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CONCRETE TEMPERATURE CONTROL STUDIES TENNESSEE-TOMBIGBEE WATERWAY PROJECTS

by

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Pertinent physical properties of three mass concrete mixtures were determined and used as input to finite-element programs to calculate temperature rise and resulting thermal strains in mass concrete. This concrete mixture proportioned with coarse aggregate having an elastic modulus of 7.8 x 10^6 psi produced 1.3 times higher creep and 1.5 times greater increase in ultimate strain capacity, with lower modulus of elasticity, than did concrete with a coarse aggregate with a modulus of 11 x 10^6 psi. Concrete mixtures tested produced values for

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20. ABSTRACT (Continued)

adiabatic temperature rise between 39°F for 3000-psi concrete with 25 percent pozzolan replacement and 85°F for 4000-psi with no pozzolan.

Finite-element computer simulations of construction of three structural features namely, dam abutment, spillway, and lock wall monoliths were made at three seasonal construction start dates and several placement rates. The resulting thermal tensile strains when compared with strain capacity of the concrete were excessive for several days after placement of the spillway overflow surface and the lower lifts in the pier of the spillway monolith. Although maximum tensile strain versus strain capacity ratios were 0.78 and 0.55 in dam and lock wall monoliths where a value of 1.0 is required to cause cracking, consideration of rapid ambient temperature drop and accelerated initial temperature rise at elevated placement temperatures could increase these ratios to near the critical 1.0 value.

Recommendations include consideration of a reduction in placement temperature for 3000- and 4000-psi concrete; that pozzolan replacement be used in all concrete mixtures; and that insulation be considered for use in selected areas of the structures if cracks occur.

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PREFACE

A program to conduct mass-concrete temperature rise and thermal strain analysis for structures of the Tennessee-Tombigbee Waterway was authorized by DA Form 2544, Intra-Army Order for Reimbursable Services, No. 75-012, dated 5 August 1974, with attachment and inclosures, from US Army Engineer District, Mobile.

The work reported herein was conducted between August 1974 to May 1977 at the Concrete Laboratory of the US Army Engineer Waterways Experiment Station (WES) under the direction of Messrs. Bryant Mather, Leonard Pepper, and Mrs. Katharine Mather, by or under the supervision of Messrs. B. R. Sullivan, J. E. McDonald, W. O. Tynes, and W. G. Miller. Mr. A. A. Bombich was project leader and with Messrs. Sullivan and McDonald prepared this report.

Commanders and Directors of WES during conduct of this study and preparation and publication of this report were COL G. H. Hilt, CE, and COL J. L. Cannon, CE. Mr. F. R. Brown was Technical Director.

CONTENTS

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
pound force per square inch per minute	114.91267	pascal per second
pound (force) per square inch (psi)	6894.757	pascal
calorie per gram	4.184	Joule per kilogram
Fahrenheit degrees	5/9	Celsius degrees*
pounds per cubic yard	0.5932764	kilograms per cubic metre
Btu per hour · square foot · degree Fahrenheit	5.678263	watt per square metre · Kelvin
Btu · inch per hour · square inch · degree F	20.7688176	watt per metre · Kelvin
inch per degree Fahrenheit	0.014111111	metre per Kelvin
pound (mass) per cubic foot	16.01846	kilogram per cubic metre
Btu per pound (mass) · degree Fahrenheit	4186.8	Joule per kilogram · Kelvin
square foot per hour	0.0000258064	square metre per second

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings use: $K = C + 273.15$.

CONCRETE TEMPERATURE CONTROL STUDIES - TENNESSEE-TOMBIGBEE WATERWAYS PROJECTS

PART I: INTRODUCTION

Background

1. A primary concern during and immediately following completion of mass concrete structures is the control of thermal cracking. During construction, heat is produced by the hydration of cement causing a temperature rise in the concrete. Subsequent thermal gradients occur due to cooling by external temperature change. Concrete temperature change causes proportional volume change that if restrained either externally or internally by the mass of concrete itself will produce thermal strains. Thermal strains can occur at any time during construction or after completion of a structure and can be sufficient to cause the structure to undergo random cracking.

2. Various techniques to reduce the potential for thermal cracking have been developed. These include reducing the potential temperature rise of mass concrete by limiting the heat of hydration of cements used, minimizing cement content, or replacing part of the cement with pozzolan. Other measures include precooling the aggregate to reduce placement temperature, insulating surfaces to control thermal absorption or loss, and post-cooling of the mass by embedded cooling pipes. The degree of controls specified for a particular structure depends largely upon the size and geometry of the structure, climate, economics, and severity of cracking if controls are not specified.

3. Numerical methods have been developed to predict temperature distribution and resulting thermal stresses and strains in mass concrete structures. The finite-element method (FEM) employed in the computer is the most effective numerical method yet developed since it is completely general with respect to geometry, material properties, and boundary conditions. The FEM programs used in this investigation provide the

capability of simulating incremental construction of mass concrete structures and include those measures considered necessary for thermal strain control.

Purpose and Scope

4. This report presents the results of an investigation to determine the extent of thermally induced strains in structures of the Tennessee-Tombigbee Waterway and includes recommendations to reduce or control excessive thermal strains. These recommendations can be used to assist in development of design memoranda for concrete temperature on all waterway projects except Bay Springs Lock and Dam. Bay Springs is not included because of its substantially larger size and was investigated* separately.

