Title

Maximum Temperature Stresses in Dworshak Dam

Authors

Raphael, Jerome
Clough, Ray
Wilson, Edward

Publication Date

1967-07-01

JMRaphael

REPORT NO.
67-14

STRUCTURES AND MATERIALS RESEARCH
DEPARTMENT OF CIVIL ENGINEERING

MAXIMUM TEMPERATURE STRESSES IN DWORSHAK DAM

By
JEROME M. RAPHAEL
EDWARD L. WILSON

REPORT TO
WALLA WALLA DISTRICT
U.S. ENGINEERS OFFICE

JULY, 1967

STRUCTURAL ENGINEERING LABORATORY
UNIVERSITY OF CALIFORNIA
BERKELEY CALIFORNIA

Structures and Materials Research
Department of Civil Engineering

Report No. 67-14

MAXIMUM TEMPERATURE STRESSES IN DWORSHAK DAM

A Report of an Investigation
by
Jerome M. Raphael, Professor of Civil Engineering
and
Edward L. Wilson, Assistant Professor of Civil Engineering

to

Walla Walla District
U. S. Engineers Office
Contract DA-45-164-Civ. Eng.-66-275

Structural Engineering Laboratory
University of California
Berkeley, California

July 1967

TABLE OF CONTENTS

LIST OF TABLES

LIST OF ILLUSTRATIONS

INTRODUCTION

Dworshak Dam, which is located in North Central Idaho on the North Fork of the Clearwater, is to be a straight gravity concrete structure with vertical upstream face. The dam will have an overall length of 3,287 feet and a maximum structural height of 693 feet. The total volume of concrete in the dam is approximately 6-1/2 million cubic yards, and the maximum volume of concrete to be handled in the placement of a single five-foot lift is almost 6,000 cubic yards. With volumes of this magnitude, great care has been taken in design and control of the concrete for maximum economy. The study reported here is the second of two to determine the effects of different construction schemes on stresses in the concrete during the construction of the dam.

A serious problem in the design and planning of massive concrete structures is the control of random structural cracking. The heat given off by hydrating cement in mass concrete is largely entrapped in the mass under nearly adiabatic conditions, with the result that the concrete temperature increases rapidly above the placing temperature. The degree of temperature rise depends upon the quantity of cement in the mixture, the coefficient of diffusivity of the aggregates, and the conditions of exposure of the mass. Very little difficulty is caused by temperature rise, which usually induces compressive stresses, but the subsequent decrease in temperature to a level corresponding to the ambient conditions causes contraction of the concrete and, if uncontrolled, in turn causes random structural cracking. Control of tensile cracking has been effectuated by control of the temperature drop in two ways: by control of the initial temperature, and by artificial cooling of the concrete mass. It is planned to use both of these methods in the construction of Dworshak Dam.

Without special treatment, the initial temperature of concrete is usually very close to the average ambient temperature at the time of mixing. The final temperature of the concrete mass is very nearly the average of the annual ambient air and water temperatures. The temperature rise of the concrete is the adiabatic temperature rise due to the hydration of cement, minus the losses due to exposure. Thus the maximum temperature is the simple sum of the initial temperature and the temperature rise. Since the concrete mixture and cement content are generally controlled by strength considerations, control of the maximum temperature can be affected to a large extent by sub-cooling the concrete materials before mixing. This in turn will decrease the maximum temperature and the temperature drop that causes tensile stress and cracking.

Another method of control of cracking through control of the temperature drop is by post-cooling. Post-cooling is a method of artificial cooling of the concrete mass after placement by the circulation of cold water through the mass in an embedded pipe system. This decreases the temperature rise and the total time required for cooling to equilibrium.

Both these methods themselves cause tensile stresses in regions of sharp temperature gradients. Pre-cooling causes development of significant tensile stresses when fresh concrete at low temperature is deposited on a layer of hardened and warmer concrete, since the surface of the hardened concrete is subjected to a general lowering of temperature. Post-cooling of concrete through an embedded system of pipes causes tensile stresses by inducing sharp temperature gradients in the vicinity of the pipe. In general, the magnitude of these tensile stresses depends on the construction schedule and the properties of the concrete. The construction schedule dictates the height of the lift and the time interval of placement between successive lifts, and thereby affects the temperature history of each lift, and indeed the temperature history of each location in each lift. The constituents of the concrete mixture determine the time-dependent thermal, elastic, and creep properties of the mixture.

The finite element method of analysis has proved an effective means for evaluating the stresses in the concrete under various schemes for construction and for temperature control. In an earlier study, "Construction Stresses in Dworshak Dam," by Raphael and Clough (1965), average stresses throughout a number of lifts at the base of Dworshak Dam were determined for a number of temperature histories and construction programs. To carry out this study, an incremental procedure was devised to take into account the time-dependent geometry of the structure as well as the time-variant material properties and loads. An important conclusion of this study was that uniform maximum stresses were found over a large portion of the center of the base, with stresses diminishing towards the faces of the dam. While the main study was concerned with average stresses in a lift, a minor supplementary analysis was carried out to determine the variation of stresses within the first few lifts of the dam for several favorable thermal cooling schedules in order to evaluate a number of proposals for control of temperature

In a study reported previously, the rate of construction was such that successive lifts of concrete were placed at four or five day intervals. This represents conditions near the base of the dam during early stages of construction. However, at later stages of construction, such a wide area of the dam will be available for depositing concrete, that placement of each new layer of concrete upon an old layer will be after relatively long delays of up to two weeks. An important effect of these longer placing intervals will be to accentuate the stress differentials within the individual lifts. Thus, dealing with the average stresses in the lifts can be inappropriate. However, it is still reasonable to assume that the maximum stresses will be developed in the lower central portion of the dam where the highest rate of temperature rise and the maximum restraint against deformation will occur.

The study reported here was undertaken in accordance with Contract No. DA-45-164-Civ Eng-66-275 between the Walla Walla District of the

U. S. Army Engineers and the Regents of the University of California, dated 31 January 1966. This study is concerned with determining in as much detail as possible the stresses throughout the central portion of the lowest lifts of the dam, in a vertical zone bounded by two adjacent lines of cooling pipes. New parameters introduced in this study include the rate of construction and the cement content of the mix. The contract covered two phases of investigation as follows:

Phase I

1. Evaluation of tensile creep properties. This included simulation of beam tests by finite element analysis to determine whether the tensile creep effect is adequately modeled by the computer program.

2. Determination of elastic and creep properties of 2.75 scy mass concrete by analysis and correlation of test data available for 3.25 scy concrete.

3. Modification of the computer program to account for essential changes in material properties.

Phase II

1. Placement of 45-degree concrete on rock, 2.75 sacks of cementitious materials per cubic yard, at 8-day placement intervals and post-cooled with 50-degree coolant.

2. Same as No. 1 except concrete contains 3.25 sacks of cementitious materials per cubic yard.

3. Placement of 45-degree concrete on old concrete after a winter shutdown, at 14-day intervals, and post-cooled with 50-degree coolant. Concrete contains 2.75 sacks of cementitious materials per cubic yard.

4. Same as No. 3 except placement at 8-day intervals.

5. Placement of 45-degree concrete on rock, 2.75 sacks of cementitious materials per cubic yard, at 14-day placement intervals, and post-cooled with 50-degree coolant.

In performing these new analyses, basic data were derived from a number of sources. The temperature histories that form the basis of the analysis were supplied by the U. S. Army Engineers, Walla Walla District. These covered the ten lowest lifts on a network of six-inch vertical and horizontal spaces for a zone bounded by two lines of cooling pipes.

After completing the previous study (1), the question had arisen whether the elastic and creep properties of concrete should be assumed to be the same in tension and compression. As a part of the present study, the test results of deep beams were analyzed and reported separately (2). In this auxiliary study, no significant difference was found in the tensile and compressive properties, and therefore the present study assumes this invariance of concrete properties for the purpose of determination of stresses using the finite element technique.

The study reported previously had been computed using elastic and creep properties for concrete tested with a cement content of 3.25 sacks of cementitious materials per cubic yard of concrete. The study reported here has assumed a leaner mix containing 2.75 sacks of cementitious materials per cubic yard in determining the thermal histories. Elastic and creep properties for this leaner mix were not determined directly, but were synthesized from the previous test results using correlations based on the paste content of the mix determined in tests made previously at the Engineering Materials Laboratory of the University of California.

METHOD OF ANALYSIS

Two-Dimensional Problem

In the earlier investigation (1) it was established that the maximum stresses occur in the central portion of each monolith of the dam. In that study, the dam was treated as essentially a two-dimensional problem in stress analysis. It was assumed that the stresses did not vary in the direction of the longitudinal axis, and that neither normal nor shear stresses existed on cross-sections normal to this axis. This assumption was justified because of the presence of transverse contraction joints at a spacing relatively small in comparison with the dimensions of the dam. Fig. 1 shows the typical cross-section of Dworshak Dam analyzed in that investigation. Modifications to the cross-section made after that study, changing the upstream face to vertical, do not affect the above conclusion.

In the present investigation, for determination of stresses within the lowest few lifts, the same assumption of two-dimensional stress state has been made. The dam is post-cooled by circulation of cold water through a system of embedded piping. These pipes are placed at 5 feet spacings both vertically and horizontally, and are located at the bottom of each lift. On alternate lifts, the pipes are staggered with respect to the system at the previous lift. Fig. 2 illustrates the arrangement of cooling pipes at successive lifts. Considering symmetry of the structure about vertical sections through pipes, it was considered sufficient to examine only a 30-inch wide zone of the structure. Thus, the structure analyzed was a one-inch thick slice of concrete (assumption of two-dimensional stress state), thirty inches wide (from symmetry about pipe locations), and rising from the foundation up to a few lifts in height. The analysis was carried out in two phases:

1. For a temperature history (temperature history No. 5), which was expected to give the highest tensile stresses in the dam, an analysis was made to determine the complete stress distribution and the stress history for the first four lifts including 11 feet of the foundation. The analysis was limited to four lifts because of limited storage available on the computer.
2. Investigation of 1 above showed that maximum tensile stresses were to be expected only at contact planes of successive lifts. It was also found that to obtain stress history at any contact plane it was not necessary to carry out a detailed analysis of the entire structure. Sufficiently accurate stress history for any contact plane could be obtained by considering the structure 30 inches below to 30 inches above the joint.

Representative joint planes were examined in detail for their stress history for each of the five temperature histories.

For the purposes of the analysis, the structure was considered to be position-fixed vertically on its base and horizontally on its sides.

Fig. 3 shows the finite element representation of the structure analyzed in the first phase.

The Finite Element Method

The two-dimensional stress problem was analyzed using the finite element method. This method has been developed sufficiently to take into account incremental load application, incremental structure and time dependent material properties. Since the method has been reported fairly extensively in the literature (1, 4, 5, 6), no attempt will be made to include details in this report.

Basically the method consists in visualizing a given body as an assembly of discrete elements of appropriate shape, interconnected only at their corners or nodal points. Deformations in each element are limited to certain specified forms. Based on these assumed displacement functions, the force-displacement relationships for each element may then be determined. These relationships also depend on the individual geometry of the element and its material properties. Thus if the material properties are time dependent, so are the force-displacement relationships.

For the present investigation, triangular elements having linearly varying displacement patterns were used. This assumption of linear displacement function implies a state of uniform strain and hence, for a linear stress-strain law, of uniform stress within each element. This assumption was necessary in order to use existing computer programs without extensive modifications. It is recognized, however, that a more precise picture of stress-distribution can be realized using a computer program having higher order elements and including the time varying structural geometry, loads and material properties.

For the solution of the problem, the direct stiffness approach has been used. By this procedure, the stiffness of the complete assemblage is obtained by adding together the stiffnesses of all the elements contributing to each nodal point. The equations of equilibrium are then solved simultaneously to determine the joint displacements resulting from the given loads. Finally, the stresses in the individual elements are found from nodal displacements, using the element stiffness properties.

Selection of Time Intervals for Computation

The most important way in which the computation of stress in a growing concrete structure differs from that in the usual structure is the time dependence of loads and material properties. The non-linear nature of these makes short time intervals very attractive, but short time intervals greatly lengthen the computation process. Hence two separate studies were directed toward finding the optimum time intervals for the application of loads and for allowing for the effect of creep in the concrete.

The various temperature histories studied in the investigation were based on 8-day and 14-day placement schedules. Also, cooling commenced simultaneously with the placement of a new lift. Thus these stages had necessarily to be represented in the incremental procedure. However, the temperature rise is non-linear, and the elastic modulus varies with age. The rate of temperature rise and the elastic modulus vary most rapidly at early age. Therefore, it would be inappropriate to apply the temperature load increment at large intervals of time.

The temperature data were available at one-day intervals. Ideally, a one-day interval for application of loads and calculation of material properties would appear to be the best. It is not necessary to use intervals shorter than one day because the approximation involved in any interpolation of temperature data offsets the advantage gained by use of a shorter time interval in the analysis. A larger interval of two days was tried. It was found that the results of the analysis were practically identical with those for the one-day time interval throughout, and also for one-day intervals at early age increased to two-day intervals later on. It was, therefore, decided to use a one-day time interval just after placement of each new lift and to extend the interval to two days at later stages.

Studies of stress relaxation due to creep were carried out using 1-day time intervals for feedback of unbalanced load as well as using the series of smaller intervals -- 1/8 day, 1/8 day, 1/4 day, 1/2 day. The difference in stress levels was found to be within about 5 percent, with the longer time intervals giving less stress-relaxation. Shortening time intervals to improve accuracy in this respect was considered unnecessary. Fig. 4 shows the comparison of stress relaxation curves for 3.25 scy concrete loaded at 1 day age for the long and the short time increments. For the purpose of calculating stress changes due to relaxation under constant strain, a time interval of 1/8 day had been found appropriate in the investigation reported previously (1), and the same interval was retained in the present study.

Biaxial Creep

For extension of uniaxial creep behavior to the biaxial case, it was assumed that the uniaxial characteristics can be applied to each principal stress component independently. As the principal stress directions may change somewhat as the nodal point constraints are relaxed at the beginning of each time increment, the creep constants have to be modified to allow for this rotation of the axes. To achieve this, the strain rate parameters are assumed to have the same properties as strains and are transformed to new axes by the conventional method used for strain transformation. Influence of Poisson's ratio in biaxial stress-relaxation under complete restraint is also included in the analysis.

Computer Program

A computer program for two-dimensional analysis allowing for incremental construction, creep, time-dependent elastic modulus, and foundation flexibility had been evolved for the previous study on Dworshak Dam (1) and was available. Modifications were needed to incorporate arbitrary boundary conditions for the structure--in the present case of lateral restraint on vertical sides of the 30-inch slice and vertical restraint on the base. Supplementary programs were developed to calculate temperature changes in elements from the nodal point temperatures furnished for each of the 5 histories by the Corps of Engineers, and to compile the creep data for the 2.75 scy concrete from the information available for the 3.25 scy concrete. Mesh generation programs already developed for the previous study were available and were adapted for the present investigation.

PROPERTIES OF DWORSHAK DAM

Concrete Properties

In connection with the earlier investigations for construction stresses in Dworshak Dam (1), the non-linear visco-elastic creep phenomenon in concrete was examined in some detail. It was found that the total strain represented by the equation:

$$\epsilon = \frac{\sigma}{E(t)} + \epsilon_c (\sigma, t, T)$$

where ϵ_c is defined as linearly dependent on σ, the stress level, by McHenry's equation:

$$\epsilon_c (\sigma, t, T) = \sigma \sum_{i=1}^{N} a_i(T) \left\{ 1-e^{-m_i(t-T)} \right\}$$

gave an adequate representation of the phenomenon. It was also found that N=2 gave satisfactory agreement with available experimental data. In these equations t is the time variable for definition of strain and T is the age at the time of loading. The time dependent modulus of elasticity is symbolized by $E_{(t)}$.

Fig. 5 shows schematically the typical creep test results. It will be noted that the elastic strain decreases continuously as the modulus of elasticity increases. The creep strain ϵ_c, therefore, defined in the manner above, represents not only the increase in total strain over initial strain, but also includes the reduction in elastic strain which takes place with time.

Using the numerical analysis procedures described by King (4) the constants a_1, a_2, m_1, m_2 were determined for the concrete containing 3.25 sacks of cementitious material per cubic yard. In the previous study (1) these coefficients were computed to fit experimental data obtained in tests carried out by Pirtz (7). It had been found that for fit to experimental data, m_1, m_2 could be taken as constants for the concrete considered and a_1, a_2 were then computed as functions of T, the age at loading.

