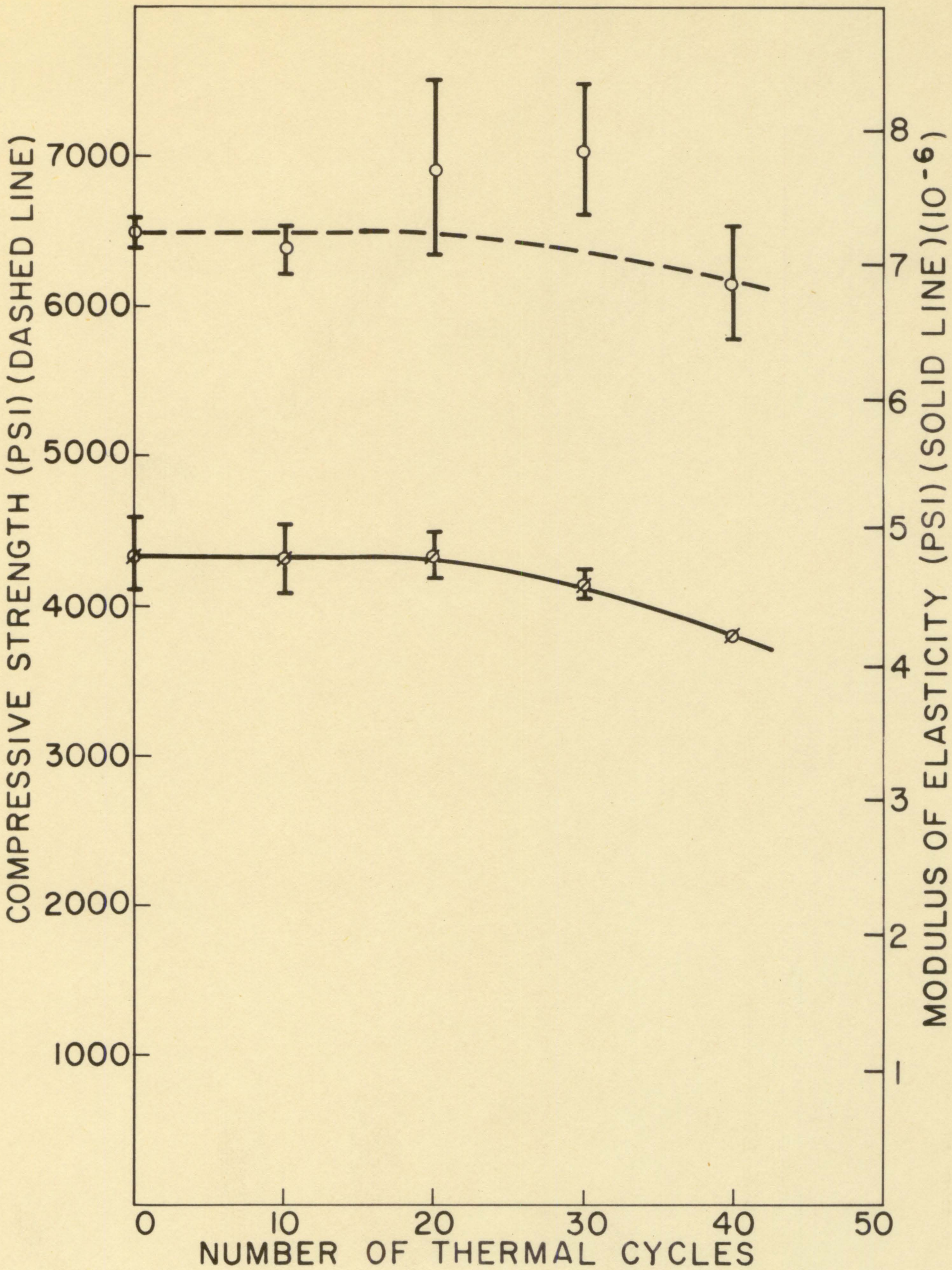


Figure 11. Variation of compressive strength and compressive modulus of elasticity with thermal cycling at 200 degrees F (cylinders tested thirty-one days after casting)