5. The properties of several required concrete mixtures containing materials most available or most likely to be representative of those used in the concrete for the Waterway are determined for input to the computer simulations. Computer runs provide simulated construction of several structural geometries typical of Waterway projects. Several concrete placement rates and construction start dates are simulated within a range of concrete placement temperatures. The FEM programs calculate temperature distribution histories and resulting thermal stresses and strains in structures modeled for the various construction parameters. Tensile strains are compared with ultimate tensile strain capacity test results for the concrete mixtures used in this study. These comparisons are the basis for thermal strain control recommendations.

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* Letter report dated 20 May 1976 from US Army Engineer Waterways Experiment Station to US Army Engineer District, Nashville, subject: Thermal Study - Bay Springs Lock and Dam.

PART II: LABORATORY TESTS OF CONCRETE

6. This part of the overall investigation was intended to develop information on the physical characteristics of concrete mixtures used. The concrete mixtures tested were proportioned to meet strength requirements and contain materials likely to be used in the Tennessee-Tombigbee Waterway projects. These concrete properties were necessary for input to the computer simulations. The total number of mixtures possible as a result of the combinations of compressive strength requirements and concrete materials selected could not all be prepared or tested completely. Therefore, only those mixtures were proportioned and tests conducted to provide the basic properties necessary and to assess the effect on concrete properties resulting from the possible concrete material combinations.

Concrete Materials

7. Fine aggregate was a natural sand (MOB-39 S-1) provided by the Mobile District taken from the vicinity of the Tombigbee River in Mississippi. Coarse aggregates from two sources in the general vicinity of the Waterway were chosen for concrete mixtures to be used in this study. These included crushed limestone coarse aggregates from Central Alabama (CRD G-31) and northeast Mississippi (MOB-39 G-1) that were assumed representative of the area. The Alabama aggregate was stocked at the WES Concrete Laboratory while the Mississippi aggregate was obtained for this study. The possibility that natural chert gravel coarse aggregate will be used in any of these structures is believed to be small because gravel is not usually available in 1-1/2-in. maximum size in the area.

8. Modulus of elasticity, compressive strength, and thermal diffusivity were rock properties desired for these aggregates. Modulus of elasticity and compressive strength were already known for the Alabama aggregate. Four each, 2- by 4-in. nominal size rock cores were drilled

from selected pieces of large aggregate from the Mississippi source. The cores were sawed and ends ground before mounting two longitudinal strain gages, diametrically opposite, on each specimen. Cores were tested to determine compressive strength and modulus of elasticity.

9. An additional four cores of 4-in. diameter were drilled, two from each aggregate type. Ends of the cores were sawed to produce cores of maximum available length. Cylinder lengths ranged from 4-1/2 to 7 in. Holes, 1/4 in. in diameter, were drilled axially into the specimens and thermocouples embedded at the centroid. Thermal diffusivity was determined for each specimen by CRD-C 36.[1] Resulting values of modulus of elasticity, compressive strength, and thermal diffusivity for the two aggregate types are shown in Table 1.

10. Type II low-alkali portland cement with a heat of hydration limit of 70 cal/g at 7-days age (Type II moderate heat of hydration) was the cement used for all concrete mixtures proportioned during this study. The pozzolan was a fly ash produced in Alabama (MOB-39 AD-1). Two additional cements were considered early in the study when availability problems appeared possible with Type II nominal moderate heat of hydration cement for impending waterway projects. These cements included a Type I portland cement and a Type I portland cement meeting normal Type II specifications (Type II). Chemical and physical properties were determined for the three cements and the fly ash used.

11. Heat of hydration (CRD-C 229) determinations were made for the three cements and a cement-fly ash blend. These data were developed for information and to supplement concrete adiabatic temperature rise data if necessary. The Type II nominal moderate heat and Type II cement with 25 percent pozzolan replacement were tested at isothermal storage temperatures of 73°F at 4 ages, 100°F at 3 ages, and 130°F at 3 ages. The 7-day tests at 130°F are probably low due to moisture loss during hydration. Type II and Type I cements were tested at standard 73°F storage temperatures for 1, 3, 7, 28, and 7 and 28 days, for comparison. All cement properties are shown in Tables 2 and 3.

Type II moderate-heat cement became the only cement used when availability problems eased; thus, no further use was made of the other Type I and II cements.

Concrete Mixtures

12. Concrete mixture proportions for 90-day compressive strengths of 2000, 3000, and 4000 psi were derived from those used at Gainesville Lock and Dam. The cement content for the 2000- and 3000-psi mixtures was increased by 24 lb/yd^3 to conform with a similar modification at Gainesville and the maximum aggregate size was reduced from 6 to 4-1/2 in.

13. Three concrete mixtures designated A, B, and C were proportioned with the natural sand and crushed limestone coarse aggregate (CRD-G 31) to conform to the Gainesville mixtures for compressive strengths of 3000, 4000, and 2000 psi, respectively. These mixtures contained Type II moderate-heat cement without fly ash replacement and were the primary mixtures used in this study. The cement content of mixture B was reduced by 56 lb/yd^3 when trial batches yielded excessive compressive strength when the Gainesville proportions were used. Mixture B-ATR was proportioned with the Gainesville cement content and adiabatic temperature rise was determined. All other concrete properties tests were determined for mixture B. This procedure was followed so that mechanical properties would be determined for 4000-psi concrete with the required compressive strength, but the heat produced would be determined for the mixture with maximum cement content that might be used.