In order to obtain the creep characteristics for 2.75 scy concrete, the linear correlation between creep strain and paste content of concrete reported by Polivka and others (8) was utilized. In this correlation, the creep strain is defined as the strain increase observed in excess of the initial straining. Thus the creep coefficients for the 3.25 scy concrete as computed in the earlier study (1) were used to determine the creep strain as defined by Polivka and others. These creep strains were scaled in accordance with the paste content of the 2.75 scy concrete and added to the initial elastic strain for this concrete to construct its strain-time history. This information was then processed to obtain a_1, a_2, m_1, m_2 giving least square fit for different age of loading of concrete. An estimate of ultimate strain was used as an additional parameter to obtain best fitting coefficients. It was found that m_1, m_2 could be assumed to have constant values

$$m_1 = .57$$

$$m_2 = .045$$

without seriously affecting the fit of the equations to the strain-time histories. Assuming $m_1 = .57$, and $m_2 = .045$, least square curve fitting gave values of a_1, a_2 shown in Fig. 6. Fig. 7 shows the synthesized creep strain curves, for 2.75 scy concrete, based on these coefficients. Fig. 8 shows the relaxation of stress due to creep.

The modulus of elasticity of 2.75 scy concrete (including 30% pozzolanic replacement) was found to be adequately represented by the following expression:

$$E_{(t)} = 800{,}000 + 663{,}000 \log t$$

where t is the time since casting in days. Poisson's ratio for this concrete was taken to be constant at 0.17.

The coefficient of thermal expansion of concrete was taken as 5.94×10^{-6} per degree Fahrenheit in accordance with tests, on wet screened samples of 3.25 scy concrete (including 30% pozzolanic replacement), carried out at the Waterways Experiment Station (9). It is assumed that this coefficient which depends mainly on the aggregates, is the same for the 2.75 scy concrete.

Foundation Properties

The foundation was assumed to have a modulus of elasticity of 4×10^6 psi and a Poisson's ratio of 0.17 (same as that of concrete). In cases where concrete is placed after a shutdown, the old hardened concrete constituting the "foundation" for the new lift was assigned these elastic constants.

Temperature Studies and Construction Schedule

Temperature changes in concrete may lead to undesirable stress distribution and tensile cracking. The cracking will be governed by the maximum tensile stresses developed and their relation to the tensile strength of concrete. As concrete has relatively lower tensile strength at early ages, it is important to control the development of tensile stresses. To achieve this, a combination of several measures is often adopted including control of placement temperature of concrete, post-cooling of concrete to limit temperature rise, planning of construction to allow each lift to lose heat to atmosphere and to develop strength before the next lift is placed, and use of special low heat mixes.

For this investigation, the Corps of Engineers developed the temperature control procedures. Five different temperature histories were computed representing varying placement intervals, mix design and environmental temperature conditions. Placement of concrete after a winter shutdown was also represented in these analyses. Studies 1, 2, and 5 represent placement of concrete on rock in warmer air temperatures whereas studies 3 and 4 relate to spring placement of concrete after a winter's shutdown. Study 2 envisaged use of 3.25 scy concrete as against 2.75 scy concrete in all other temperature studies. Placement is at 8-day intervals in studies 1, 2, and 4 whereas the interval is 14 days in studies 3 and 5. Table 1 shows the salient features of each temperature study. Figs. 9 and 10 show the time history of placement of the lowest lifts in the dam and the corresponding air temperatures for the various temperature studies.

Temperature Study 5 was selected for detailed analysis of stress distribution in the lowest four lifts. Temperature data were furnished by the Corps of Engineers on a 6-inch square network. Fig. 11 gives the temperature history at the horizontal construction joint between lift 1 and 2 for Temperature Study No. 5. This is typical of all joints.

It was seen as a result of this detailed analysis for Temperature Study 5 that the maximum horizontal stresses were associated with the sudden drop of temperature at the surface of a lift when the next lift was placed, and the vertical stresses were linked with the difference of temperature between the coldest point at an elevation and the mean temperature across that section. This again, attained maximum value at the joints between successive lifts where the cooling pipes were located. So, for all other studies, the maximum temperature drop and the greatest temperature differential associated with each lift joint have been listed in Tables 2 and 3.

RESULTS OF ANALYSIS

Stress Distribution Within Lowest Layers of Dam

In Phase II of the investigation, it was originally planned to study all the five temperature histories and to obtain the maximum tensile stresses set up during the placement of the lowest portions of the dam. The Corps of Engineers furnished data covering the ten lowest lifts for each of the five temperature histories. A comprehensive analysis of this type covering ten lifts would involve use of 1210 nodal points in order to utilize fully all the temperature data provided. Considering the extent of the analysis, special techniques and considerable computer effort would be required. It was therefore decided to investigate carefully the stress distribution and stress history in the first four lifts for a typical temperature history and to study the correlation between temperature history and stress history with respect to maximum tensile stresses. Temperature Study 5 included the maximum temperature gradients in both space and time plots and was, therefore, selected for this pilot analysis.

In order to utilize as much available temperature data as possible, the finite element mesh selected had nodal points at 6-inch vertical and horizontal spaces except in the foundation where a vertical spacing of 12 inches was adopted in order to conserve computer time. The finite elements used were right angled triangles. These elements are not very efficient in stress portrayal, but with the use of averaging techniques led to meaningful results. Effect of time interval had been studied in the course of the previous investigation (1) and was once again checked in preliminary analyses. It was found that for application of temperature load and for release of restraints imposed in the stress-relaxation process, an interval of two days would give sufficiently accurate results. The interval was reduced to one day just after a new lift was placed in order to allow for the rapid non-linear change in temperature and in material properties. Relaxation of stress at constant strain introduces unbalanced nodal forces which have to be released to restore equilibrium. In the computer program these are released simultaneously with the application of the load increment. An analysis was carried out to examine if the interval of time selected for load application was also acceptable for this release of nodal constraints. Fig. 4 shows the stress history for the time sequence selected in comparison with a very refined time sequence using one-eight day, one-eighth day, one quarter day, and one-half day as the initial four time increments. The agreement is quite satisfactory.

For relaxation of stress at constant strain and calculation of change in stress level, an interval of one-eighth day had been established in the previous investigation (1). It is important to make this interval sufficiently small as linearization of the stress relaxation function is possible only if short time intervals are used.

Early Age Properties of Concrete

In gravity dams, lean concrete mixes are generally used. In Dworshak Dam, the concrete will have an equivalent cement content as low as 2.75 scy.

It has been observed that such concretes have very low strength and elasticity at very early ages. At the same time, appreciable heat is given off by the cement before the matrix develops enough strength for the concrete mass to act as an elastic solid. This initial behavior can be approximated either by assigning a very low initial modulus to concrete or by delaying the time when the concrete is assumed to set. In the analysis, temperature change during any interval of time is applied to the structure at the end of the time increment. In the present investigation, as in the previous study (1), definite properties have been assigned to concrete at the end of the first day, and therefore the temperature rise during the first day after placement of a lift has in effect been disregarded.

Discussion of Results of the Pilot Analysis

Fig. 12 shows the plot of average horizontal stress against elevation at various times. Studying these graphs together with the temperature history plots of Fig. 10, leads to the following observations:

A. Horizontal Stresses

When a concrete lift is placed at low initial temperature in high atmospheric temperatures, the surface temperature rises rapidly. This leads to development of compressive stresses throughout the lift. However, with passage of time, the stresses at the surface reduce due to creep and stresses in the middle of the lift continue to rise along with the temperature.

Because of influence of creep, the rate of rise of stress is significantly less than the rate of increase of temperature. Thus at the end of 14 days, the lift has some residual compressive stresses throughout its volume. These stresses have maximum value in the middle of the lift and are least at the cooling pipe location and at the top surface.

When the next lift is placed, the top surface of the lower lift experiences a rapid fall in temperature. At this stage the concrete is somewhat hardened and has a higher modulus of elasticity than at earlier ages. This leads to development of relatively high tensile stresses in the top portion of the lift. Extreme stresses are seen to occur at the cooling pipe location, where two cooling effects combine: one due to low placement temperature of the upper lift, and a second due to cooling by circulation of coolant through the pipe. However, the tensile stresses diminish rapidly with depth into the lift. At the same time that the top of the lift is experiencing development of tensile stresses, the cooling cycle also being 14 days, the pipe at the bottom of the lift becomes inoperative in checking the temperature rise due to hydration. This leads to an increase in compressive stresses near the base of the lift.

With creep and general increase of temperature due to contact with the rapidly hydrating new lift of concrete, the peak tension falls off rapidly at first and then more gradually. As the rate of heat generation in the new lift decreases, the temperature drops and the tensile stress shows a tendency to increase. However, at 14 days after the new lift was placed, the cooling cycle is cut off leading to immediate drop in tensile stresses.

By this time, the concrete is 28 days old and there is no artificial cooling by pipes in it nor is the temperature rise due to hydration of much significance. Also, the creep is less marked. This explains the relatively stress-free state of the concrete after 28 days.

Concrete is not a good conductor of heat and the influence of pipe cooling or temperature changes at the top surface of any lift is not appreciable on stresses at the bottom of the lift in the 14 days cycle of pipe cooling.

For each lift, it is noticed that the maximum horizontal stress occurs at the junction of this lift with the next one and occurs immediately after the new lift is placed. This is because it corresponds to the large and sudden drop in temperature, of the top surface of the lift, caused by contact with the cold fresh concrete and by the cooling water circulating in the pipe.

B. Vertical Stresses

The mean vertical stress across any cross-section must equal the dead load of concrete above that elevation. The tensile stresses in this direction are associated with the non-uniformity of stress distribution. This non-uniformity is, in turn, associated with the local cooling effect of pipes. As a new lift is placed, the concrete temperatures are low and uniform. As the cement hydrates, its temperature rises, but this rise is prevented at the pipe location. This causes non-uniformity in temperature and consequently induces tensile stresses in the vertical direction. The temperature gradient increases with time and reaches a maximum some days after placement of the lift. The stresses are affected by creep and, therefore, attain maximum values some time ahead of the maximum temperature differential.

The temperature differential applies both to the new and the old concrete at a joint. The lower lift being older has a higher modulus of elasticity and less creep. Thus it will, in general, develop higher tensile stresses. However, it is noticed that the maximum tensile stresses in the vertical direction are not excessive. For Temperature History No. 5, these are very much lower than the maximum horizontal tensile stresses. Fig. 13 shows the history of vertical stresses at the location of the cooling pipe for the joint between lift 2 and lift 3. This is typical of all joints in this study.

C. Stress Distribution Within a Lift

Fig. 14 shows the stress distribution in lift 2 at the time of occurrence of maximum tensile stress. It was noticed in the analysis that for all practical purposes and for practically all zones of the lift, the horizontal and the vertical directions can be regarded as the principal directions. The figure shows the horizontal stresses, the vertical stresses and a combined sketch showing the maximum principal stress contours. As would be expected, the maximum tensile stress occurs at the location of the cooling pipe. In the upper part of the lift, horizontal stresses constitute the major principal stresses. In the lower portion, at the stage of occurrence of maximum tensions near the top, the cooling has already been discontinued. This results in compressive horizontal stresses. Thus, the vertical stresses form the major principal stress in a part of the lift, but as will be noticed, these are insignificant in magnitude.

Another analysis was carried out to study the distribution of stress in lift No. 2 in Temperature History No. 2 to obtain the distribution for cooling pipes active both at the top and bottom of a lift. Fig. 15 shows the stress-distribution at 9 days after placement of lift 2. In this case, the placement schedule is 8 days and cooling is going on both at top and bottom of the lift at the time maximum tensile stresses occur. However, horizontal stress is again the major principal stress throughout the upper part of the lift, whereas the vertical stress is the major stress in the lower part.

D. General Observations

An important observation emerged from the study. This was that the critical tensile stresses are to be expected only at the location of joints between lifts. Thus, if it is intended only to determine the maximum tensile stresses and not the stress distribution within a lift, attention can be focussed on the stress history of joints rather than that of complete lifts. This was expected to save considerably on computer time and also to make possible a study of joints of lifts higher than the fourth. It would be extremely wasteful of time and effort if the entire structure, from the foundation upwards, were to be analyzed.

An analysis was carried out for the joint between lifts 2 and 3 for Temperature History No. 5. Only 30 inches below and 30 inches above this joint were included in the analysis. The results were compared with those from the complete four-lift analysis. Perfect agreement was found to exist.

For all subsequent work, therefore, only a 100-element structure was analyzed for each of the representative lift joints. Selection of representative joints was based on the observations from the four-lift pilot analysis:

i. The maximum horizontal tensile stress is associated with the rapid temperature drop occurring in the surface of a lift when the next lift is placed on top of it.

ii. The maximum vertical tensile stress is associated with the maximum temperature differential across a joint. A measure of this temperature differential is the difference between the mean temperature at a joint and the minimum temperature across it.

Selection of Representative Joints

Tables 2 and 3 give significant temperature data, for Temperature Histories 1 to 5, for all lift joints. Based on this, joints were analyzed as indicated in the table below.

Temperature History No.	Horizontal Construction Joint Between Lifts No.
1	3, 4 7, 8 9, 10
2	2, 3 6, 7 9, 10
3	4, 5 7, 8
4	old concrete, 1 4, 5 8, 9 9, 10
5	2, 3 4, 5 6, 7

Stress Histories for Representative Joints

Figs. 16 to 20 show plots of stress histories for the representative joints for the five temperature studies.

It was observed that the vertical tensile stresses in all cases are relatively low (20 to 40 psi) and extremely localized. Keeping in mind the fact that the temperature calculation was based on the assumption that the concrete temperature is the same as the coolant temperature at the location of the cooling pipe, the actual stresses are likely to be significantly smaller in magnitude. This is because, for the heat transfer to take place, there has to be a considerable temperature differential between the concrete temperature on the exterior of the embedded pipe and the constant temperature. It appears rational, therefore, to ignore the vertical stresses as being insignificant.

Fairly high horizontal stresses are indicated in the region of the top surface of each lift immediately after the next lift is placed. This is due to the general cooling caused by the contact of the hardened warm concrete with fresh concrete at low placement temperature. Maximum local stresses occur, as would be expected, at the cooling pipe.

Table 4 summarizes the maximum local horizontal stresses at cooling pipe locations. Table 5 lists the maximum average horizontal stress at the elevation of the cooling pipe. The maximum stresses occurred immediately after placement of the next lift. However, in case of rock and old concrete (placed before winter's shutdown) the material was not expected to creep and the maximum stresses were reached towards the end of the cooling cycle.

Fig. 21 shows the relationship between maximum one-day temperature drop and the corresponding average horizontal stress at top of any lift for various temperature studies. Practically linear relationship was found to exist in all cases. For the same temperature drop, stresses developed in the surface of old concrete or rock will be much higher than those arising in 8 day or 14 day old concrete, due to the significant difference in the elastic moduli. Thus, data for stresses in old concrete and rock do not follow the linear relationship and have, therefore, been excluded from these plots.

CONCLUSIONS AND RECOMMENDATIONS

The parameters examined in this investigation included:

1. cement content
2. placement interval
3. (a) placement in spring after winter's shutdown
   (b) placement in summer with decreasing ambient temperatures
4. insulation of each lift or four lifts in spring placement after a winter's shutdown

Influence of Cement Content

As would be expected, the maximum value of tensile stresses using 2.75 scy concrete is somewhat lower than the corresponding figure for 3.25 scy concrete. Fig. 21 illustrates this by the line for Temperature Study No. 1 being lower and flatter than the line for Temperature Study No. 2. However, even with reduced cement content, significantly high tensile stresses will develop when atmospheric temperature is 75° F and placement is at 45° F. This is because this combination of atmospheric temperature and placement temperature will result in higher temperature drop at the top surface of any lift when the next lift is placed. Insulation of the surface at early stages to limit early rise of surface temperature through the gain of heat from the atmosphere would be expected to result in considerable reduction in the maximum stresses. In Studies No. 1, 2, and 5 the concrete temperature at the surface rises to the atmospheric temperature soon after placement while the concrete is still plastic and cannot develop any stresses. For this reason, when the next lift is placed, the temperature drop is applied to a practically stress-free surface resulting in severe tensions. If by early insulation (and possibly subsequent exposure) the temperature rise is delayed to a stage when concrete has developed significant elastic properties, a compressive stress can be induced in the concrete surface to neutralize, to some extent, the effect of the sudden temperature drop.

Influence of Placement Interval

Comparison of results for Study No. 1 and No. 5 shows clearly that the longer placement interval implies greater maximum tensions. This is because of the drop in temperature occurring at a time when the elastic modulus of concrete is higher. Thus a shorter placement interval for summer placement appears desirable. However, the comments in the preceding paragraph regarding delaying the temperature rise of surface of concrete apply regardless of the placement interval.

Spring Placement versus Summer Placement

When concrete is placed after a winter's shutdown, the old concrete is already hardened and cold. Placement of a fresh lift does not induce tensions because the old concrete is experiencing a temperature rise. In all the five temperature studies, summer placement is on rock. Very high tensile stresses in rock surface are to be expected though these may not be of much importance. Summer placement on hardened concrete, due to prolonged delays in placement of successive lifts, is always avoided as part of good construction planning and therefore needs no further comment.