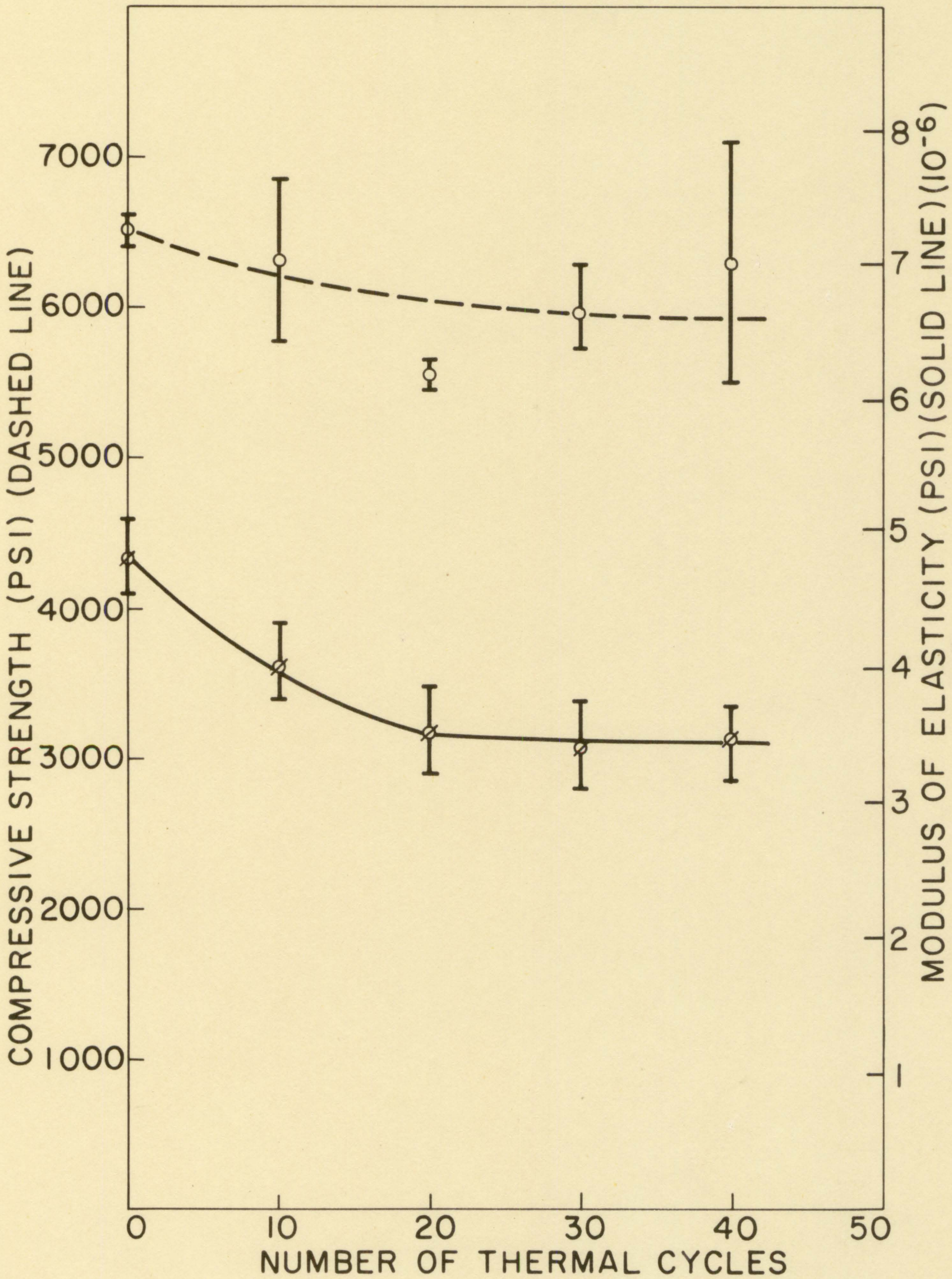


Figure 12. Variation of compressive strength and compressive modulus of elasticity with thermal cycling at 300 degrees F (cylinders tested thirty-one days after casting)