14. Two additional mixtures were proportioned to isolate the changes in mechanical properties resulting from use of the second coarse aggregate and the adiabatic temperature rise resulting from the use of pozzolan replacement. Mixture D was proportioned with the same natural sand and with crushed limestone coarse aggregate (MOB-39 G-1) from the second source for 90-day compressive strength of 3000 psi. The properties of this mixture can be compared with those of mixture A to observe the effects of using different coarse aggregate. Mixture AF,

derived from mixture A, was proportioned with 25 percent fly ash replacement to determine the resulting difference in adiabatic temperature rise. The proportions of all mixtures used can be found in Table 4.

15. Average physical characteristics of the freshly mixed concrete used for all test specimens, excluding those prepared for adiabatic temperature rise tests, were as follows:

	Mixture			
	A	B	C	D
Slump,* in.	2	3-3/4	2	1-3/4
Air content,* %	5.4	5.8	4.8	4.8
Concrete placement temperature, F	50.0	52.7	51.3	50.5

*Determined on that portion of the mixture passing 1-1/2-in. sieve.

The concrete materials were precooled to approximately 35°F to obtain the lowered placement temperature.

Fabrication of Concrete Specimens

16. Test specimens excluding those for adiabatic temperature rise tests were fabricated from the four concrete mixtures as follows:

	Mixture			
Specimens	A	B	C	D
6-in. by 12-in. cylinders	33	27	16	33
6-in. by 16-in. cylinders	10	8	8	10
12-in. by 12-in. by 66-in. beams	6	3	3	6

With the exception of mixture B which had 1-1/2-in. maximum size aggregate, all cylinders were fabricated from concrete wet-sieved through the 1-1/2-in. sieve. A Carlson strain meter was embedded in each 6- by 12-in. cylinder. All cylinders except 3 each from mixtures A, B, and D used for thermal diffusivity were cured in their molds under moist burlap for three days then stripped and sealed to prevent drying during storage at approximately 73°F prior to testing.

17. Carlson strain meters were positioned in the beam forms parallel to the top and bottom surfaces, 1-1/2 in. from the concrete surface, and centered within the middle third of a 60-in. single span prior to concrete placement. All beams were consolidated internally, the surfaces finished, and the beams cured in their molds under moist burlap at approximately 73°F for three days. Beams were then stripped, turned so that the two parallel molded surfaces became the top and the bottom, and sealed to prevent moisture loss during storage and testing.

Compressive Strength, Modulus of Elasticity, and Poisson's Ratio

18. Four strain gages, two each lateral and longitudinal, were mounted on the surface of each cylinder one day before testing. Three specimens of mixtures A and D were tested at each of 3-, 7-, 28-, 90-, and 180-days age. Similar tests were conducted on specimens from mixtures B and C except that 180-day tests were not made and, in the case of mixture C, only two specimens were tested at each age. Test results are presented in Table 5 and compressive strength gain characteristics are shown in Fig. 1.

Splitting Tensile Strength

19. Fifty 6- by 12-in. cylinders were tested in accordance with CRD-C 77 to determine splitting tensile strengths of the four concrete mixtures. Testing ages and the number of specimens tested at each age were the same as previously described for the compressive strength tests. Results are shown in Table 5 and splitting tensile strength-gain characteristics are given in Fig. 2.

Coefficient of Linear Thermal Expansion

20. Six 6- by 16-in. concrete specimens, two each from mixtures A-C, were subjected to temperatures of 40, 60, and 80°F. Strain readings were made immediately prior to removal from storage at 73°F and then over a three-day period during which the specimens were subjected to each of the three testing temperatures for 24 hr. Results indicated the coefficients of linear thermal expansion to be 5.64, 5.84, and 5.76 millionths per degree F for mixtures A, B, and C, respectively.

Adiabatic Temperature Rise

21. Four adiabatic temperature rise tests were conducted in accordance with CRD-C 38. Adiabatic temperature rise was determined for mixtures A, AF, and B-ATR at an initial concrete placement temperature of approximately 55°F. Results (Fig. 3) indicate temperature rise at 28 days to be 46.4°F, 38.8°F, and 80.6°F, respectively. The adiabatic temperature rise curve for mixture C also shown in Fig. 3 was calculated from that of mixture A in proportion to the cement contents of the two mixtures. Adiabatic temperature rise was also determined for mixture A at an initial temperature of approximately 80°F to evaluate how initial accelerated heat production for concrete placed at higher temperature affects adiabatic temperature rise. Temperature rise at 28-days age was 45.8°F or very near the same (46.4°F) as for mixture A placed at 55°F. However, the adiabatic temperature rise difference at 1-4 days-ages (Fig. 4) was nearly 8°F. Although higher placement temperature results in near equal ultimate adiabatic rise, early adiabatic rise is substantially accelerated.

Thermal Diffusivity

22. Six 6- by 12-in. cylinders, three each from mixtures A and D, were fabricated from concrete wet-sieved through the 1-1/2 in. sieve.