Insulation of All or Four Lifts

In temperature Studies No. 3 and No. 4, insulation of concrete surface in cold weather is proposed. The effect of insulation is to prevent heat loss from the concrete surface so that there is a certain amount of compressive "prestress" when the temperature drop occurs on placement of the next lift. Again, as would be expected, 14-day placement implies more time for the prestress to reduce due to creep and a higher elastic modulus at the time of occurrence of the temperature drop, and for these reasons Temperature Study No. 3 has higher tensions than Study No. 4. Thus, a shorter placement interval is again desirable for spring placement. Value of insulation in cold weather is doubtful because if there is no insulation, the surface temperatures will correspond to air temperatures and the temperature drop will not be so sharp as it is for concrete temperatures rising almost adiabatically. In warmer periods, of course, insulation is useful for reasons discussed earlier.

Approximate Analysis Procedure

The examination of the results of these analyses indicates that the change in horizontal stress $\Delta\sigma$ at a point due to an instantaneous temperature change ΔT may be approximated by

$$\Delta\sigma = E\alpha\Delta T$$

where α is the thermal coefficient of expansion and E is the instantaneous modulus of elasticity. Of course, this is true only for this type of structure and because the concrete locally is in approximately a confined state. This incremental stress will relax with time, and its magnitude can easily be predicted. Tables 6 and 7 show stress histories for a one-degree (F) temperature drop applied at different ages for two different concrete mixes. These tables were computer generated and include the effects of the variation of modulus of elasticity with time.

With the aid of a table of this type, it is possible to predict readily the horizontal stresses as a function of time when the concrete is subjected to a known temperature history. In order to calculate the stress at any time, the effect of all the daily temperature changes must be added. Thus, if ΔT_1, ΔT_2, --- and ΔT_m are the daily temperature changes which occur up to m days and C_{1m}, C_{2m}, ----- and C_{mm} are the temperature stress influence coefficients for a concrete age of m days, then the total stress at age m is given by

$$\sigma_m = \Delta T_1 C_{1m} + \Delta T_2 C_{2m} + \text{-----} \; \Delta T_m C_{mm}$$

or

$$\sigma_m = \sum_{i=1}^{m} \Delta T_i C_{im}$$

Therefore, it is possible to calculate an approximate stress history for different placement and cooling schedules without the use of a complex digital computer program. Of course, this approximate technique may be applied only because the mass concrete is highly confined in the vicinity of the closely spaced cooling pipes.

The temperature history near the top of lift No. 2, Temperature Study No. 2, has been selected to illustrate the use of the approximate method. The temperature history for this case is shown in Fig. 22. The resulting approximate horizontal stresses versus time are shown in Fig. 23. For this case a typical calculation for the stress at age nine days is shown below:

Day	Temperature (°F)	Temp. Drop ΔT_i	C_{i9}	$\Delta\sigma_9 = \Delta T_i C_{i9}$
0	45			
1	70.5	-25.5	0	0
2	73.1	- 2.6	2.863	- 7.5
3	74.4	- 1.3	3.522	- 4.6
4	75.1	- 0.7	4.210	- 2.9
5	75.5	- 0.4	4.993	- 2.0
6	75.5	0	5.929	0
7	75.5	0	7.175	0
8	75.4	+ 0.1	9.320	+ 0.9
9	62.1	+13.3	14.458	+192.3

$$\sigma_9 = \sum_{i=1}^{9} \Delta T_i C_{i9} = +176.2$$

The approximate nine-day stress of 176 psi agrees very well with the 168 psi obtained by the finite element analysis. As indicated by

Fig. 23, comparison of the two methods is reasonable for the entire temperature history.

To illustrate the approximate method further, two hypothetical temperature histories were studied and are illustrated in Fig. 24. Temperature History A represents typical behavior of surface concrete which is placed and exposed to 75°F atmospheric condition. Temperature History B represents a concrete which is surface-insulated for the first 7 days; therefore, the temperature change is essentially adiabatic. Fig. 25 illustrates the resulting horizontal stresses. In this case, the superiority of Temperature History B is clearly illustrated. Approximately one hour of hand calculation was required for this comparison.

SUMMARY

The objective of this study was to find maximum stresses throughout a number of lifts of concrete under a variety of construction and cooling programs, using known properties of mass concrete for Dworshak Dam. This was accomplished on the computer using finite element techniques, incorporating functions for the creep of concrete, as well as its time-dependent elastic properties. When it became apparent that maximum stresses were generated at the top and bottom surfaces of the lifts, further studies were directed to these localities to find average stresses for a number of temperature conditions. Finally, an approximate method was devised by which further studies can be made rapidly of the effects of new temperature histories, omitting the digital computers, if desired, and using only desk calculators.

ACKNOWLEDGMENTS

The investigations reported here were performed at Berkeley under the terms of Contract DA-45-164-CIVENG-66-275 between the Walla Walla District of the U. S. Army Corps of Engineers and the Regents of the University of California. The U. S. Engineers at Walla Walla were responsible for physical data on the dam and temperatures computed by Civil Engineer James S. Gobble, under the general supervision of Chief Concrete Engineer Ivan E. Houk, Jr. The stress analyses were performed at Berkeley, under the general direction of the faculty investigators, Professors J. M. Raphael, R. W. Clough, and E. L. Wilson, by Junior Specialist R. S. Sandhu, assisted by Research Assistant S. Ghosh. Concrete properties were determined at the University under the direction of Professor David Pirtz and at the U. S. Engineers Concrete Laboratories at the Waterways Experiment Station, Jackson, Mississippi, and at the North Pacific Division Laboratory at Troutdale, Oregon.

LIST OF REFERENCES

1. "Construction Stresses in Dworshak Dam," by Jerome M. Raphael and Ray W. Clough. Final Report to Walla Walla District, U. S. Engineers Office. Report No. 65-3, Structural Engineering Laboratory, University of California, Berkeley, April 1965.

2. "Structural Behavior of Mass Concrete Beams," Report to Walla Walla District, U. S. Engineers Office. Report No. 67-16, Structural Engineering Laboratory, University of California, Berkeley, August 1967.

3. "Concrete Laboratory Studies, Dworshak (Bruce's Eddy) Dam, Idaho," Miscellaneous Paper No. 6-613, Report 1 and Report 3; U. S. Army Engineer Division Laboratory, North Pacific; U. S. Army W. E. S.; Vicksburg, Mississippi.

4. "Finite Element Analysis of Two-Dimensional Time Dependent Stress Problem," by Ian P. King, Ph. D. Dissertation, Department of Civil Engineering, University of California, Berkeley, January 1965.

5. "The Finite Element Method in Plane Stress Analysis," by Ray W. Clough, Proceedings, 2nd ASCE Conference on Electronic Computation, Pittsburgh, Pa., 1960.

6. "Finite Element Analysis of Two-Dimensional Structures," by Edward L. Wilson, University of California, Berkeley, Structural Engineering Laboratory, Report No. 63-2, June 1963.

7. "Creep Characteristics of Mass Concrete for Dworshak Dam," by David Pirtz, Structural Engineering Laboratory Report No. 65-2, University of California, 1965.

8. "Studies of Creep in Mass Concrete," by M. Polivka, D. Pirtz and R. F. Adams, Paper No. 12, Symposium on Mass Concrete, American Concrete Institute.

9. "Concrete Laboratory Studies, Dworshak (Bruce's Eddy) Dam, Idaho," Report 2; U. S. Army Engineer Division Laboratories, North Pacific, U. S. Army W. E. S.; Vicksburg, Miss.

TABLE 1 - BASIC DATA FOR TEMPERATURE STUDIES NO. 1 TO 5

Temperature Study No.	Cement Content Scy	Adiabatic Temperature Rise °F	Placement Interval Days	Concrete Placed On	Remarks
1	2.75	36.10	8	75°F rock	summer placement, no insulation
2	3.25	41.40	8	75°F rock	summer placement, no insulation
3	2.75	36.10	14	43°F concrete	spring placement, 4 lifts insulated
4	2.75	36.10	8	43°F concrete	spring placement, each lift insulated
5	2.75	36.10	14	75°F rock	summer placement, no insulation

Placement temperature of concrete = 45°F

Temperature of coolant = 50°F

Cooling period = 14 days immediately after placement

Diffusivity of concrete = .035 sq. ft./hour

Diffusivity of rock = .042 sq. ft./hour

Conductivity = 1.2 BTU/ft./hour/°F

Surface coefficient = 5.0 BTU/sq. ft./hour/°F

Mean annual temperature = 51°F

Lift thickness = 5 ft.

Pipe spacing = 5 ft. (staggered)

TABLE 2 - TEMPERATURE DROP IN ONE DAY AFTER PLACEMENT OF NEXT LIFT

Top of Lift No.	Temperature Study				
	1	2	3	4	5
Rock or Old Concrete	16.4	15.5	-6.1*	-6.1*	16.4
1	15.2	14.2	3.0	2.8	15.6
2	15.0	14.1	3.6	3.2	15.4
3	15.1	14.0	5.3	3.2	15.4
4	15.1	14.0	6.9	3.2	13.3
5	12.9	12.0	5.1	6.5	13.3
6	13.0	12.0	5.2	6.9	7.8
7	13.0	12.0	9.3	7.0	7.8
8	13.0	12.0		7.0	-0.7*
9	7.4	6.4		10.0	-2.0*

* Temperature rose on placement of new lift of concrete. Maximum drop in any one day was insignificant.

TABLE 3 - MAXIMUM DIFFERENTIAL BETWEEN MEAN AND MINIMUM TEMPERATURE

Joint of Lift No./Lift No.	Temperature Study				
	1	2	3	4	5
Rock or Old Concrete/1	10.3	10.8	4.0	4.0	10.3
1/2	12.9	13.6	7.4	8.2	12.2
2/3	13.2	14.0	8.0	8.9	12.6
3/4	13.3	14.1	8.7	9.0	12.4
4/5	13.1	14.0	9.0	9.3	12.1
5/6	12.8	13.6	9.4	9.8	11.6
6/7	12.8	13.6	9.5	10.0	10.5
7/8	12.8	13.6	10.5	10.0	9.6
8/9	12.3	13.3		10.2	8.8
9/10	11.4	12.2		10.7	

TABLE 4 - MAXIMUM LOCAL HORIZONTAL STRESSES AT COOLING PIPE - PSI

Top of Lift No.	Temperature Study				
	1	2	3	4	5
Rock or Old Concrete				-8	406
1	186	214	10	+5	238
2	186	212	24	+7	235
3	188	210	44	+7	235
4	188	210	85	+7	206
5	160	180	40	52	206
6	163	180	50	57	125
7	163	180	104	61	118
8	163	180		61	
9	93	94		90	

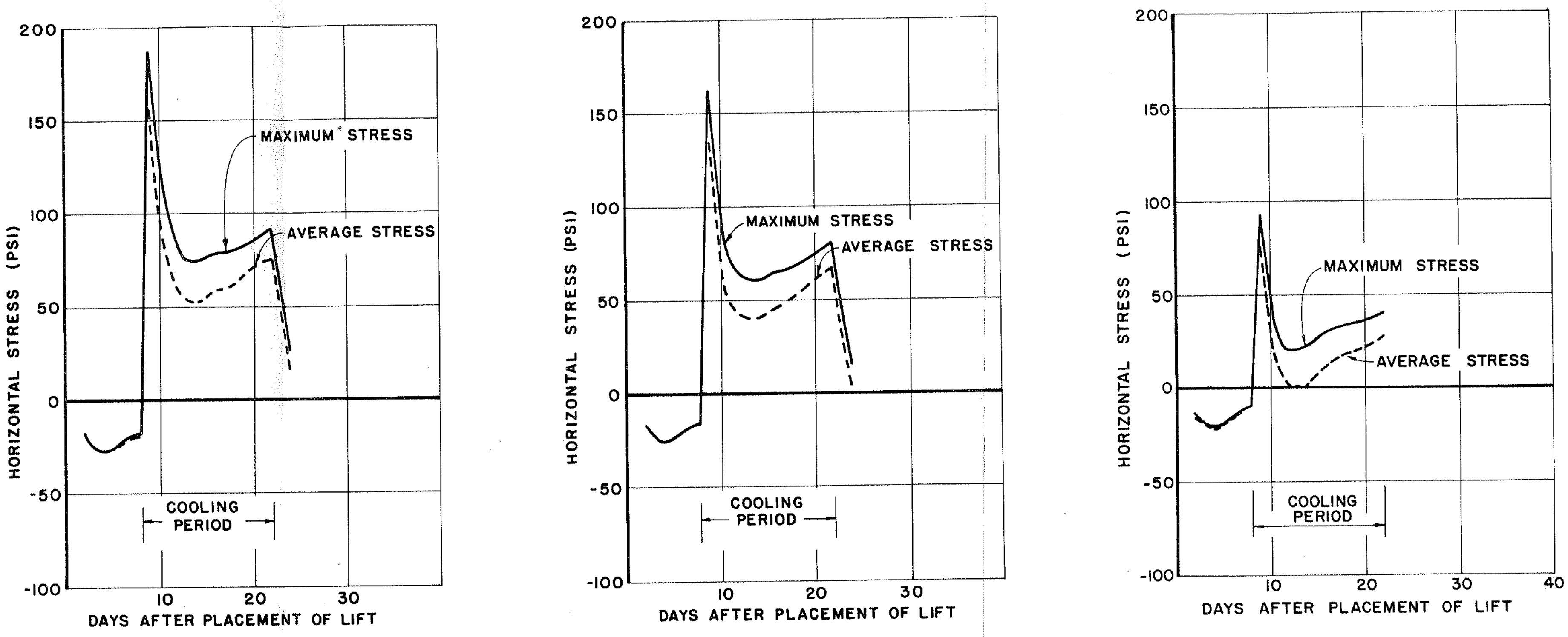
TABLE 5 - MAXIMUM AVERAGE HORIZONTAL STRESSES AT ELEVATION OF COOLING PIPE - PSI

Top of Lift No.	Temperature Study No.				
	1	2	3	4	5
Rock or Old Concrete				-12	383
1	158	170			205
2	158	168	10		203
3	158	168	28		203
4	158	168	66	-3	175
5	134	145	25	36	175
6	136	145	42	40	106
7	136	145	83	41	99
8	136	145		41	
9	75	72		80	

TABLE 6 - TEMPERATURE STRESS INFLUENCE COEFFICIENTS

(2.75 SCY CONCRETE)

TIME OF LOADING - ONE DEGREE F DECREASE	AGE OF CONCRETE (DAYS)																													
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
0- 1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1- 2		6.349	3.998	3.078	2.737	2.592	2.513	2.457	2.408	2.362	2.318	2.276	2.234	2.194	2.155	2.118	2.081	2.046	2.013	1.980	1.949	1.919	1.891	1.863	1.837	1.811	1.787	1.764	1.742	1.720
2- 3			8.361	5.262	4.157	3.731	3.534	3.415	3.324	3.244	3.168	3.096	3.026	2.958	2.892	2.829	2.767	2.708	2.652	2.597	2.544	2.494	2.446	2.399	2.355	2.312	2.271	2.232	2.194	2.158
3- 4				9.686	6.258	5.019	4.518	4.271	4.116	3.995	3.888	3.787	3.691	3.598	3.509	3.422	3.339	3.258	3.181	3.107	3.035	2.966	2.900	2.837	2.776	2.718	2.663	2.609	2.558	2.509
4- 5					10.676	7.150	5.825	5.264	4.975	4.787	4.639	4.507	4.385	4.268	4.155	4.047	3.942	3.841	3.744	3.650	3.561	3.475	3.392	3.313	3.236	3.163	3.093	3.026	2.962	2.901
5- 6						11.464	7.899	6.512	5.903	5.580	5.367	5.198	5.049	4.910	4.777	4.650	4.528	4.410	4.296	4.187	4.083	3.982	3.885	3.793	3.704	3.619	3.537	3.459	3.383	3.312
6- 7							12.118	8.564	7.136	6.490	6.142	5.910	5.726	5.563	5.412	5.268	5.130	4.998	4.870	4.748	4.630	4.517	4.408	4.304	4.204	4.108	4.016	3.928	3.844	3.763
7- 8								12.694	9.153	7.693	7.022	6.656	6.411	6.217	6.045	5.885	5.732	5.587	5.447	5.313	5.184	5.060	4.941	4.827	4.718	4.613	4.512	4.416	4.324	4.235
8- 9									13.270	9.669	8.167	7.473	7.094	6.841	6.640	6.462	6.297	6.141	5.991	5.848	5.710	5.578	5.451	5.330	5.213	5.101	4.994	4.891	4.792	4.698
9-10										13.846	10.152	8.604	7.889	7.500	7.240	7.034	6.853	6.686	6.527	6.375	6.230	6.091	5.957	5.828	5.705	5.587	5.474	5.366	5.262	5.162
10-11											14.422	10.613	9.016	8.280	7.880	7.614	7.404	7.220	7.050	6.889	6.735	6.588	6.447	6.311	6.181	6.057	5.938	5.823	5.714	5.609
11-12												14.998	11.043	9.388	8.626	8.213	7.941	7.727	7.539	7.366	7.203	7.047	6.897	6.754	6.617	6.485	6.359	6.238	6.122	6.012
12-13													15.575	11.484	9.773	8.988	8.565	8.288	8.070	7.881	7.706	7.540	7.383	7.232	7.088	6.949	6.817	6.690	6.568	6.451
13-14														16.151	11.938	10.178	9.373	8.944	8.663	8.445	8.254	8.078	7.913	7.755	7.604	7.459	7.320	7.188	7.060	6.938
14-15															16.727	12.370	10.556	9.732	9.295	9.010	8.789	8.597	8.420	8.253	8.094	7.942	7.796	7.657	7.523	7.395
15-16																17.303	12.775	10.898	10.050	9.602	9.311	9.086	8.891	8.711	8.542	8.380	8.226	8.078	7.937	7.801
16-17																	17.879	13.185	11.248	10.378	9.918	9.623	9.395	9.197	9.015	8.844	8.680	8.524	8.375	8.232
17-18																		18.456	13.603	11.608	10.715	10.249	9.950	9.719	9.520	9.337	9.164	9.000	8.843	8.693
18-19																			19.032	14.013	11.958	11.043	10.567	10.263	10.030	9.828	9.642	9.468	9.302	9.143
19-20																				19.608	14.414	12.295	11.355	10.867	10.557	10.318	10.112	9.923	9.745	9.576
20-21																					20.184	14.820	12.639	11.674	11.175	10.858	10.615	10.405	10.213	10.032
21-22																						20.760	15.231	12.990	12.001	11.492	11.169	10.992	10.709	10.515
22-23																							21.336	15.642	13.339	12.325	11.804	11.475	11.223	11.006
23-24																								21.913	16.052	13.685	12.644	12.109	11.771	11.512
24-25																									22.489	16.467	14.038	12.970	12.421	12.074
25-26																										23.065	16.889	14.398	13.303	12.740
26-27																											23.641	17.317	14.766	13.646
27-28																												24.217	17.753	15.148



a. TOP OF LIFT NO. 3

b. TOP OF LIFT NO. 7

c. TOP OF LIFT NO. 9

FIG. 16 STRESS HISTORY: TEMPERATURE STUDY NO. I

TABLE 7 - TEMPERATURE STRESS INFLUENCE COEFFICIENTS

(3.25 SCY CONCRETE)