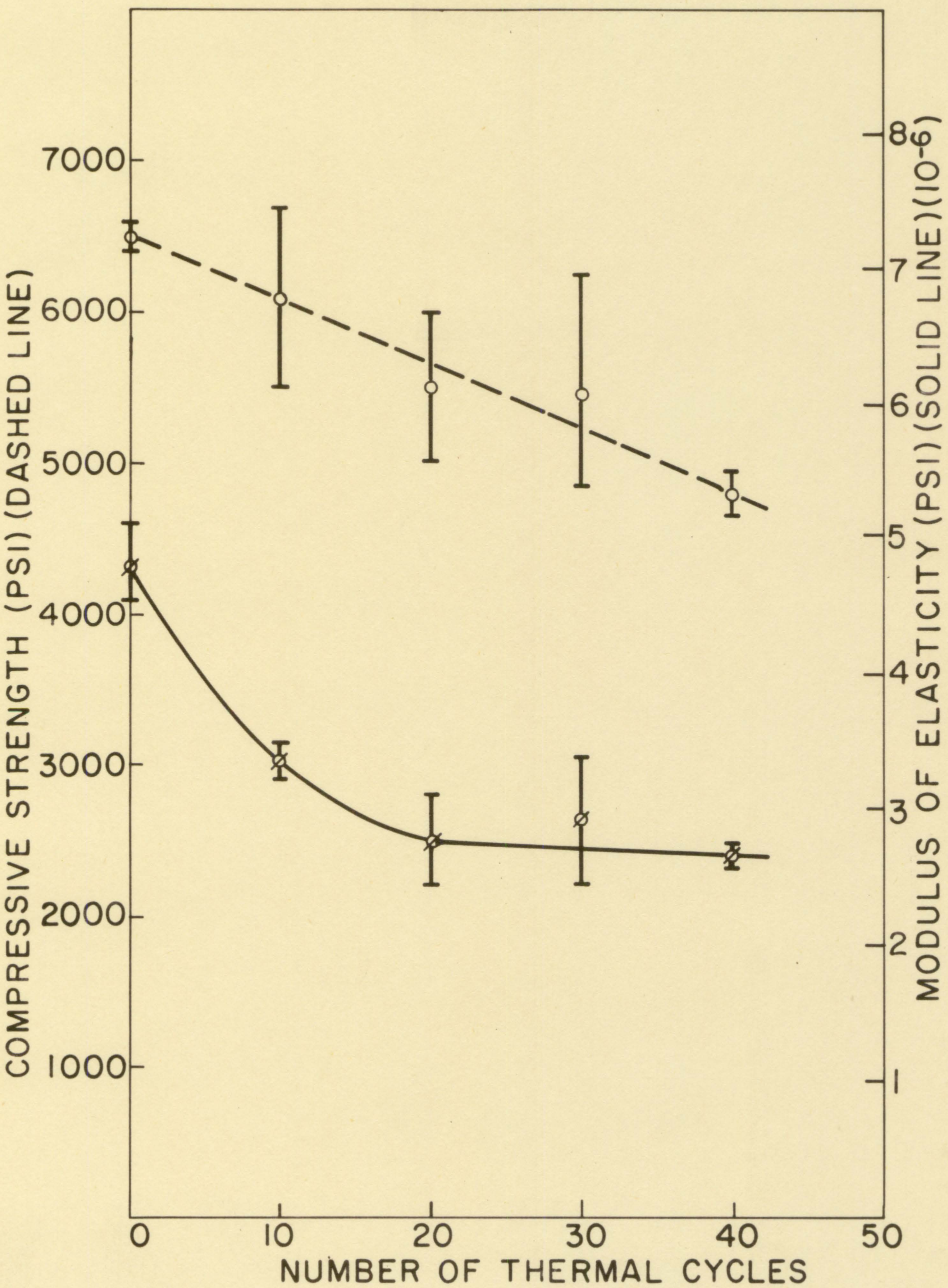


Figure 13. Variation of compressive strength and compressive modulus of elasticity with thermal cycling at 400 degrees F (cylinders tested thirty-one days after casting)

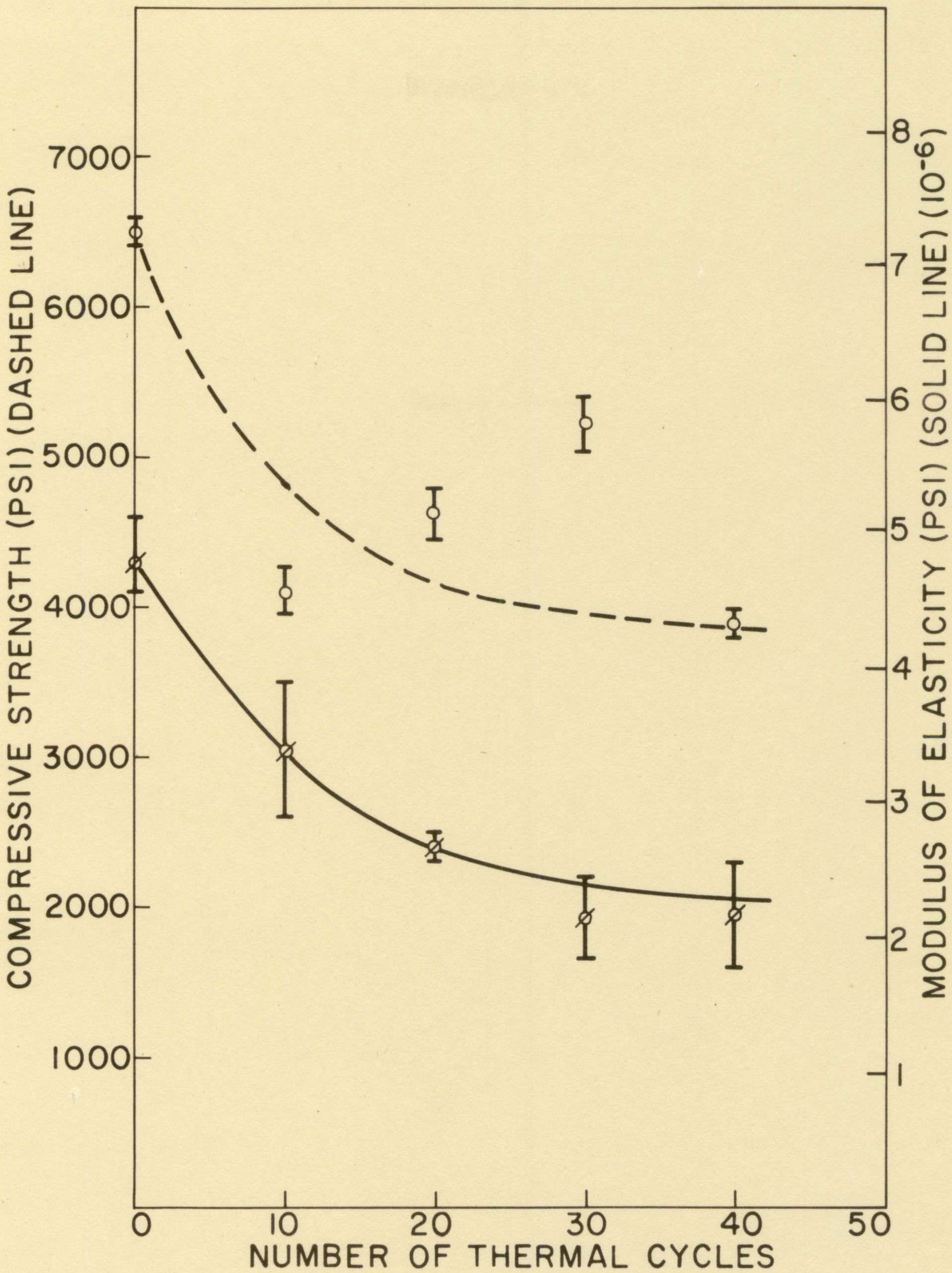


Figure 14. Variation of compressive strength and compressive modulus of elasticity with thermal cycling at 500 degrees F (cylinders tested thirty-one days after casting)