Three additional 6- by 12-in. cylinders were fabricated from 1-1/2-in. maximum size aggregate mixture B. Copper-constantan thermocouples were cast at the centroid of each cylinder. Specimens were cured at 100 percent relative humidity. The average thermal diffusivity of these specimens was determined at 28-days age in accordance with CRD-C 36. The thermal diffusivities of mixtures A, B, and D were 0.0373, 0.0325, and 0.0288 ft^2/hr, respectively.

Uniaxial Creep

23. Eight 6- by 16-in. cylinders, two each from mixtures A, B, C, and D were loaded incrementally in uniaxial compression to 670 psi at 7-days age. In addition, four specimens, two each from mixtures A and D, were similarly loaded to 1970 and 760 psi at 90 and 28 days, respectively. Results of periodic strain measurements on these specimens during the sustained loading period are shown in Fig. 5 and 6. Two 6- by 16-in. specimens from each mixture were used as controls to determine volume changes under each set of static temperature and moisture conditions. Curves-of-best-fit based on least squares analyses were computed as shown in Tables 6-11 and Fig. 7 and 8 for the creep test data which have been corrected for the appropriate volume changes.

Ultimate Strain Capacity

24. Both rapid- and slow-loading rates were used in testing the strain capacity beams under 1/3 point flexural loading. One beam from each mixture was loaded to failure in the closed-loop test system at 7-days age using a conventional rapid-loading rate of 40-psi fiber stress per minute. Strains were recorded after each 500-lb increment of load. A second beam from each mixture was started in its slow-loading cycle at 7-days age, and a third beam was stored under no load at approximately $73^{\circ}F$. In the slow-loading cycle, increments of 25-psi fiber stress were applied to the beam weekly until failure occurred. At the time of

failure in the slowly loaded beam, the third beam was tested to failure using conventional rapid loading. The remaining beams from mixtures A and D were tested in a similar sequence with testing initiated at 90- and 28-days age, respectively. Individual beam test results are shown in Table 12. Load-strain data obtained from the beam tests are shown in Fig. 9 through 12.

25. The tensile strain capacity (extrapolated outer fiber strain at 90 percent of ultimate load) in the 7-day slow-loading tests ranged from 1.23 to 1.89 times that obtained in the rapid-loading tests at 7-days age. Mixture D exhibited the greatest increase in strain capacity under slow loading. Similarly, mixture D exhibited the highest creep in tests initiated at 7-days age. The strain capacity of beams, tested in rapid loading the same day comparison beams failed in the slow-loading tests, ranged from 57 to 76 millionths. In comparison the strain capacity of the slow-loaded beams ranged from 64 to 104 millionths. This represents an increase in strain capacity, in slow versus rapid loading, of 0.98, 1.58, 1.56, and 1.37 times for mixtures A, B, C, and D, respectively.

26. Increasing maturity of concrete has a dual effect on tensile strain capacity. With significant increases in stress capacity (Fig. 13) the tensile strain capacity would, by definition, be expected to increase. However, at the same time the modulus of elasticity is increasing (Fig. 14), and this increased stiffness tends to reduce the strain capacity. In the case of the rapid-load tests the combined result of these two effects was slight increases in strain capacity with time (Fig. 15).

PART III: FINITE-ELEMENT METHOD COMPUTER PROGRAMS

27. Two two-dimensional finite-element method (FEM) computer programs were used in this study. The first program, developed by Dr. Edward Wilson of the University of California at Berkeley[2] and modified for use at the WES, calculates temperatures within a mass concrete structure. A second program, written by R. S. Sandhu and associates also at Berkeley[3] and modified at WES, calculates the thermal stresses and strains within the structure resulting from gravity and the thermal loads produced by the temperature-calculation program.

28. Both programs use the same FEM model of the structure. The model subdivides the structure into a grid pattern in which the intersection points are called nodes and the enclosed areas are elements. Lift and material interfaces must correspond to an element boundary. Plots of models used in this study are shown in Fig. 17, 19, and 21. Both programs incrementally simulate placement of concrete in lift thicknesses and at elapsed placement times as prescribed by the user.

Temperature-Calculation Program

29. The temperature program calculates temperatures at each node in the FEM model. Temperature calculations are based upon concrete placement temperature, hydration heat generated, and the thermal properties of the concrete which govern heat flow within and loss or gain from the structure due to ambient conditions controlled by a surface heat transfer coefficient. Calculated temperatures are output at one-day intervals for all nodes in the model at the particular stage of construction.

Stress/Strain Calculation Program

30. This program calculates the displacements at each node and the strains and stresses developed in each element in the FEM model due

to thermal and gravity loads. When creep is considered, stresses at each time step in the analysis are modified for stress relaxation allowing no strain for the interval up to the next analysis time. The creep parameters are stored and the change in stress stored as residual stress to be included in the next time step analysis. When these stored values are applied during the next time step analysis, strains due to creep are eliminated.

31. Since creep removes those strains due to inelastic deformation, the remaining strain should be completely elastic. In order to determine whether the strains calculated are sufficient to cause cracking they must be compared with a crack threshold strain.

32. The cracking threshold used is the ultimate rapid-load tensile strain capacity. Rapid-load strain capacity tests are conducted at a rate of loading of 40 psi/min which is sufficiently rapid to not allow significant inelastic strains to occur. Thus, elastic tensile strains calculated in the FEM analysis can be compared with tensile strain capacity for the age of the concrete in the element under consideration. If the tensile strain reaches 100 percent of strain capacity, it can be assumed that the cracking has begun.