		AGE OF CONCRETE (DAYS)																													
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
TIME OF LOADING - ONE DEGREE F DECREASE	0- 1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	1- 2		7.537	4.669	3.573	3.190	3.039	2.962	2.909	2.863	2.821	2.779	2.739	2.698	2.659	2.620	2.582	2.544	2.507	2.471	2.435	2.400	2.366	2.333	2.300	2.268	2.236	2.206	2.176	2.146	2.118
	2- 3			9.549	5.661	4.355	3.892	3.700	3.595	3.522	3.459	3.400	3.343	3.287	3.232	3.178	3.125	3.073	3.021	2.971	2.921	2.873	2.825	2.778	2.733	2.688	2.644	2.602	2.560	2.520	2.480
	3- 4				10.874	6.582	5.128	4.586	4.346	4.210	4.111	4.025	3.944	3.867	3.791	3.716	3.643	3.571	3.500	3.430	3.362	3.295	3.229	3.165	3.102	3.040	2.980	2.921	2.864	2.808	2.754
	4- 5					11.864	7.439	5.880	5.273	4.993	4.829	4.708	4.602	4.503	4.407	4.314	4.222	4.132	4.043	3.956	3.870	3.787	3.704	3.624	3.545	3.468	3.393	3.320	3.248	3.178	3.110
	5- 6						12.652	8.202	6.578	5.929	5.624	5.444	5.309	5.193	5.084	4.978	4.875	4.774	4.675	4.578	4.482	4.388	4.296	4.206	4.118	4.032	3.948	3.866	3.785	3.707	3.631
	6- 7							13.306	8.843	7.175	6.497	6.175	5.983	5.839	5.714	5.597	5.484	5.376	5.266	5.159	5.055	4.953	4.853	4.755	4.659	4.565	4.474	4.384	4.297	4.212	4.129
	7- 8								13.882	9.320	7.595	6.888	6.551	6.351	6.202	6.073	5.952	5.835	5.721	5.609	5.500	5.393	5.288	5.185	5.084	4.986	4.889	4.795	4.704	4.615	4.528
	8- 9									14.458	9.784	8.002	7.268	6.919	6.712	6.558	6.425	6.300	6.180	6.062	5.948	5.836	5.726	5.618	5.513	5.410	5.309	5.210	5.114	5.021	4.930
	9-10										15.034	10.227	8.385	7.627	7.266	7.053	6.894	6.757	6.629	6.505	6.385	6.268	6.153	6.041	5.931	5.823	5.718	5.616	5.515	5.418	5.323
	10-11											15.610	10.651	8.749	7.967	7.595	7.375	7.211	7.070	6.939	6.812	6.690	6.570	6.452	6.338	6.225	6.116	6.008	5.904	5.802	5.703
	11-12												16.186	11.051	9.083	8.273	7.888	7.660	7.491	7.346	7.211	7.081	6.956	6.833	6.713	6.596	6.481	6.369	6.260	6.153	6.050
	12-13													16.763	11.464	9.430	8.592	8.193	7.958	7.784	7.635	7.497	7.365	7.237	7.112	6.989	6.870	6.753	6.639	6.528	6.420
	13-14														17.339	11.889	9.793	8.927	8.514	8.272	8.094	7.942	7.801	7.667	7.536	7.409	7.285	7.163	7.045	6.930	6.818
	14-15															17.915	12.330	10.174	9.280	8.855	8.607	8.426	8.271	8.128	7.992	7.860	7.731	7.605	7.483	7.363	7.247
	15-16																18.491	12.787	10.576	9.660	9.226	8.973	8.789	8.633	8.489	8.352	8.219	8.089	7.963	7.840	7.721
	16-17																	19.067	13.225	10.957	10.017	9.573	9.314	9.127	8.969	8.823	8.683	8.549	8.418	8.291	8.167
	17-18																		19.644	13.638	11.305	10.337	9.879	9.613	9.421	9.258	9.109	8.966	8.829	8.696	8.566
	18-19																			20.220	14.061	11.665	10.669	10.198	9.925	9.728	9.561	9.409	9.264	9.124	8.989
	19-20																				20.796	14.496	12.039	11.017	10.534	10.254	10.052	9.883	9.728	9.582	9.440
	20-21																					21.372	14.930	12.412	11.363	10.865	10.577	10.370	10.197	10.039	9.890
	21-22																						21.948	15.363	12.781	11.701	11.187	10.889	10.676	10.498	10.335
	22-23																							22.524	15.807	13.163	12.053	11.523	11.216	10.997	10.813
	23-24																								23.101	16.263	13.561	12.424	11.880	11.565	11.340
	24-25																									23.677	16.704	13.942	12.776	12.217	11.893
	25-26																										24.253	17.126	14.297	13.099	12.523
	26-27																											24.829	17.557	14.662	13.431
	27-28																												25.405	17.997	15.037

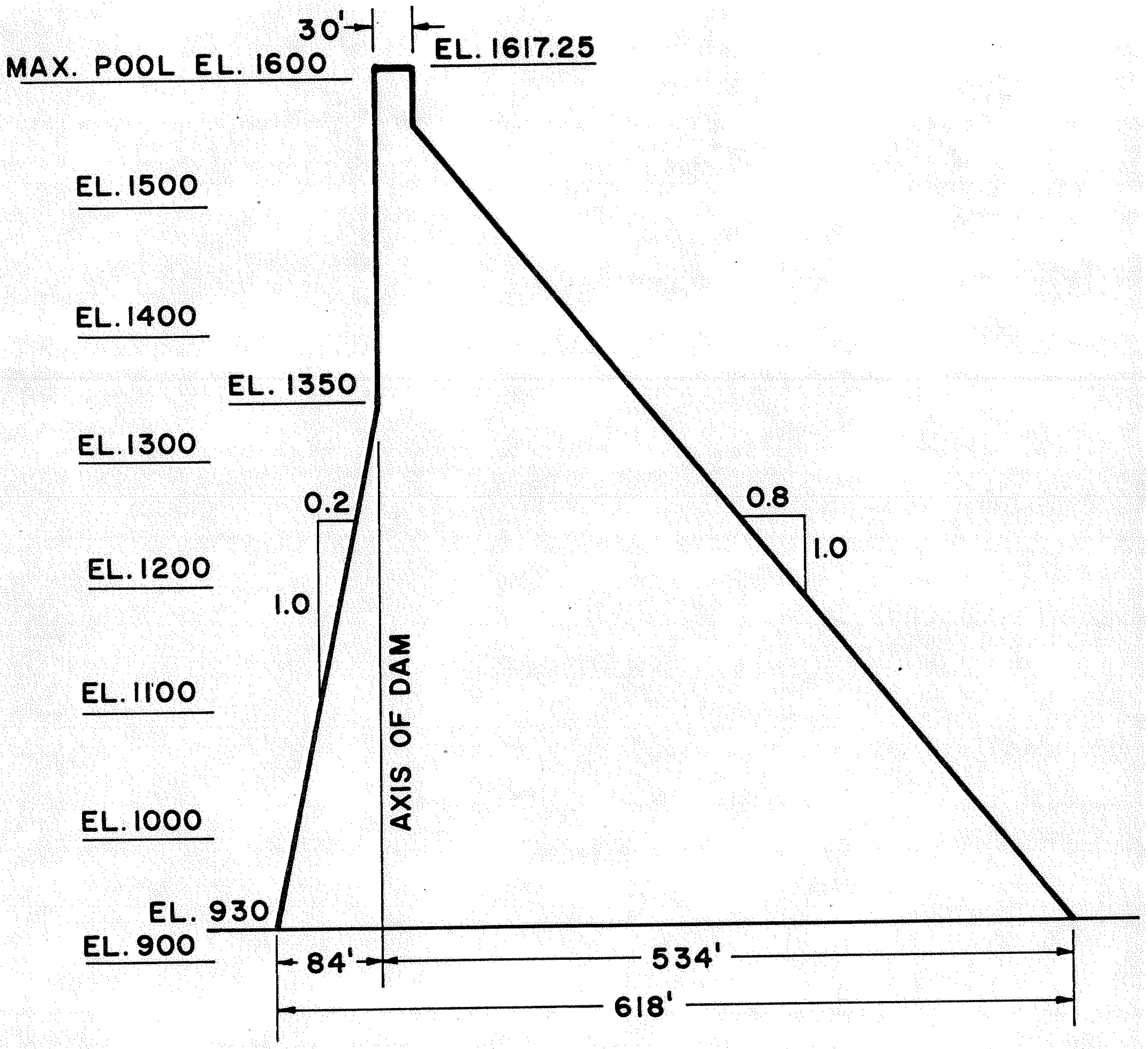


FIG. 1 - CROSS SECTION OF DWORSHAK DAM

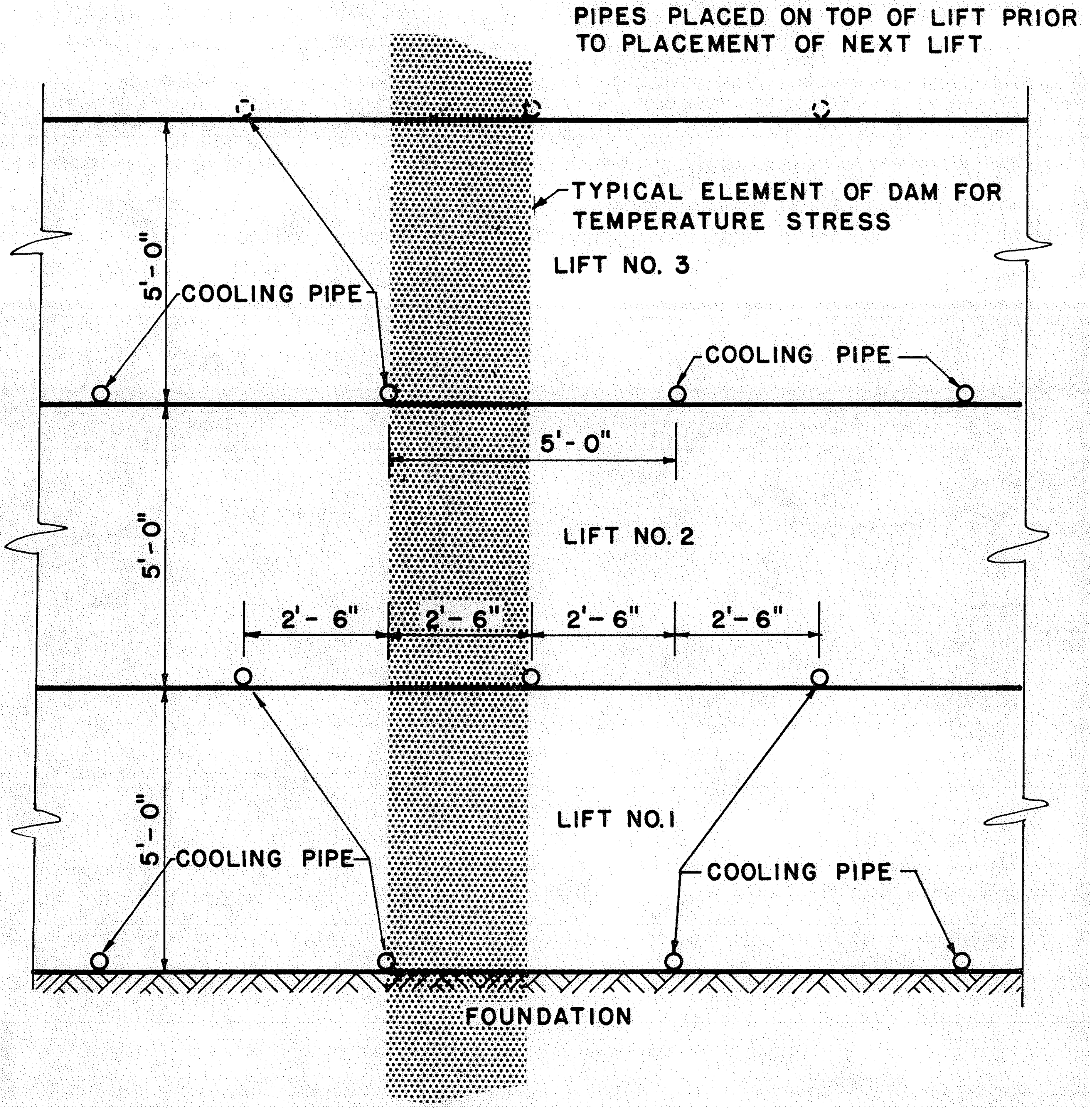
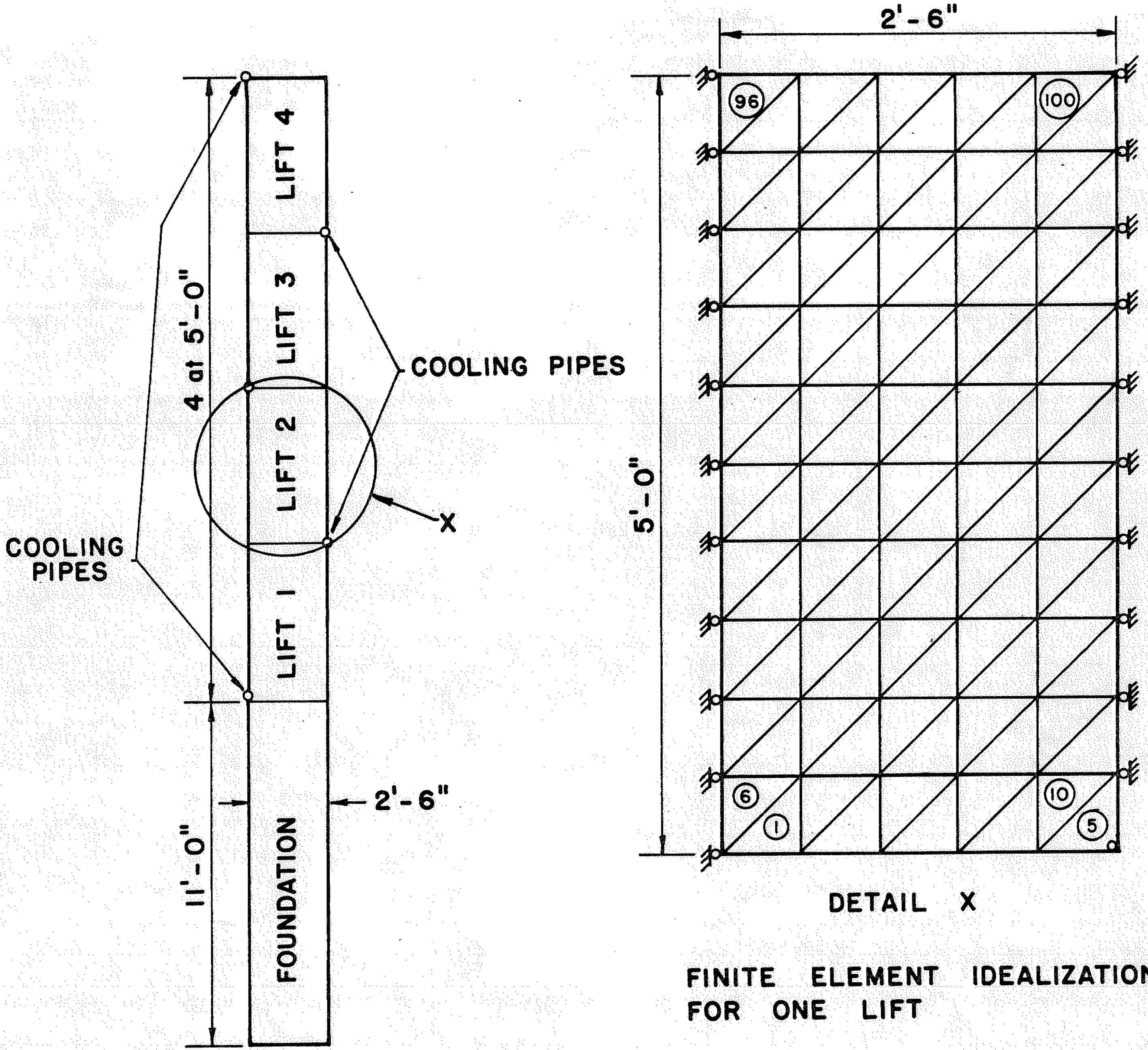