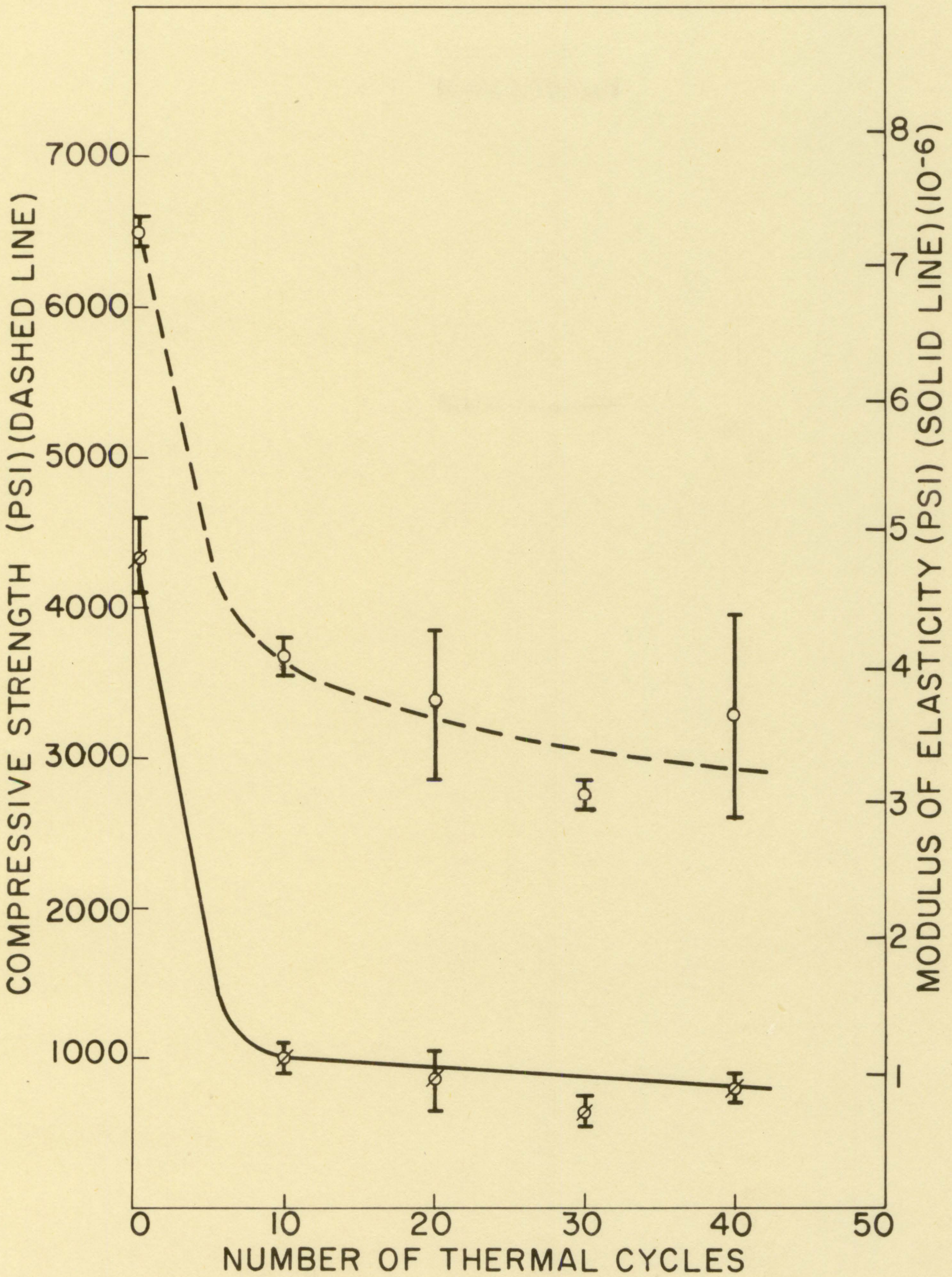


Figure 15. Variation of compressive strength and compressive modulus of elasticity with thermal cycling at 750 degrees F (cylinders tested thirty-one days after casting)