33. The stress program simulates construction in the same manner as the temperature program for a given problem solution and uses the nodal temperatures calculated to determine thermal loads. The stress program requires time-dependent material properties for each unique material in the model.

34. The input value that instructs the program when to apply temperature changes as volume changes is the stress-free temperature. A value of stress-free temperature is determined for each element by the temperature program at 8 hr after placement and is the value of temperature at which an element is assumed to be stress-free. Stress-free temperatures for an element and calculated nodal temperatures are stored on magnetic tape for subsequent input to the thermal strain analysis

program. Subsequent temperature changes produce volume changes proportional to the coefficient of thermal expansion of an element. When differential volume changes are produced, stresses result. Stresses and strains calculated are also functions of initial external forces or displacements applied as boundary conditions.

PART IV: FINITE-ELEMENT COMPUTER MODELS

25. FEM computer models prepared for this study represent two-dimensional cross-section of structural features of Tennessee-Tombigee Waterway projects. Structural features selected were those typical of Waterway projects, but based upon plans for Aliceville Lock and Dam. The structural features or monoliths modeled were those in which excessive thermal strains may appear during and following construction, where the chosen cross-section will experience heat flow only within the two-dimensional plane, and where the structural analysis can be legitimately described by the assumption of plane strain. The three FEM models prepared represented cross-sections of a gravity dam, a spillway, and a lock wall monolith.

Gravity-Dam Model

36. The gravity-dam model is representative of a major portion of the right abutment wall and of monolith D-2 in particular. Figure 16 shows the general configuration detailing location of 2000- and 3000-psi concrete mixtures, and foundation and concrete lift placement interfaces. The sloping bottom of this monolith was idealized to be horizontal in the FEM model for simplicity. This should not generally affect temperatures or thermal stresses and strains calculated. Figure 17 is the FEM model. Lift interfaces are shown as darker lines.

Spillway Model

37. The spillway monolith model is representative of one half of the cross-section (because of symmetry) taken parallel to the axis of the dam in monoliths D-8 - D-10 of the gated spillway section. The plane of the model is a cross-section midway between the upstream and downstream ends of the monolith. Figure 18 details the geometric configuration and Fig. 19 is the resulting FEM model.

Lock Wall Model

37. The lock wall monolith model is typical of a cross-section taken between filling and emptying ports of monoliths 8L to 16L and 16R to 24R. The general geometry shown in Fig. 20 includes a 4000-psi culvert liner as exists in monoliths 7L and 15R that may not exist on the monoliths noted, but if used would contribute to higher thermal strains in the adjoining lower portion of the structure. The FEM model of the lock wall is shown in Fig. 21.

PART V: COMPUTER INPUT DATA - CONCRETE AND FOUNDATION PROPERTIES

38. Documentation of temperature and thermal strain analysis input data used in the computer simulations are detailed below. This information includes the actual thermal and mechanical properties of concrete and foundation materials in addition to placement parameters and boundary conditions applied in the computer runs. Concrete properties are idealizations of thermal and mechanical data obtained for the 2000-, 3000-, and 4000-psi concrete mixtures. These mixtures proportioned with Type II moderate-heat portland cement without pozzolan replacement and coarse aggregate from the Alabama source are C, A, and B (or B-ATF), respectively. It was assumed that the results of simulations using these concrete data would produce higher or at least equally damaging thermal strains as would simulations using concrete properties from mixtures containing pozzolan replacement (mixture AF) characterized by lower temperature rise and coarse aggregate from the Mississippi source (mixture D) characterized by lower modulus of elasticity and higher creep and strain capacity. This should not be misinterpreted as an endorsement of aggregate from a particular source, but merely as a basis for establishing an upper bound upon which a temperature control plan must be based regardless of the combination of acceptable concrete materials.

39. Concrete and foundation properties required by the temperature and thermal strain calculation programs are as follows:

- a. Temperature calculation - thermal data required:
  - (1) Specific heat.
  - (2) Density (unit weight).
  - (3) Thermal conductivity.
  - (4) Adiabatic temperature rise vs age.
- b. Thermal strain calculations - mechanical properties required:
  - (1) Modulus of elasticity vs age.
  - (2) Poisson's ratio vs age (constant in this study).
  - (3) Coefficient of thermal expansion (constant used).

(4) Rapid-load strain capacity vs age.

(5) Creep vs age (creep vs age of loading vs load duration).

Concrete Thermal Input Properties

40. The value of specific heat used in this study was 0.22 Btu/lb-^{o}F for all concrete mixtures. Unit weights (densities) obtained from concrete mixture designs were 152.1, 152.4, and 146.3 lb/ft^3 for the 2000-, 3000-, and 4000-psi mixtures, respectively. Thermal conductivity was calculated by CRD-C 44 using values of thermal diffusivity, specific heat, and (unit weight) density. Note that diffusivity was not conducted for mixture C; thus, the value used was that of mixture A differing slightly in cement content. Resulting conductivities used were 0.1054 Btu-in./hr-in.^{2}-^{o}F for 2000- and 3000-psi mixtures and 0.0874 Btu-in./hr-in.^{2}-^{o}F for the 4000-psi mixture. Adiabatic temperature rise used for mixtures A, B, and C are found in Fig. 3. All curves are input in digital form.