FIG. 2 - TYPICAL ARRANGEMENT OF COOLING PIPES



DETAIL X

FINITE ELEMENT IDEALIZATION FOR ONE LIFT

FIG. 3 - STRUCTURE ANALYZED IN PILOT ANALYSIS

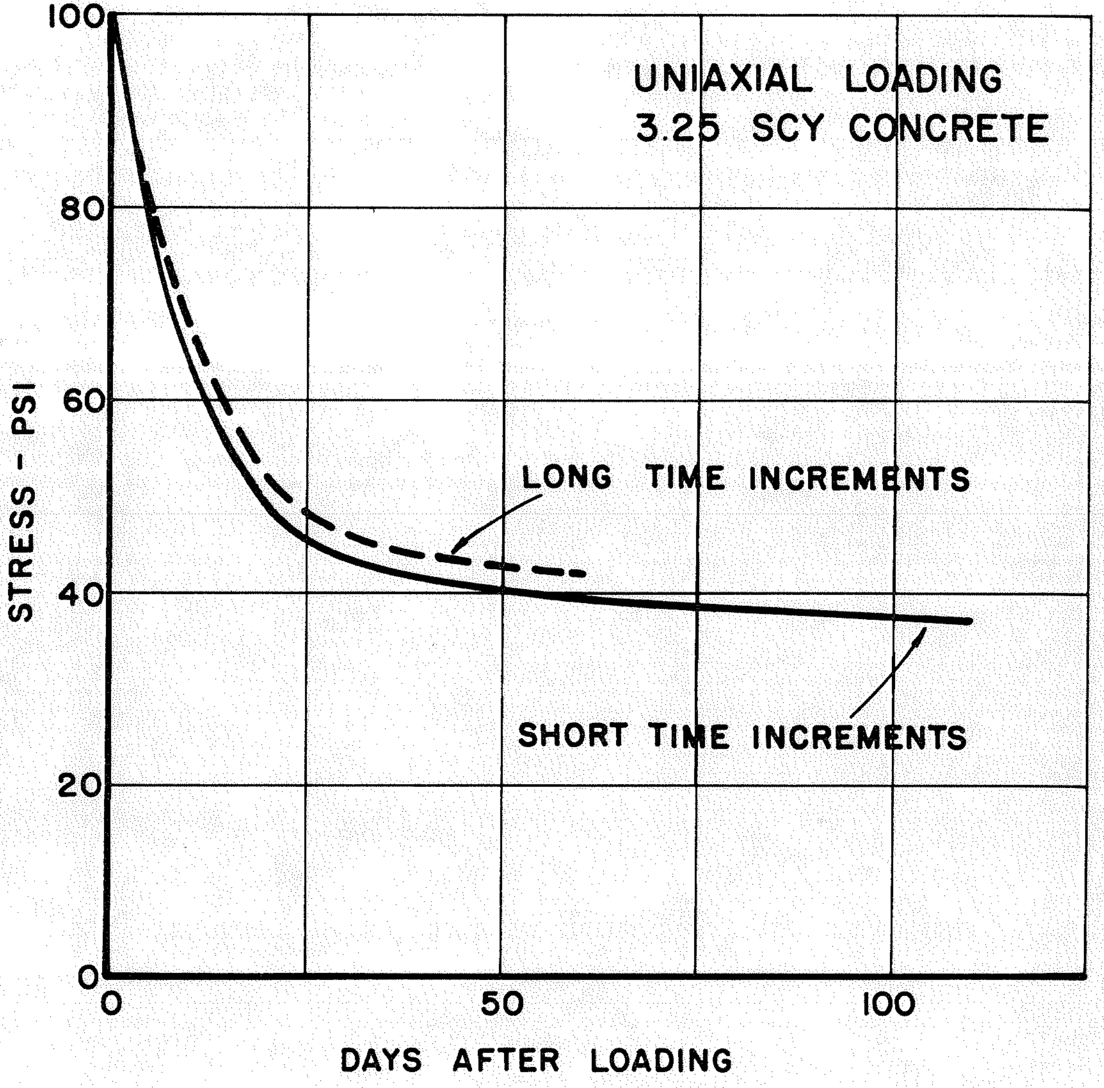


FIG. 4 - INFLUENCE OF SIZE OF TIME INCREMENT ON STRESS RELAXATION

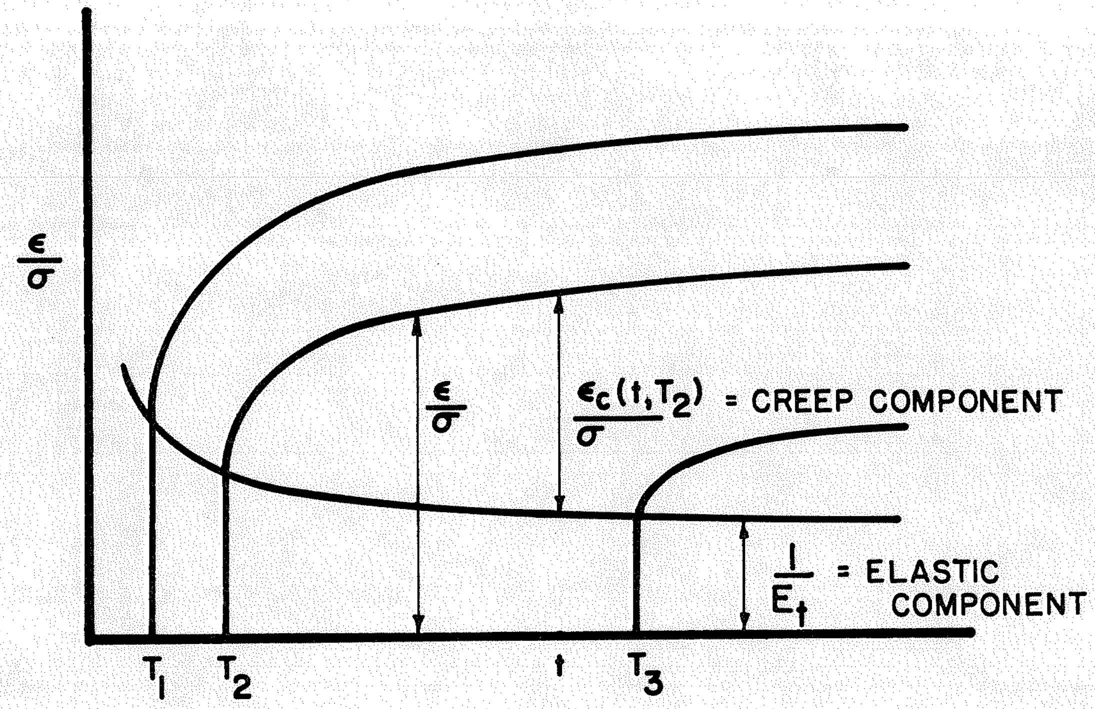


FIG. 5 - TYPICAL CREEP TEST RESULTS

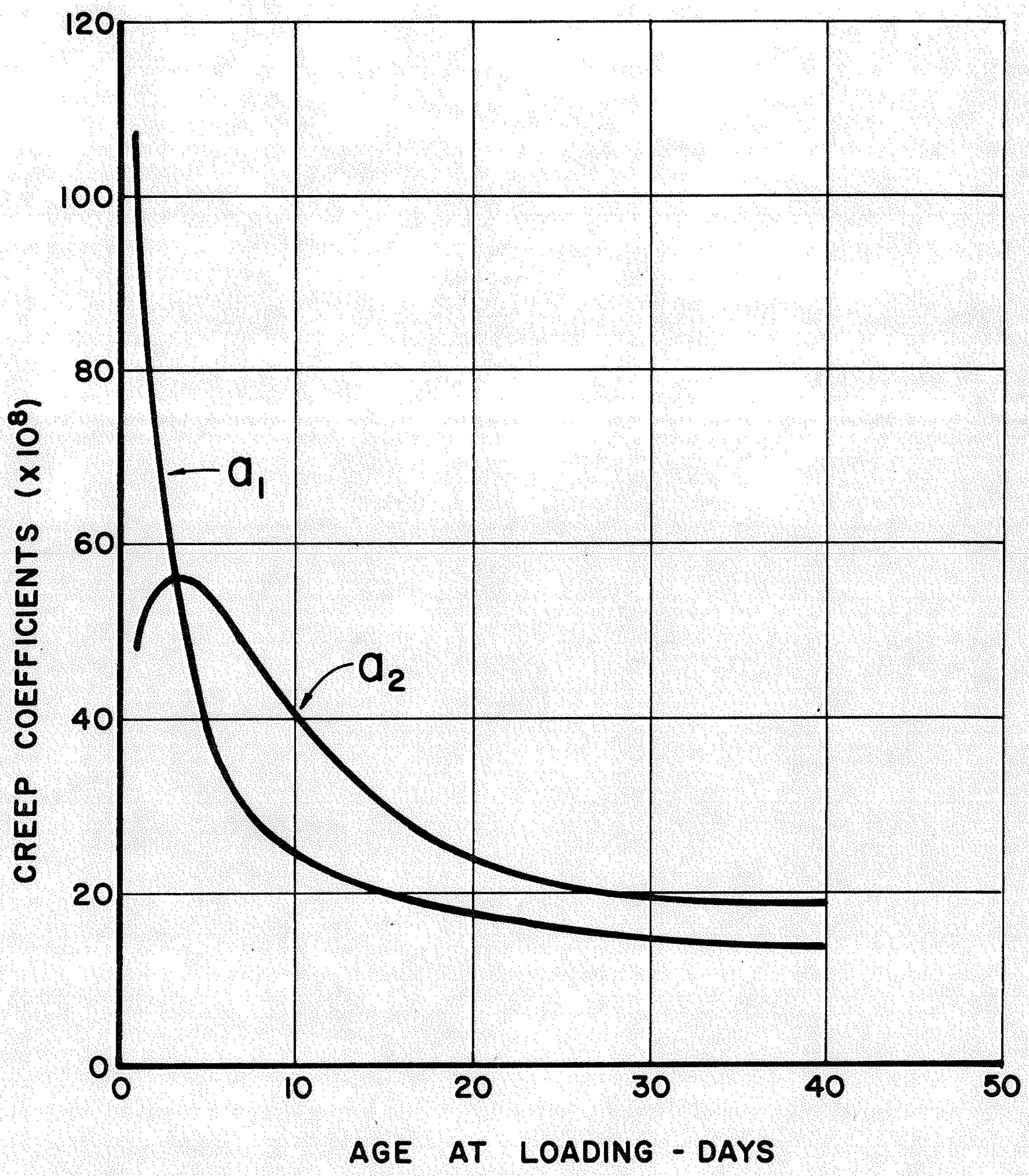


FIG. 6 - VARIATION OF CREEP COEFFICIENTS WITH AGE: 2.75 SCY CONCRETE

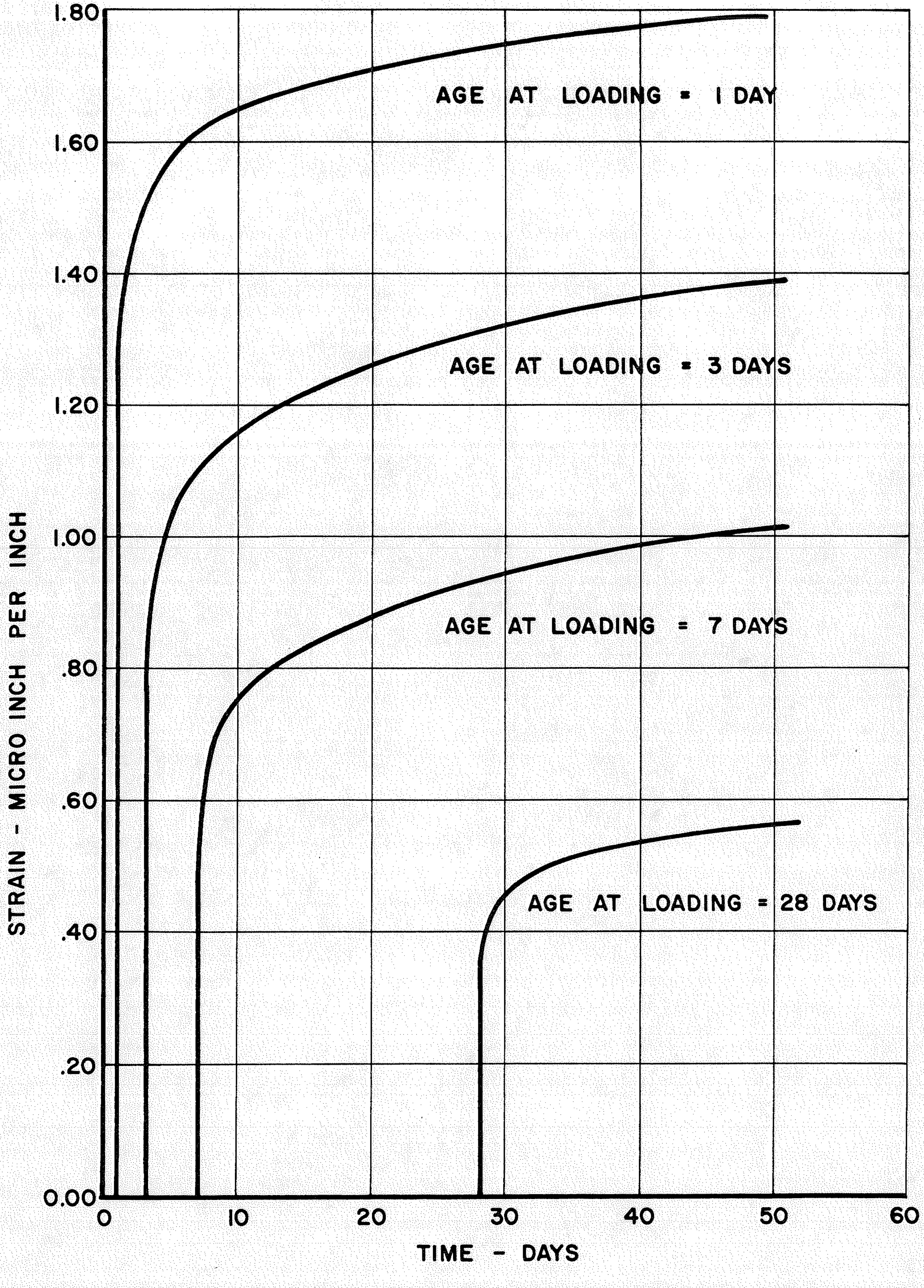


FIG. 7 - CREEP OF 2.75 SCY CONCRETE FOR DWORSHAK DAM (COMPUTED CURVES)