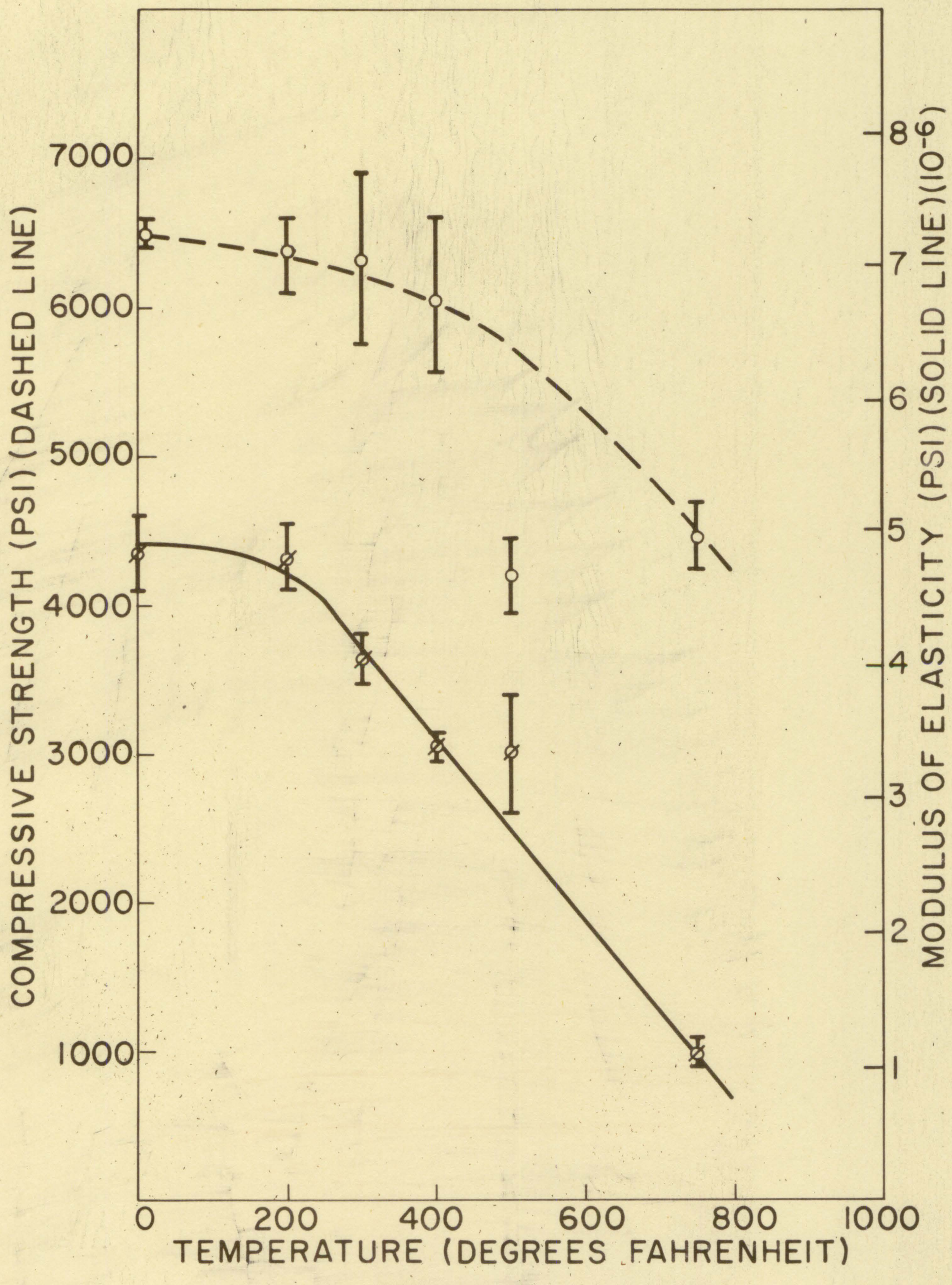


Figure 16. Variation of compressive strength and compressive modulus of elasticity with temperature at ten thermal cycles (cylinders tested at thirty-one days after casting)

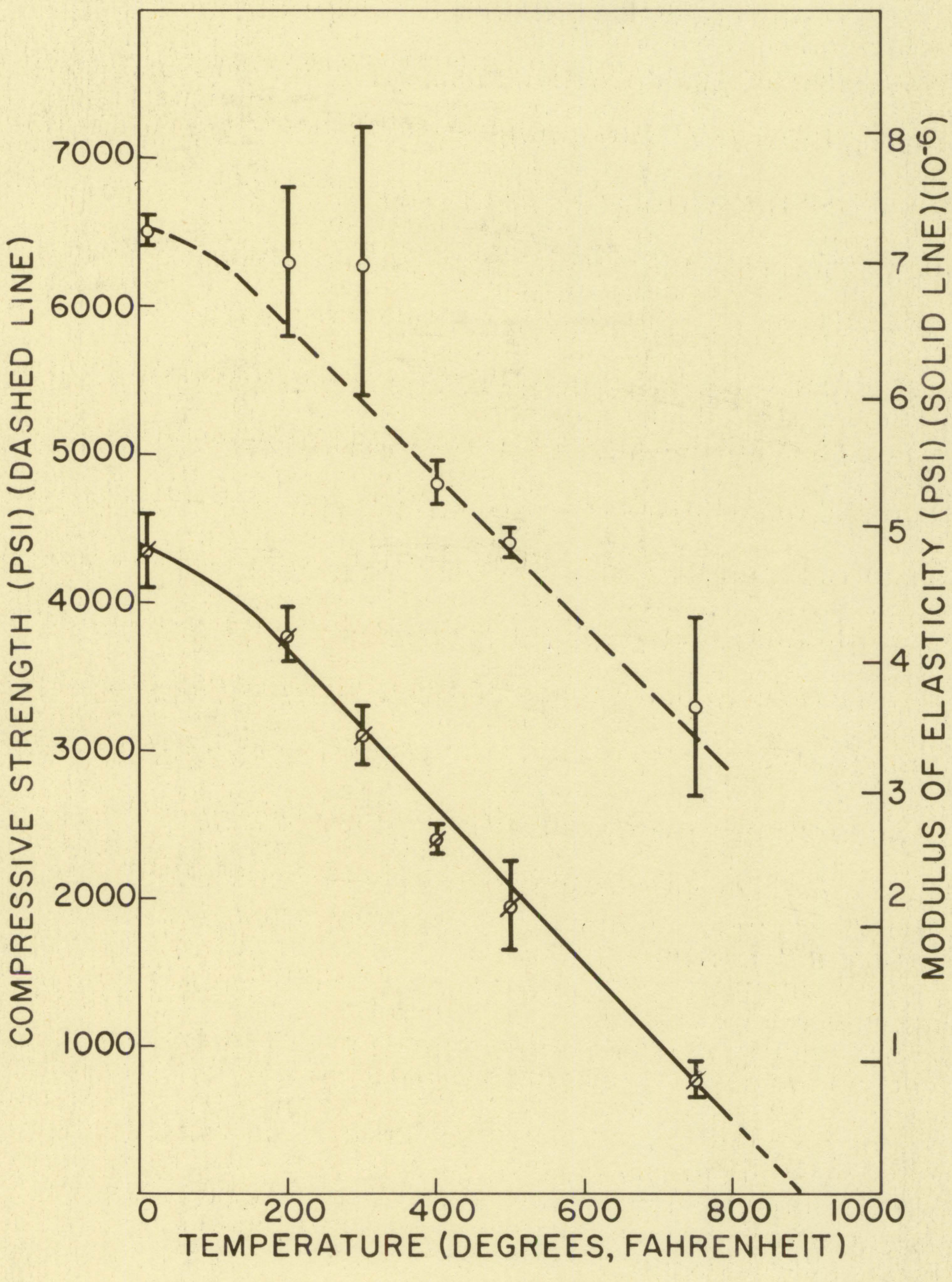


Figure 17. Variation of compressive strength and compressive modulus of elasticity with temperature at forty thermal cycles (cylinders tested at thirty-one days after casting)