Concrete Mechanical Input Properties

41. Input values of Poisson's ratio were 0.22, 0.22, and 0.21 for mixtures A, B, and C, respectively. Coefficients of thermal expansion were 5.64-, 5.84-, and 5.76-x10^{-6} in./^{o}F, respectively. The curves actually used for modulus of elasticity and rapid-loading tensile strain capacity with age are shown in Fig. 22 and 23, respectively. The portion of the rapid-load strain capacity curves representing 1-6 days age shown in Fig. 23 is a revision made since the results of the original analysis of this work was submitted in a preliminary report. The revision came as result of additional strain capacity tests conducted for Bay Springs Lock and Dam in which early age properties were more closely examined. The result is that some of the tensile strains found earlier to be excessive are acceptable when measured against the new, higher, early age tensile strain capacity values. The form in which creep data are

input to the FEM thermal strain program is represented by McHenry's equation:[4,5]

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

where σ = applied stress

E_c = creep strain

t = time after placement

T = age at loading

$N = 2$ was found to give a satisfactory fit of experimental data. Values of creep coefficients A1 and A2 versus time given in Fig. 24 represent the average creep of mixtures A, B, and C. Values of constants $m_1 = 0.36$ and $m_2 = 0.145$ were used.

Foundation Properties

42. The following values for properties of the foundation material were used as input to the FEM programs:

a. Thermal conductivity: 0.03024 Btu·in./hr·in.^{2}·°F*
b. Specific heat: 0.45 Btu/lb·°F*
c. Density: 125 lb/ft^3*
d. Elastic modulus: 3000 psi
e. Poisson's ratio: 0.35
f. Coefficient of thermal expansion: 5.5×10^{-6} in./°F

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* Value provided by Mobile District.

PART VI: COMPUTER SIMULATION INPUT - CONSTRUCTION PARAMETERS

Lift Heights

43. Five-ft lift heights were generally used in the FEM models with the following exceptions. Lifts 4-7 of the dam monolith in the vicinity of the gallery were of non-uniform lift height as seen in Fig. 16. Lifts 7 and 8 of the spillway monolith were 5.5 ft each. Lifts 2 and 3 of the lock wall monolith were of non-uniform lift height as seen in Fig. 20. Lift 6 was 4 ft high.

Ambient Temperatures

44. An annual ambient temperature curve (Fig. 25) was derived from monthly mean temperatures for Aberdeen, Mississippi. The ambient exposure temperatures used in all simulations follow this curve for the time of the year during which construction is being simulated. A daily variation in temperature which normally occurs is not simulated.

Construction Start Dates, Concrete Placement Temperature, and Rate of Placement

45. Three construction start dates were used in computer simulations. These dates were 1 April, 1 June, and 1 September. Simulated construction of the lock wall and dam monoliths was made at each start date at a placement rate of five days per lift. Simulated construction of the spillway monolith was made at each construction start date at placement rates of five days per lift in the overflow section (base) and three days per lift in the pier (wall) section. The spillway monolith was also simulated at a constant five days per lift with a 1 June start. Simulated construction of the lock wall monolith was also made for each start date at lift placement rates of 18 days per lift. Lock wall

construction at five days per lift represents the fastest rate possible and 18 days per lift a more probable rate as a result of meeting the 10-ft maximum height difference limit between lock wall monoliths.

46. Concrete placement temperatures were mean daily temperatures at the date of placement within the limits of $40^{o}F$ to $85^{o}F$ specified originally for Aliceville Lock and Dam. The exception to placement at mean daily temperature was during simulated summer construction. In this case placement was at daily mean temperature except between 15 July and 15 August. It was assumed that during this mid-summer period that placement temperatures would reach the $85^{o}F$ limit originally specified. As a result of initial temperature calculations during this investigation, an 80°F placement temperature limit was written into the temperature control plans of the Aliceville and Columbus projects.

Foundation Temperature

47. In order to approximate the foundation temperatures in the models the temperature of the entire foundation was first set equal to $65^{o}F$. This temperature represents the mean annual temperature in the vicinity of the Waterway. It was assumed that the temperature at the bottom of the foundation would remain constant throughout the year. Prior to the beginning of construction simulation, the surface of the foundation was exposed to ambient temperature and calculations begun that equilibrated foundation temperatures between ambient at the surface and the constant temperature at the bottom.

PART VII: COMPUTER SIMULATION RESULTS

48. In this part a discussion of computer simulation results is made for each of the three models studied. Because the computer simulations provided a considerable amount of output, the discussion that follows represents a condensation of these data. Tabulation of computer runs and summarized results are found in Tables 13 and 14.

Gravity Dam Monoliths

49. The result of temperature runs 1-3 produced peak temperatures of 100°F (April start), 108°F (June start), and 103°F (September start), respectively. These temperatures occurred in the 3000-psi concrete surrounding the gallery and on the upstream face near the center of the monolith. Peak temperatures in 2000-psi concrete in this area were approximately 1°F lower. Figures 26-28 show comparative temperature histories in three areas of the structure for computer runs simulating construction beginning in April, June, and September. The thermal strain calculations corresponding to these runs produced maximum tensile strain to strain capacity ratios of 0.65, 0.78, and 0.70 for the three starting dates, respectively. Assuming a ratio near 1.0 to be required to cause cracking to begin, the maximum tensile strains computed are acceptable for the ambient temperature exposure used. The maximum strains were located on the top surface of lift 4 at the gallery floor and in the top surface of lift 6 between the gallery and upstream face.