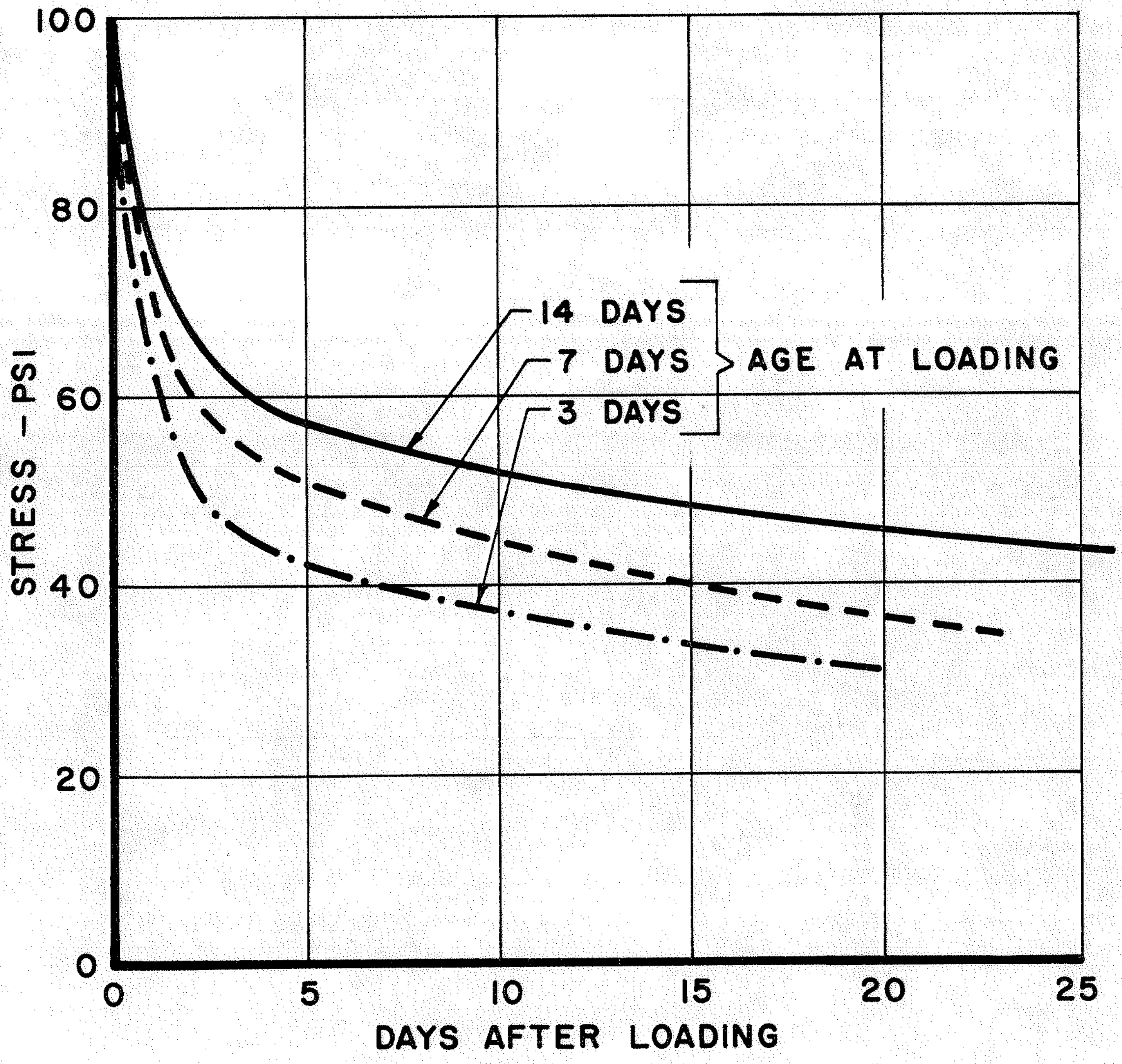


FIG. 8 - STRESS RELAXATION DUE TO CREEP (2.75 SCY CONCRETE UNDER UNIAXIAL STRESS)

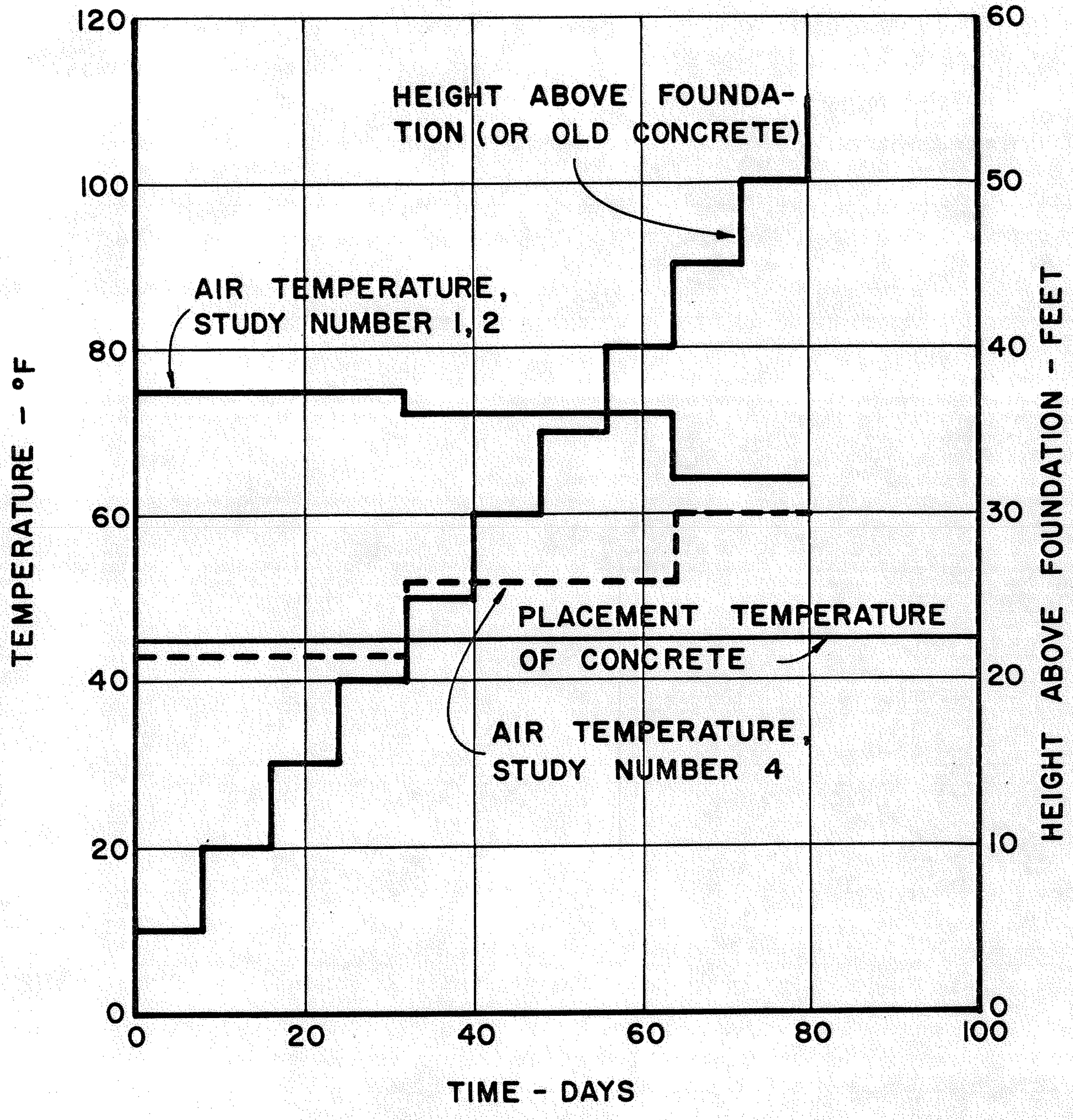


FIG. 9 - AIR TEMPERATURES AND PLACEMENT SCHEDULE: TEMPERATURE STUDIES NOS. 1,2 AND 4 (8 DAY PLACEMENT)

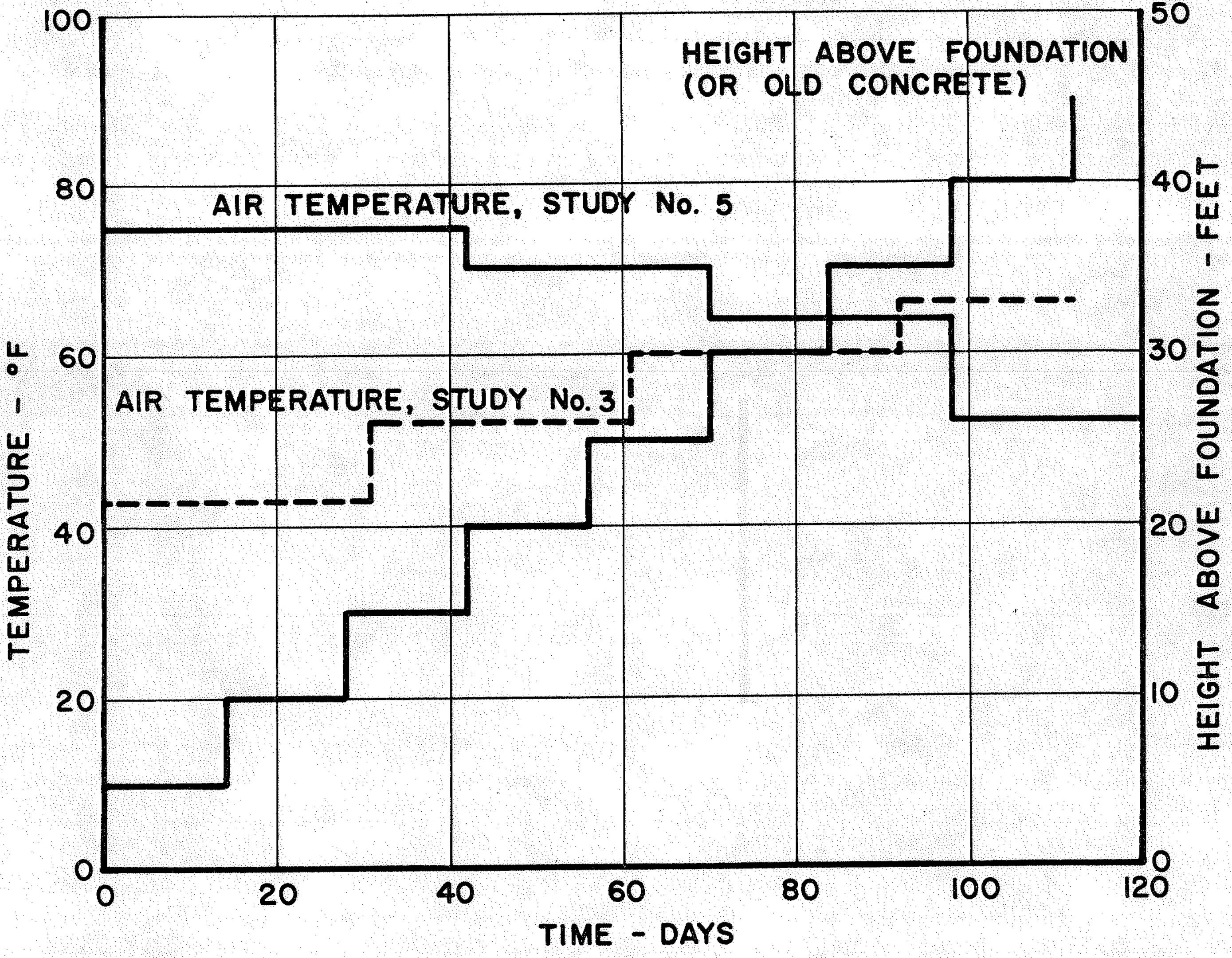


FIG. 10 - AIR TEMPERATURES AND PLACEMENT SCHEDULE: TEMPERATURE STUDIES 3 AND 5 (14 DAY PLACEMENT)