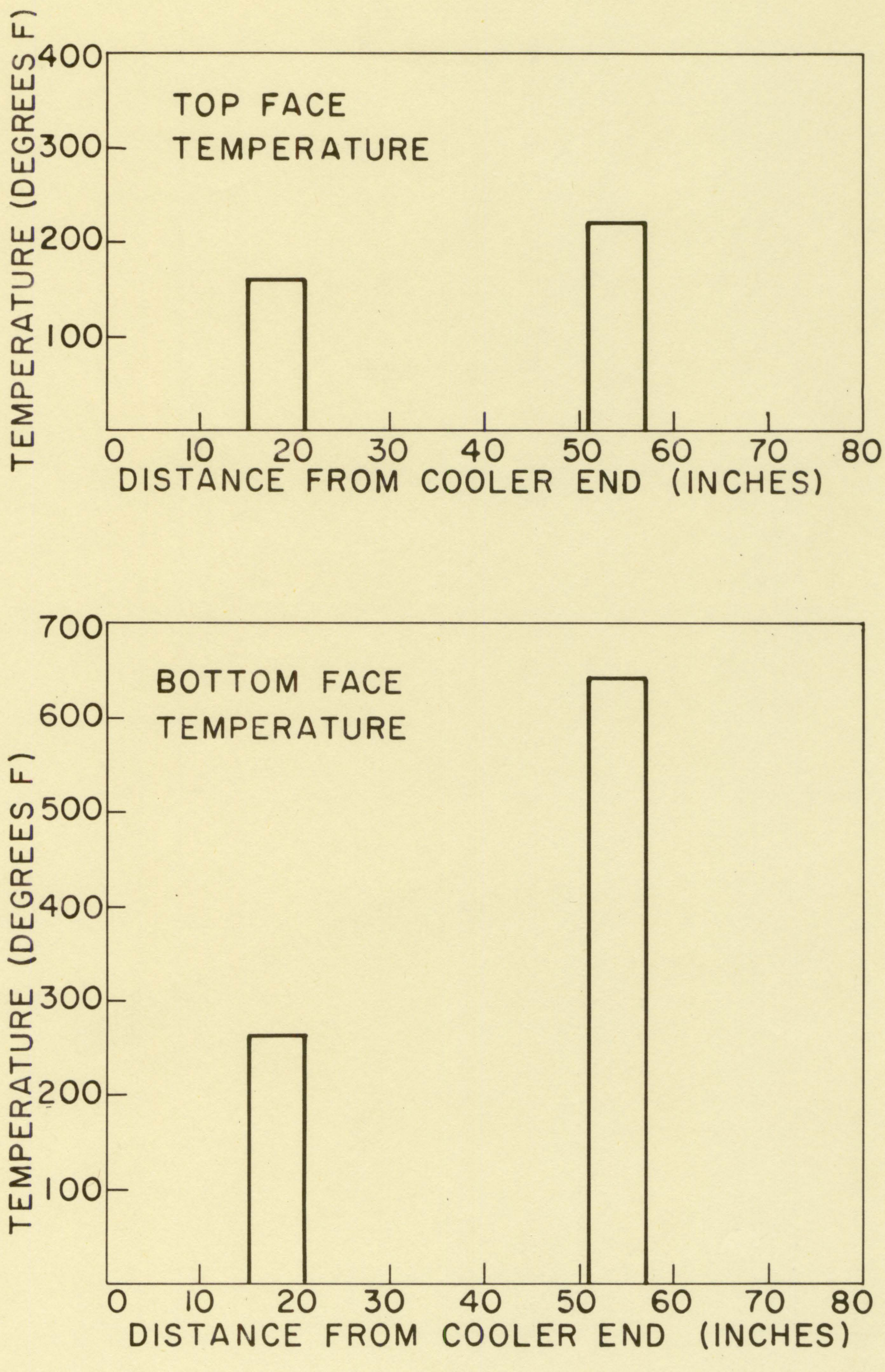
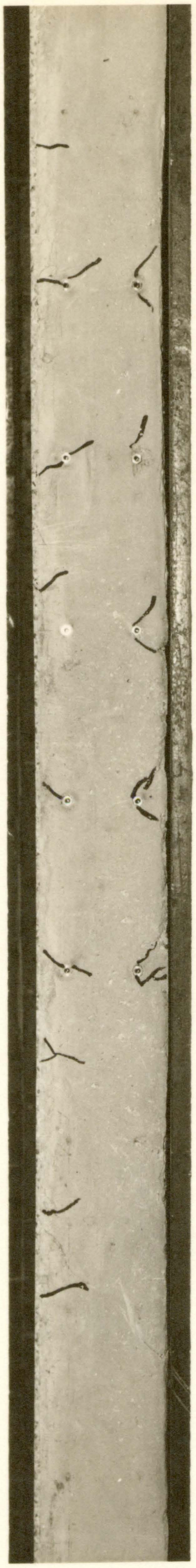
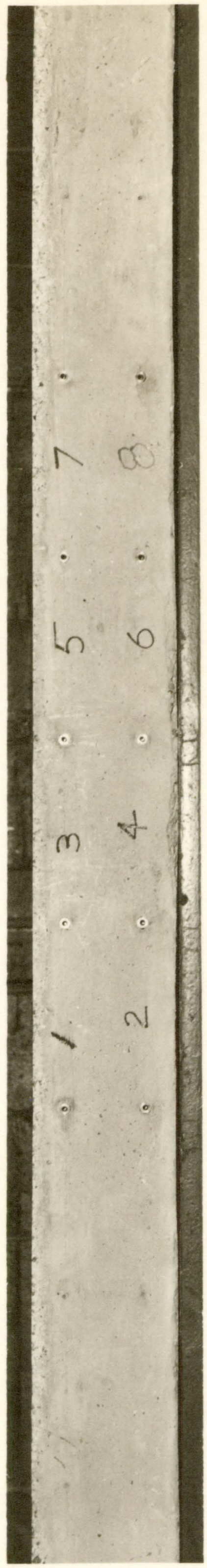