Spillway Pier Monolith

50. Temperature calculation runs 4-6 produced peak temperatures of 132°F (April start), 140°F (June start), and 118°F (September start), respectively. These temperatures occurred in lifts 11 and 12 located in the pier (wall). Peak temperatures in the overflow section (base) were substantially lower at 100°F, 110°F, and 101°F for the corresponding

start dates. The placement rate of these runs was five days per lift in the overflow section and three days per lift in the pier. Comparative temperature histories of selected areas in the spillway monolith for these runs are found in Fig. 29-31. The lower temperature, more slowly cooling overflow section, and the higher temperature, more rapidly cooling pier, are clearly evident. A fourth run (No. 7) was made with a June start and a five-day per lift placement rate throughout the monolith. Peak temperatures of 137°F were observed in lifts 11 and 12 of the pier. This was 3°F lower than those observed at a placement rate of three days per lift.

51. The corresponding thermal strain runs produced tensile strains in the pier that exceeded the assumed tensile strain capacity at one day after placement. Excessive strains remained through the second day after placement. Areas affected were the top surface of lift 10, the first in which 4000-psi concrete was used, and the vertical surfaces of lifts 11 through 15. Somewhat lower, but excessive strains were observed for placement at five days per lift. Figure 32 shows temperature and resulting vertical normal strain gradients for the three days following placement of lift 11 in the pier. Peak tensile strains occurred on the second day after placement. However, when tensile strains are compared with strain capacity the ratio of strain-to-strain capacity is highest on the first day after placement, is lower, but remains greater than 1.0 on the second day, and becomes less than 1.0 on the third day.

52. It has been learned in inquiries made to staff members at the Mobile District that this area has not indicated observable cracking thus far during construction of Aliceville Lock and Dam. Several reasons may account for the absence of observable cracks in the structure. First, the pier is highly reinforced which may raise crack threshold strain. Note that reinforcement expands or contracts with the concrete. Secondiy, the higher temperatures may cause initially higher tensile strain capacity than the value determined in laboratory tests. Finally, if cracking occurs, the results of these simulations show that tensile strains peak at two days after placement. After three days the concrete begins to

cool. If cracks do occur within the first day or two, cooling of the section will allow surface cracks to close and be difficult to observe several days later. This area of the monolith should be closely monitored within the first several days after placement since potential cracks may be observable for only a short time before closing.

53. Another section of the spillway monolith produced high thermal strains; this was the top of the overflow section (lift 8 in the model) near the bulkhead to the adjoining monolith. A vertical crack running parallel to and approximately five feet from the bulkhead surface may be produced. Cracks in this area of spillway monoliths have been observed elsewhere.

Lock Wall Monolith

54. The lock wall monolith was investigated in two sets of computer runs conducted at the three start dates each. The first set, runs 8-10, simulated a placement rate of five days per lift. Peak temperatures of $102^{\circ}F$ (April start), $119^{\circ}F$ (June start), and $110^{\circ}F$ (September start) occurred around the culvert roof and the mid-section above at node 205 in lift 6. These peak temperatures were a direct result of the use of 4000-psi concrete in the culvert liner as found at station 116.00A, monolith 7L (15R) of Aliceville Locks. Culverts in monoliths 8-17L (16-25R) are constructed entirely of 3000-psi concrete. Peak temperatures in these monoliths would be $7-12^{\circ}F$ less than those shown above and would occur near the center of mass above the culvert. Comparative temperature histories at several points in the monolith are shown in Fig. 33-35. The slower cooling of the massive section above the culvert is evident as is the more rapid cooling of the thinner sections around the culvert and the extreme top (stem) of the monolith.

55. The second series of lock wall computer runs 11-13, simulated placement at 18 days per lift. Peak temperatures were $110^{\circ}F$ (April start), $121^{\circ}F$ (June start), and $95^{\circ}F$ (September start). Even though the placement rate was reduced, the lower half of the monolith during April and June

starts produced higher peak temperatures because of summer placement or summer exposure after spring placement. The slower rate produced lower peak temperatures for the September start since the more massive section was placed during cooler weather in the fall and winter. Figure 36 shows temperature histories for several points in the structure for a June start at the 18 day per lift rate. Note that the placement of the successive lifts causes a second temperature rise. Peak temperatures in runs 11-13 should occur in monoliths 7L and 15R with a 4000-psi concrete culvert liner. Monoliths 8-17L and 16-25R should produce peak temperatures approximately 8-13°F lower because 3000-psi concrete is used exclusively in the culvert area.