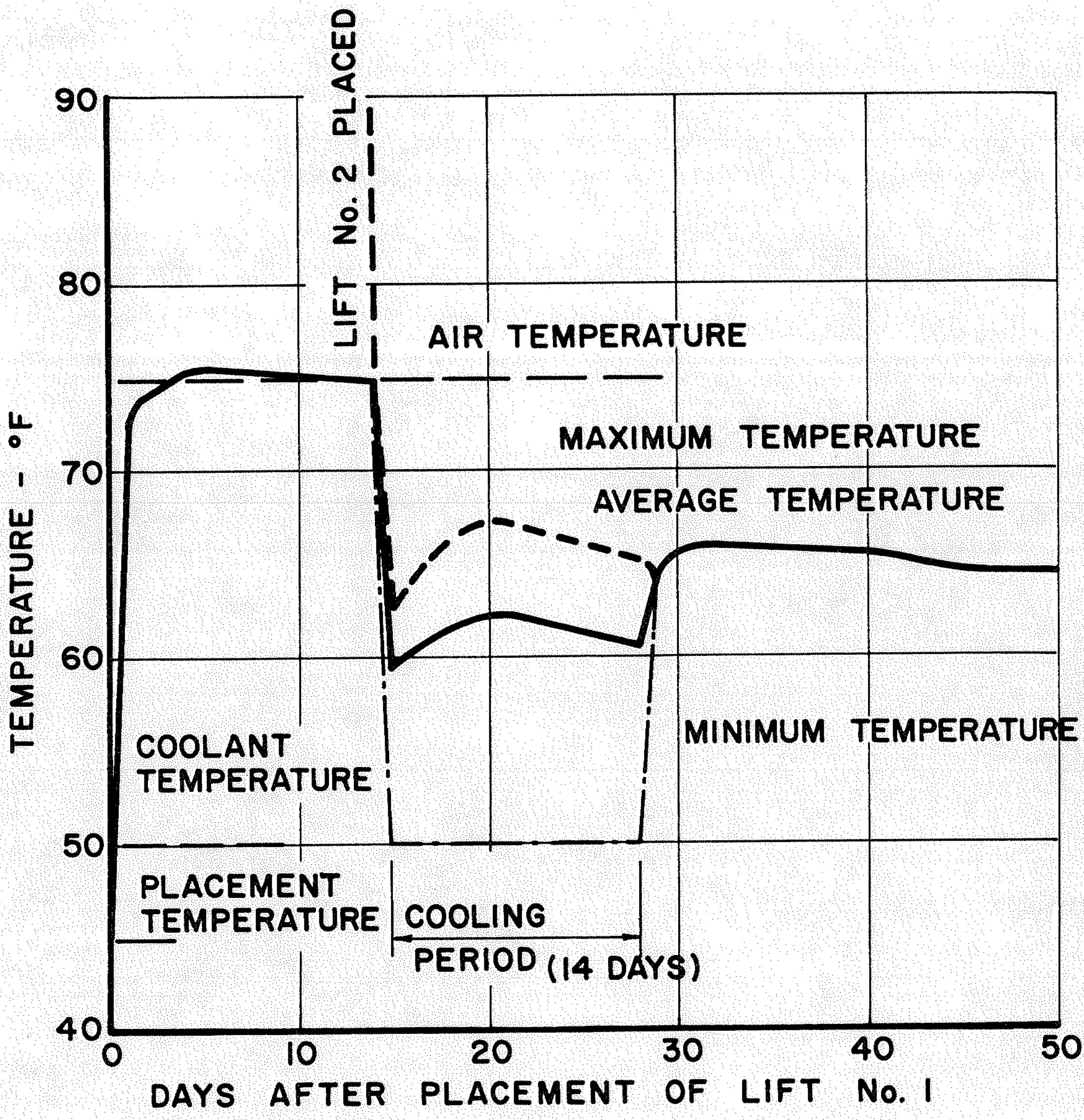


FIG. 11 - TEMPERATURE HISTORY: JOINT BETWEEN LIFTS 1 AND 2, TEMPERATURE STUDY NO. 5

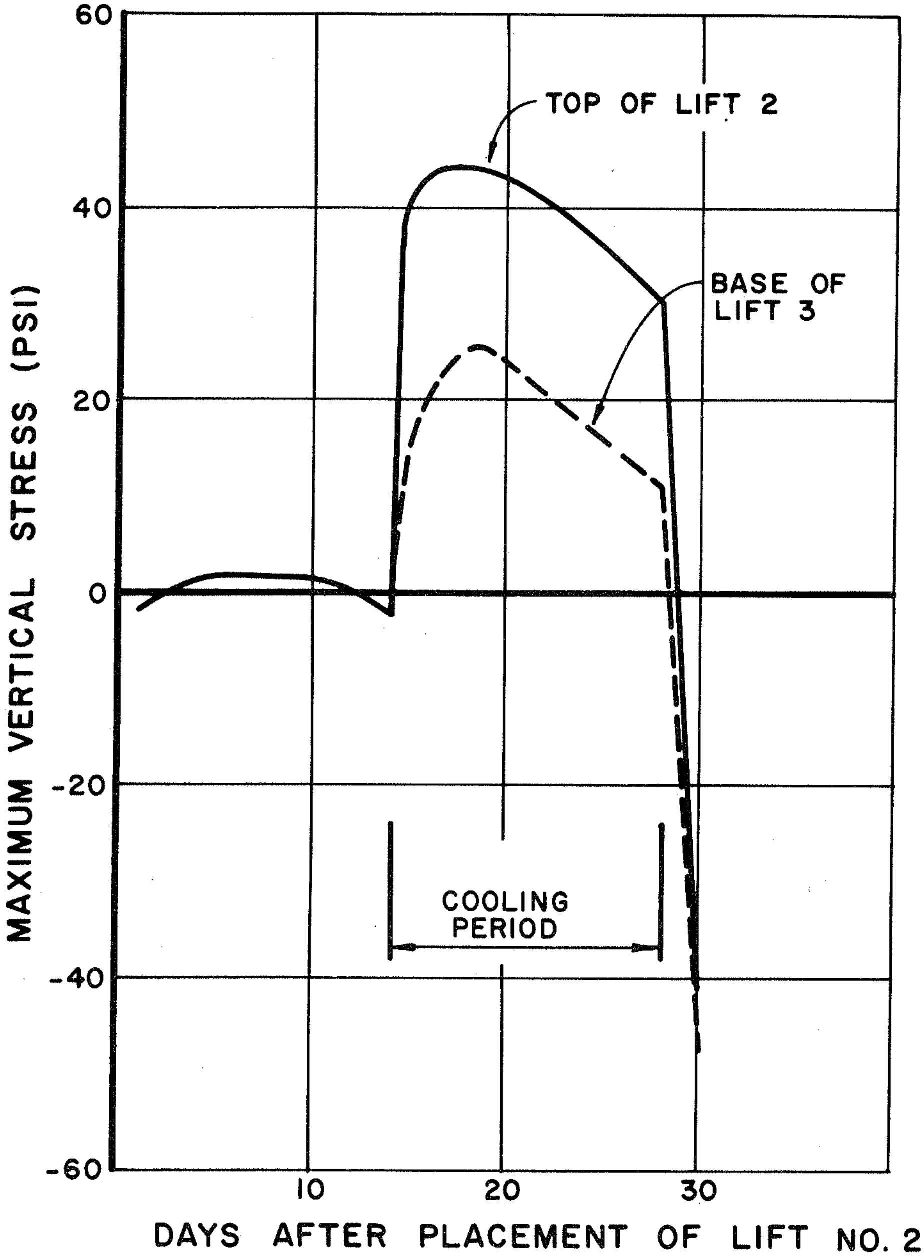
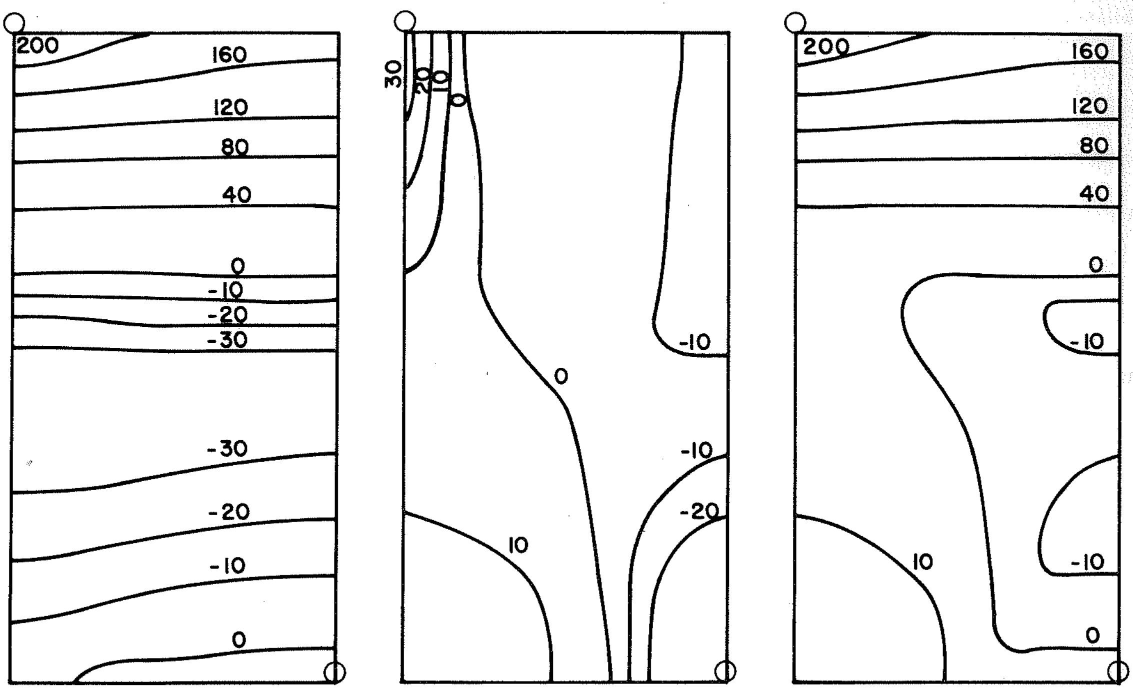
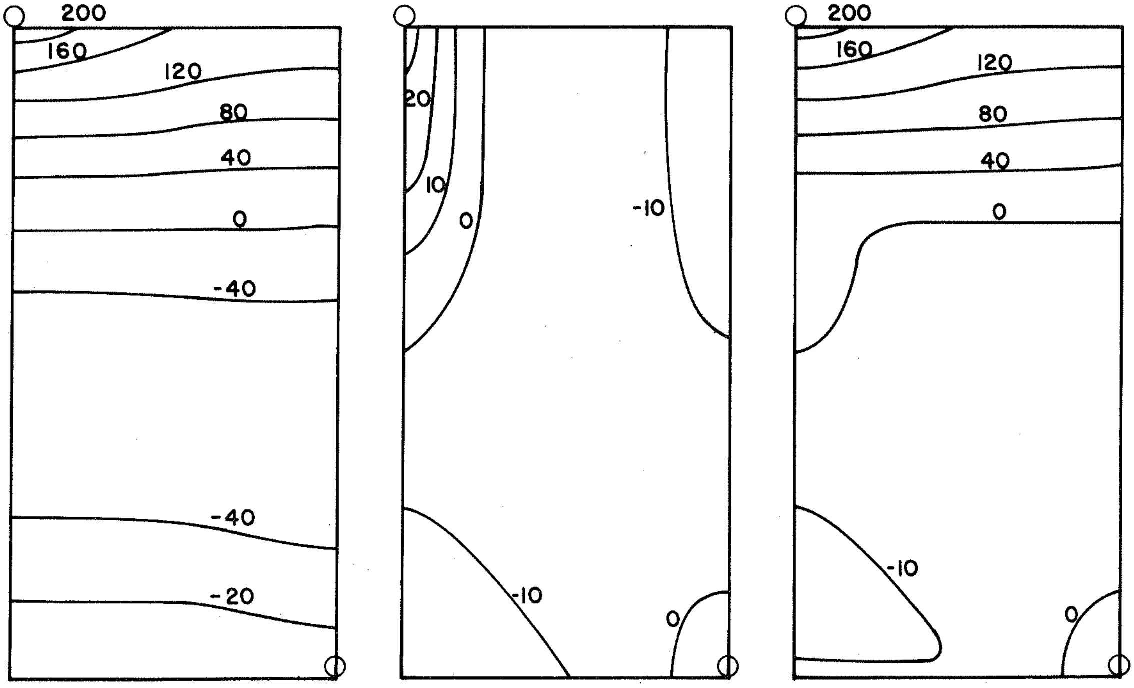


FIG. 13 - VERTICAL STRESS HISTORY AT COOLING PIPE: TEMPERATURE STUDY NO. 5



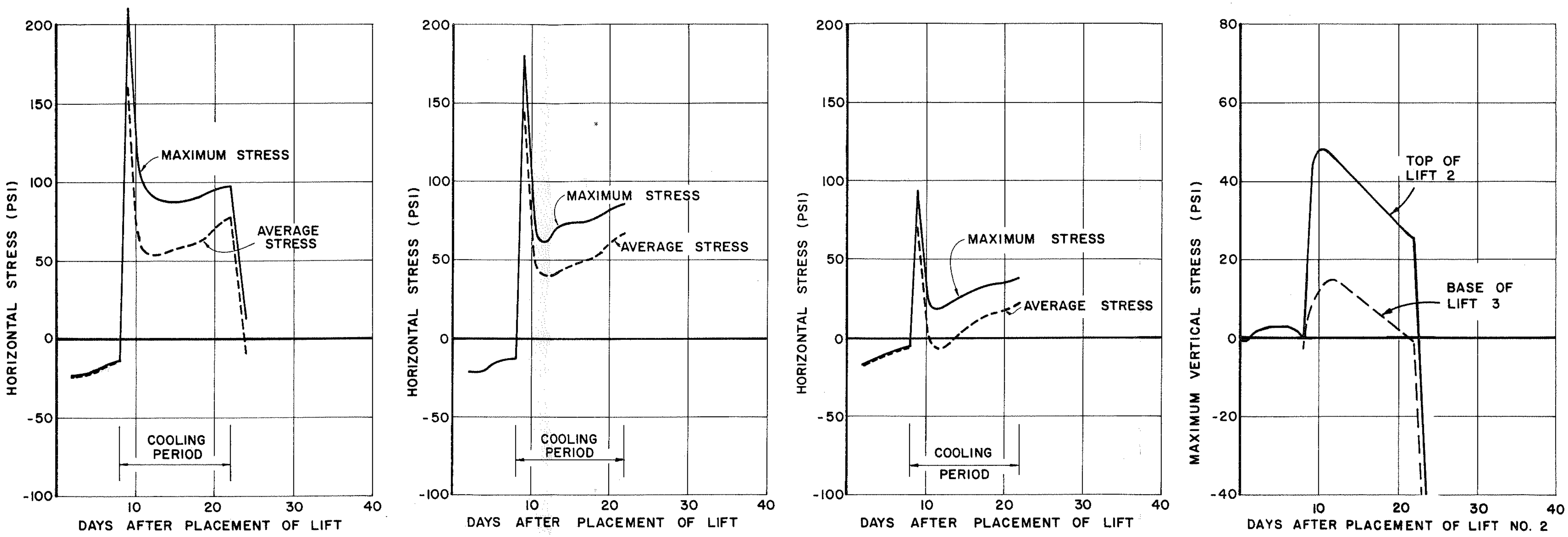
a) HORIZONTAL STRESS b) VERTICAL STRESS c) MAX. PRINCIPAL STRESS