Figure 18. Maximum temperature at centers of top and bottom beam faces after twelve hours of continuous heating at maximum heating element capacity

Figure 19. Prestressed beam appearance after five thermal cycles

Figure 20. Reinforced beam appearance after five thermal cycles

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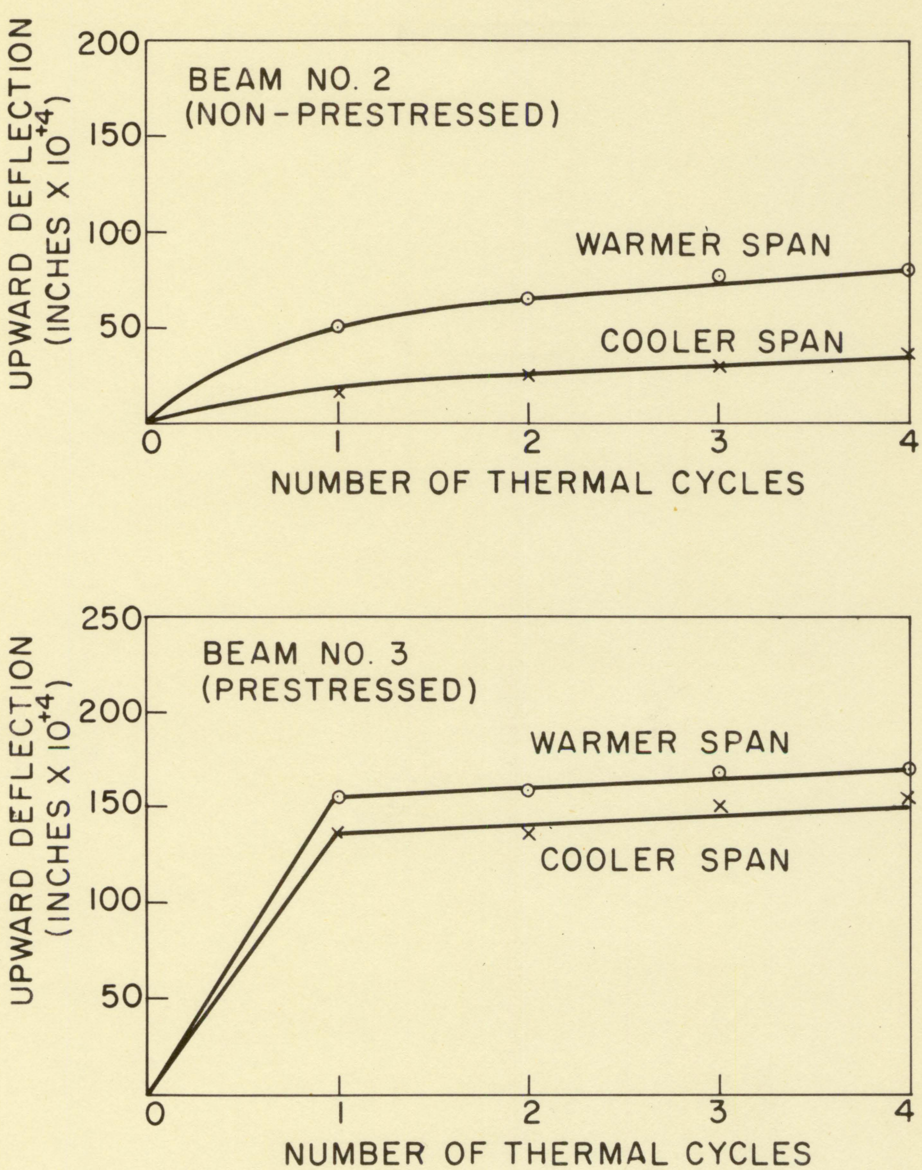

Figure 21. Inelastic beam deflections resulting from thermal cycling

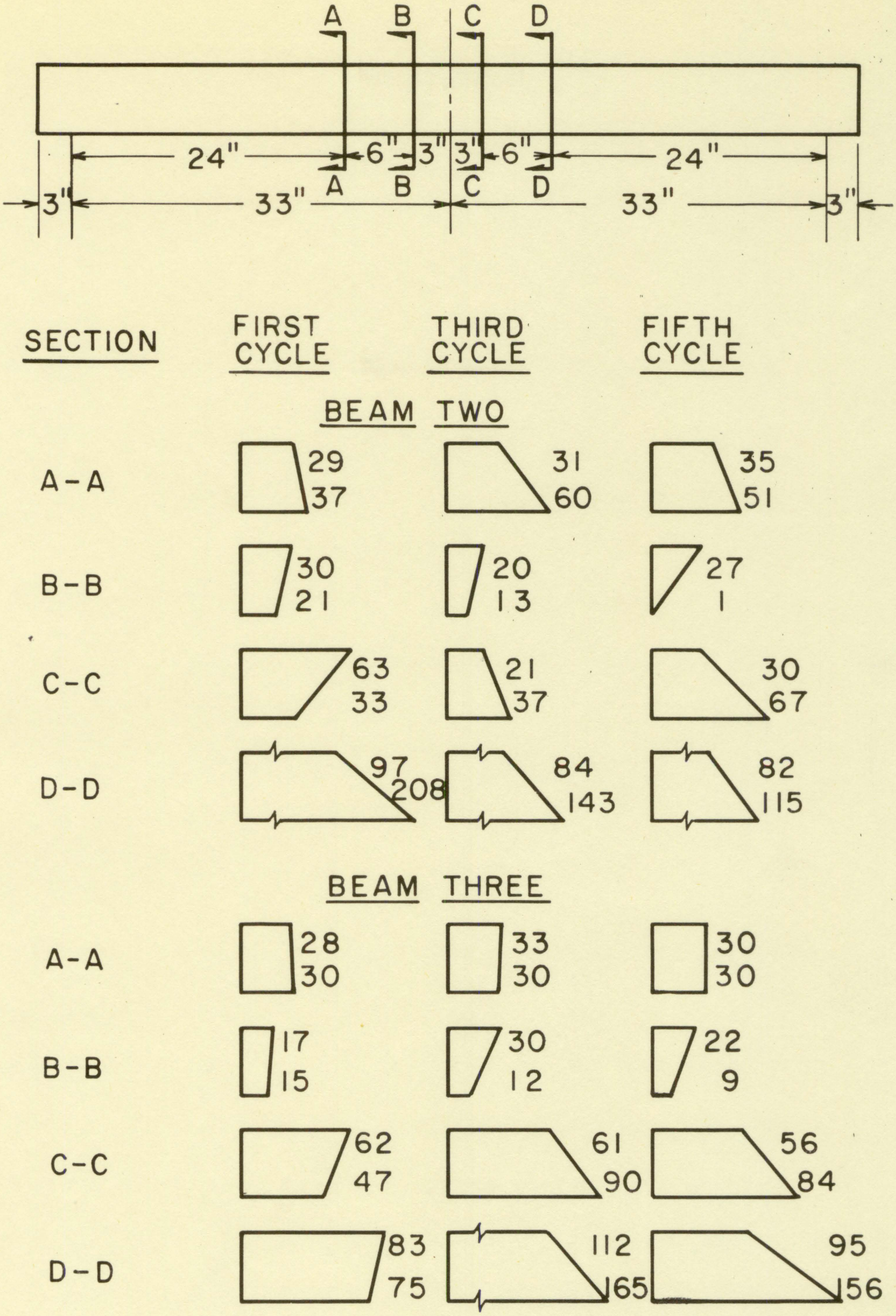

Figure 22. Maximum strain distribution at sections in beams two and three due to thermal cycles (all values shown are tensile strain values for top and bottom of beam in inches times 10^{+4})

SUMMARY AND CONCLUSIONS

Summary

This thesis describes the investigation of the effect of heat and thermal cycling on reactor concrete and related reinforcing systems. Forty-five cylinders of high strength concrete were fabricated, subjected to varying degrees of heat and thermal cycling and tested for deterioration.

Three test beams were also fabricated of high strength concrete. Two of these beams were prestressed and a third was reinforced. These beams, under suitable conditions of end restraint, were subjected to heat and thermal cycling and visually examined for deterioration. The strains and deflections caused in the beams by heating were also measured.

Conclusions

Following are the conclusions from this investigation. Without further study extending their range of validity, these conclusions can only be considered valid for the type of concrete used and over the range of temperatures covered by this investigation.

1. Thermal cycling, after a limited number of cycles, does not result in progressive, continuing deterioration in concrete. The loss of strength due to thermal cycling occurs in a relatively small number

of cycles and may, in fact, not be due to thermal cycling at all.

2. The higher the temperature, the faster final deterioration is developed in concrete.
3. Prestressed concrete appears to be somewhat more effective in resisting deterioration due to thermal stresses and thermal cycling than reinforced concrete.
4. Inelastic deformation created through thermal stresses occurs primarily during the first few thermal cycles and in the direction of the thermal gradient where structural restraints similar to those used in this research are employed.
5. Reactor concrete subjected to 750 degree F temperatures and thermal cycling will incur fifty per cent reductions in compressive strength.
6. Reactor concretes subjected to 750 degree F temperatures and thermal cycling will incur eighty per cent reductions in modulus of elasticity.
7. Reactor concrete will tolerate 420 degree F temperature differentials in four and one-half inches without appreciable surface deterioration provided proper steel reinforcement is provided to resist thermal stresses created by structural restraints.
8. Reactor concretes will tolerate 750 degree F temperatures without appreciable surface deterioration pro-

vided proper steel reinforcement is provided to resist thermal stressed created by structural restraints.

9. The compressive modulus of elasticity is a more reliable criterion for deterioration in concrete than is the ultimate compressive strength.

10. There is a strong indication that concrete, after initial heating to high temperatures, does not act elastically and structural methods of analyses and design based on elastic behavior are not valid.

SUGGESTIONS FOR FURTHER STUDY

One major problem left uninvestigated by this paper was the effect of duration of heating on deterioration of concrete. Where there was a decrease in strength with thermal cycling it was attributed to the cycling itself where in reality it might have been due alone to the duration of heating involved. It would be interesting to have the tests rerun for the same duration but without thermal cycling and compare the results obtained in such a manner with those presented in this thesis.

As a second problem the work of this thesis needs to be extended to more thermal cycling at low temperatures and to higher temperatures. Indications from results obtained are that thermal cycling may have a slow progressive deteriorating effect at lower temperatures. Indications are also that in the region of 900 to 1,300 degrees F concrete will have reached its limit of usefulness.

Finally, some work using the sonic modulus as a criterion of deterioration may have some merit and it also may be interesting to compare the usefulness of prestressed concrete and reinforced concrete at much higher temperatures.

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