FIG. 14 - STRESS DISTRIBUTION AT TIME OF MAXIMUM TENSION: TEMPERATURE STUDY NO. 5, LIFT NO. 2



a) HORIZONTAL STRESS b) VERTICAL STRESS c) MAX. PRINCIPAL STRESS

FIG. 15 - STRESS DISTRIBUTION AT TIME OF MAXIMUM TENSION: TEMPERATURE STUDY NO. 2, LIFT NO. 2



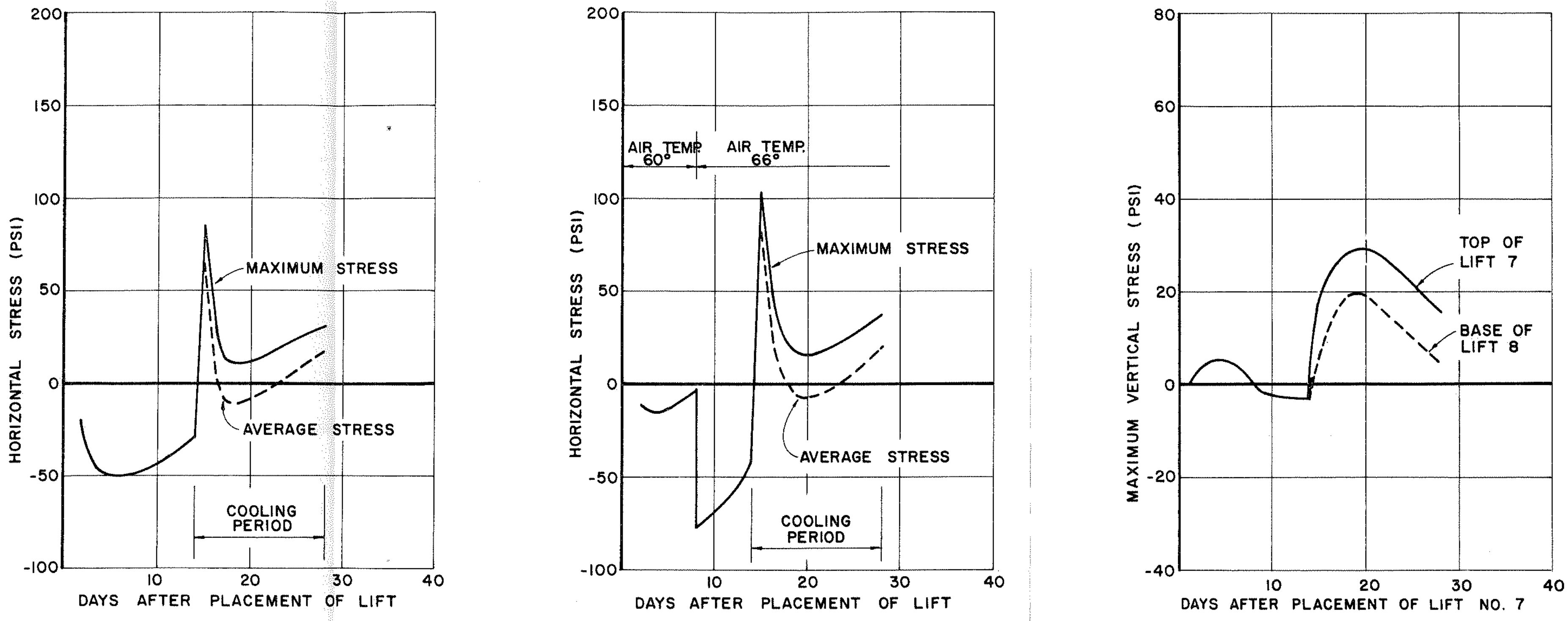
a. TOP OF LIFT NO. 2

b. TOP OF LIFT NO. 6

c. TOP OF LIFT NO. 9

d. STRESS AT COOLING PIPE JOINT BETWEEN LIFTS 2/3

FIG. 17 STRESS HISTORY: TEMPERATURE STUDY NO. 2

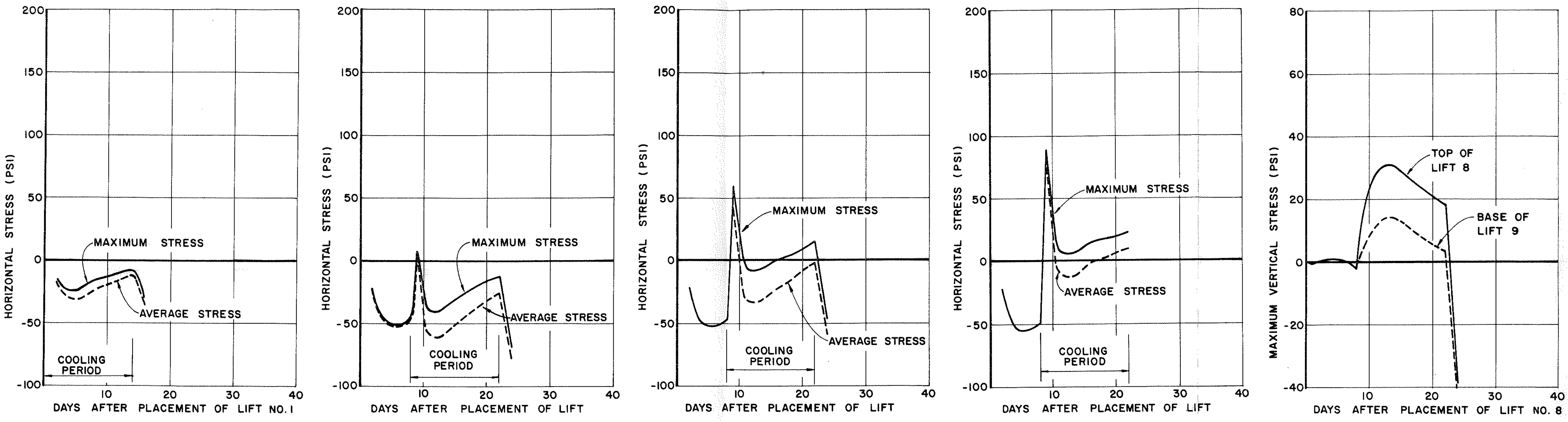


a. TOP OF LIFT NO. 4

b. TOP OF LIFT NO. 7

c. STRESS AT COOLING PIPE JOINT BETWEEN LIFTS 7/8

FIG. 18 STRESS HISTORY: TEMPERATURE STUDY NO. 3



a. TOP OF OLD CONCRETE

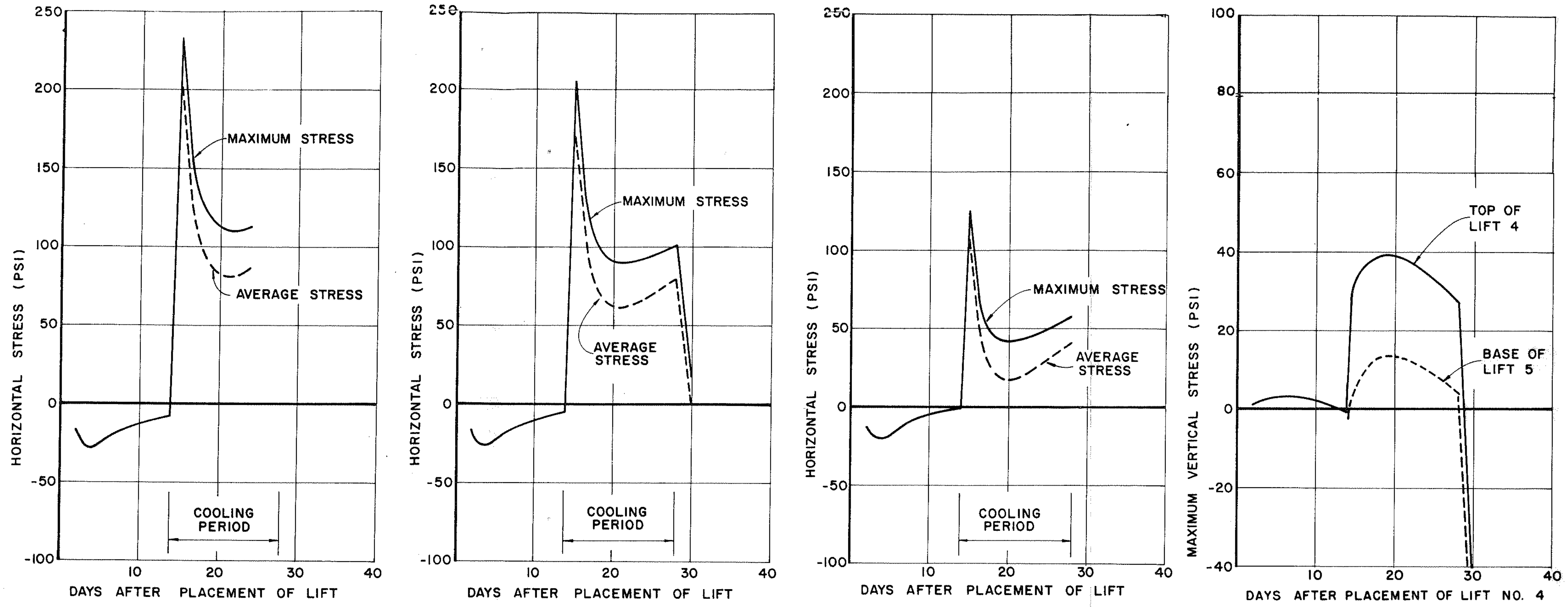
b. TOP OF LIFT NO. 4

c. TOP OF LIFT NO. 8

d. TOP OF LIFT NO. 9

e. STRESS AT COOLING PIPE
JOINT BETWEEN LIFTS 8/9

FIG. 19 STRESS HISTORY:
TEMPERATURE STUDY NO. 4



a. TOP OF LIFT NO. 2

b. TOP OF LIFT NO. 4

c. TOP OF LIFT NO. 6

d. STRESS AT COOLING PIPE JOINT BETWEEN LIFTS 4/5

FIG. 20 - STRESS HISTORY: TEMPERATURE STUDY NO. 5

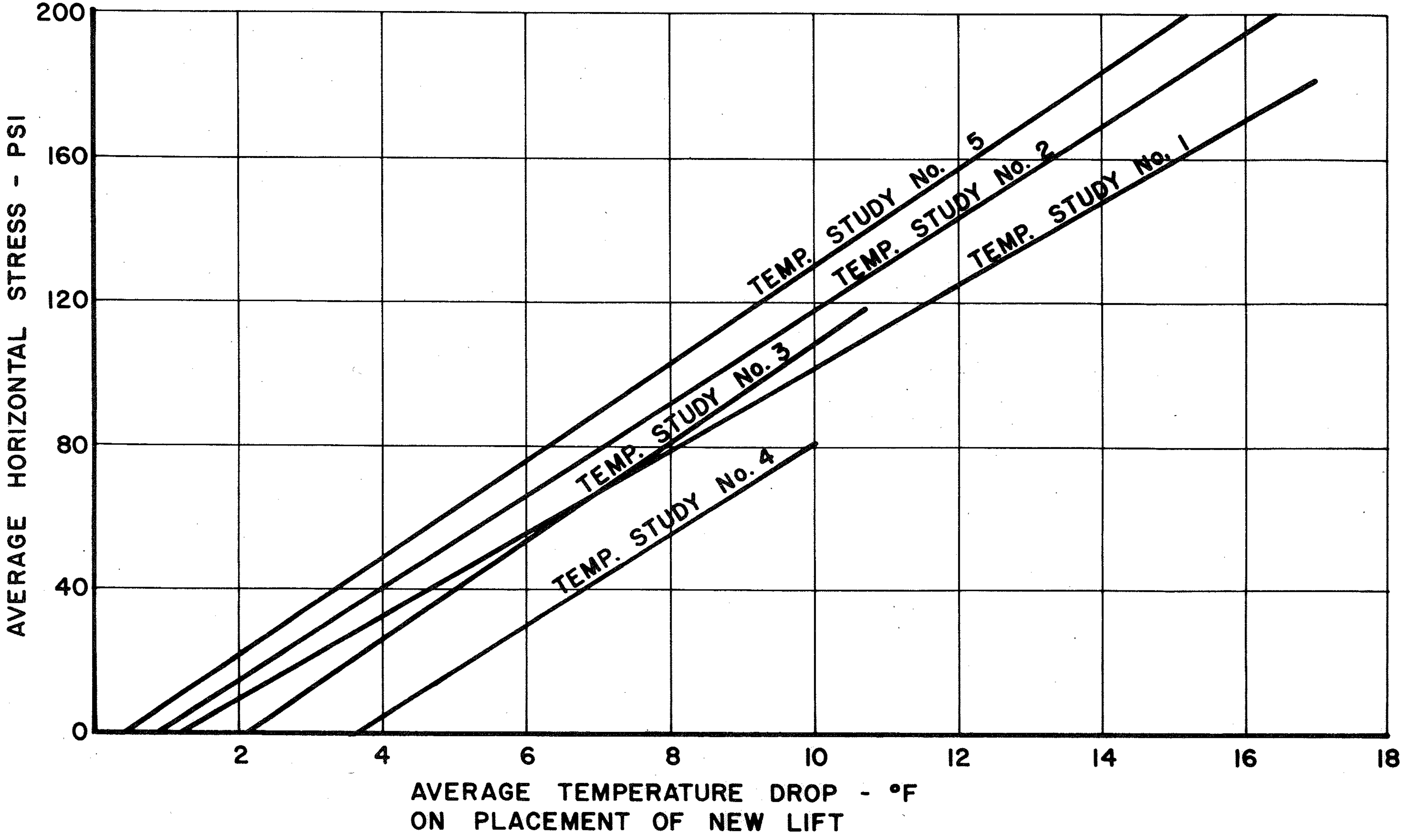


FIG. 21 - RELATION BETWEEN AVERAGE HORIZONTAL STRESS AND AVERAGE TEMPERATURE DROP ON LIFT SURFACE

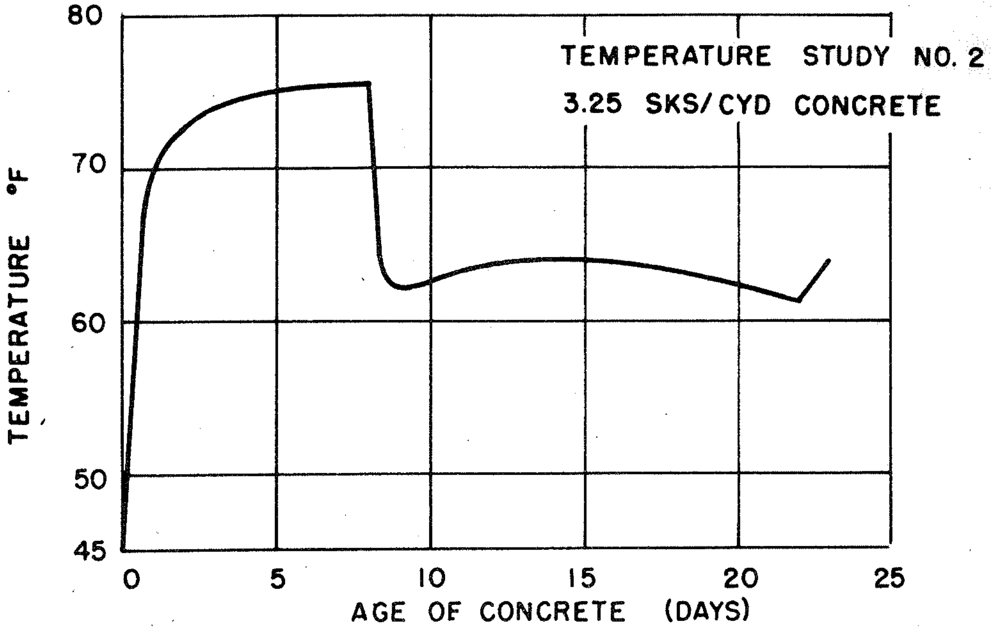


FIG. 22 - TEMP. HISTORY NEAR TOP OF LIFT NO. 2

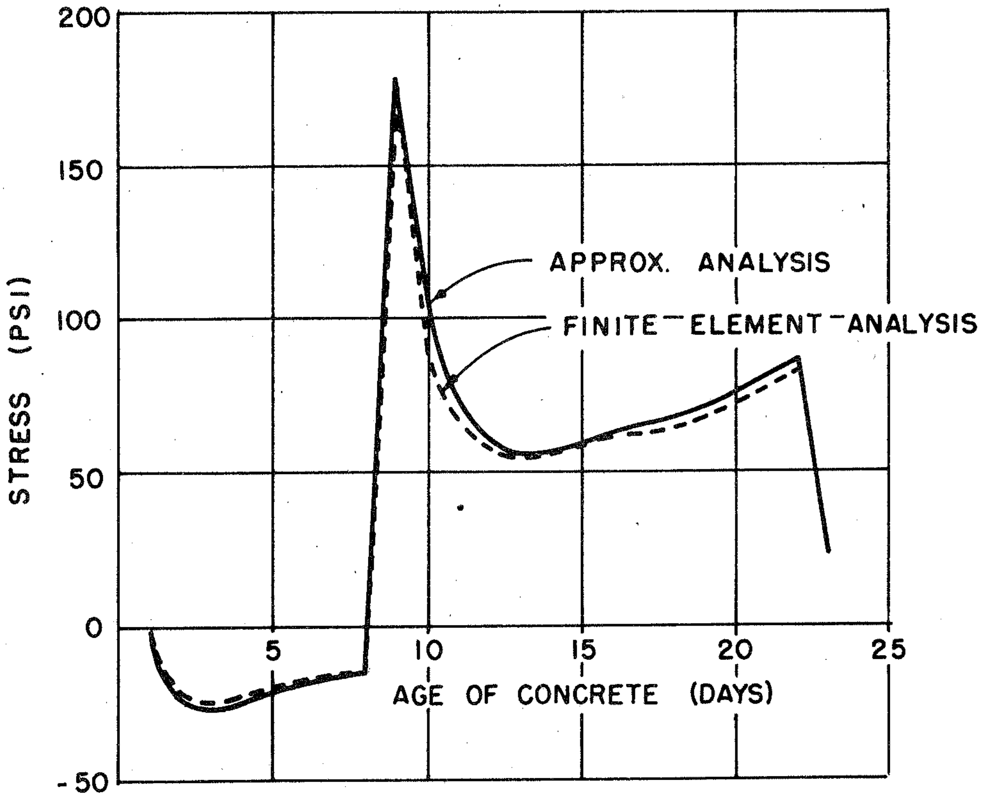


FIG. 23 - HORIZONTAL STRESSES NEAR TOP OF LIFT NO. 2

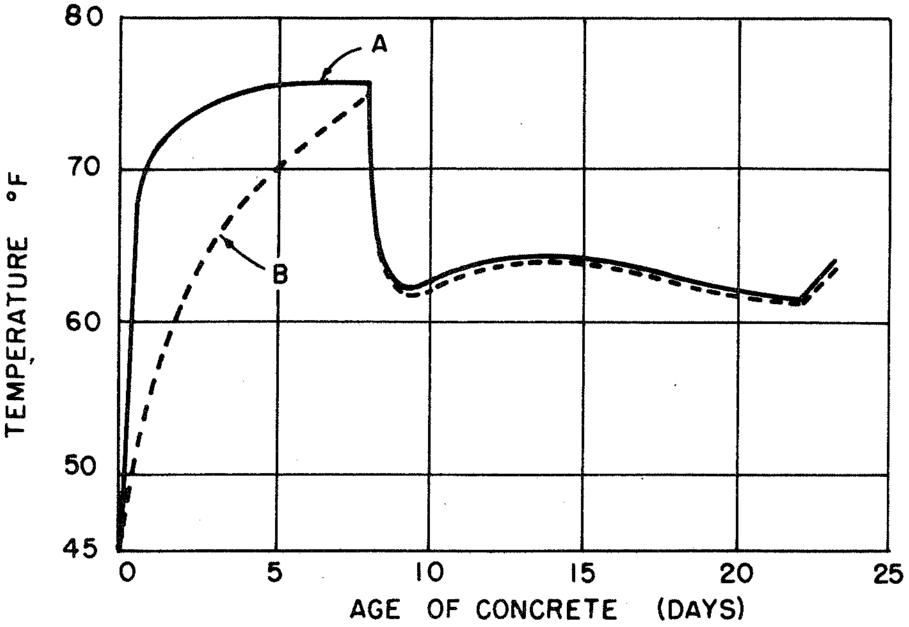


FIG. 24 - HYPOTHETICAL TEMP. HISTORIES

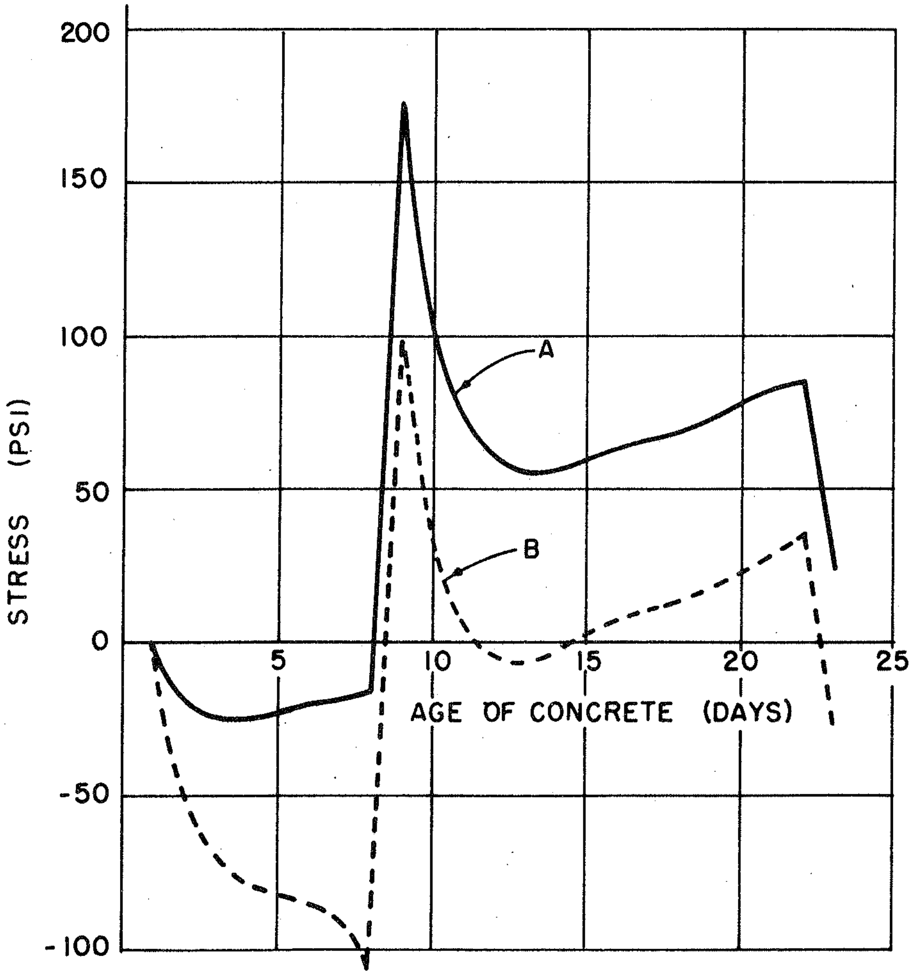


FIG. 25 - STRESSES DUE TO HYPOTHETICAL TEMP. HISTORIES