56. Throughout these six runs thermal strains produced were all less than tensile strain capacity. Lowest strains were found for construction beginning in April regardless of the placement rate. Highest tensile strains were found in freshly placed concrete for the first three days after placement when construction began in June. Maximum tensile strain was approximately 0.60 of strain capacity on exposed surfaces in the lower portion of the monolith and, near the top of the monolith, these strains are due to the thermal gradient that results from the high initial temperature rise in the interior of a lift relative to the surface. Figure 37 is a typical thermal gradient taken vertically from a horizontal lift surface for the first several days after placement. The steepness of the near-surface gradient remains about constant, however, the temperature difference increases. Thermal strains four days after placement were higher in concrete placed with a September construction start. In this case ambient temperatures are in a downward trend causing a higher rate of cooling at the surface, thus, higher thermal strains. However, by this time, tensile strains were less than 0.45 of tensile strain capacity. In all cases higher thermal strain resulted when placement was made at a rate of five days per lift. Highest strains after four days from placement occurred in thinner sections that cool more rapidly and are restrained by more massive slower cooling sections; this is particularly the case around the culvert area and in the upper stem area.

Other Considerations

57. All thermal strain simulations in this investigation were conducted using the mechanical properties of concrete mixtures A, B, and C that contained coarse aggregate having the higher modulus of elasticity. These mixtures were characterized by higher modulus of elasticity, lower creep, and lower tensile strain capacity than was mixture D that contained lower modulus coarse aggregate. Lower thermal strains will be produced if an aggregate similar to that of mixture D is used in all mixtures required in the structures because the concrete modulus is lower producing lower internal restraint. Relief of thermal strains due to creep will be higher, reducing thermal strains further. Crack-threshold strains will be higher as a result of higher tensile strain capacity with the lower modulus aggregate. The overall result of using aggregate with lower modulus of elasticity is that excessive thermal strains will be minimized.

58. All simulations in this study were made with normal mean daily ambient temperature exposure. This is the ideal exposure condition; the more severe exposure characterized by the passage of cool weather fronts was not included in the scope of this study. However, some mention must be made of the effect of such cool fronts on mass concrete structures. Although the weather in the vicinity of the Tennessee-Tombigbee Waterway is not as severe as in areas further north, the drop in mean daily temperature over the course of several days can exceed 20°F. This was the case in Mississippi early in October 1976 when the mean daily temperature dropped 22°F in a four-day period. It has been found in an investigation[6] of thermal cracking at a gravity dam in Kentucky that similar exposure can cause cracking. A sustained drop in ambient temperature allows near-surface concrete to cool sufficiently to cause intense thermal gradients to occur. The effect is most severe if cold weather occurs within the first several days after placement of a lift or after sustained warm weather. Exposed surfaces of the new concrete experience a thermal gradient increase due to the hydration temperature rise of the internal concrete and simultaneous surface temperature drop as a result of cool

weather. Although the increase in thermal strain for the structure sections investigated here cannot be given without further simulation, it was found in the study noted above that a sustained ambient temperature drop can increase surface tensile strains by over 100 percent. Because of the small size of Waterway structures relative to those of many gravity dams, and provided that mass concrete mixtures used contain at least 25 percent pozzolan replacement with cement content less than 280 lb per yd^3, problems arising from rapid ambient temperature drop should be minimal. Exposed concrete surfaces should be examined during exposure to cool weather since any cracks that may occur will be more readily observable.

59. High tensile strains may occur in other planes than those examined in this investigation. For example, cracks periodically spaced along the top of navigation lock walls constructed in the south are common. It may be that such cracks normal to the axis of the lock occur primarily as a result of thermal strains. In observing the temperature histories of different sections of a lock wall (Fig. 32-34) it can readily be seen that the narrow stem at the top cools faster relative to the lower, more massive section. Thermal strains will occur along the length of the lock wall monoliths because the shrinkage of the rapidly cooling stem is restrained by the slower cooling section below. Cracks[7] will appear at regular intervals dependent upon length to height ratio of the section, the degree of restraint, and the age of the concrete. More cracks at smaller intervals should occur in fresh concrete and decrease in number with age. This principle applies to all areas in which long, thin sections are placed upon sections of larger mass such at the lower lifts of the spillway pier above the overflow section and the upper stem in the gravity monoliths. Close examination of these areas for cracking should be made on the initial Waterway projects especially after cool weather in order that remedial measures may be taken on subsequent construction. It should be noted that cooler weather during the summer may also be suspect. Remedial actions are proposed later in this report.

60. The temperature calculation program cannot account for temperature effect on rate of temperature rise. Tests were conducted for adiabatic temperature rise at 55°F and 80°F initial temperatures producing different initial temperature rise. Only one temperature-rise history could be input per concrete material. Since the range of placement temperature was from 42°F to 85°F and it is assumed that accelerated temperature rise does not occur substantially below 70°F, the data obtained for 55°F initial temperatures were selected to be representative of the greatest portion of the placement-temperature range. This means that temperatures calculated in concrete placed over 70°F are artificially low in the first several days after placement. It is probable that 4000-psi concrete which possesses a higher rate of adiabatic rise at early age due to higher cement content is generally unaffected. However, 2000- and 3000-psi concrete may experience additional temperature rise between 5-8°F in the first several days after placement. This will increase thermal strains in areas where these concretes were placed during the summer months. The increase is estimated to increase initial tensile strains by 20-40 percent causing some strains to exceed strain capacity. Thus, this effect must be considered in the final temperature control plan.

PART VIII: REVIEW AND RECOMMENDATIONS

61. Based upon the results of this investigation, the following paragraphs contain a review of the study and recommendations made toward control of thermal cracking in structures of the Tennessee-Tombigbee Waterway.

62. Tensile strain capacity tests of mass concrete mixtures containing 4-1/2-in. maximum size aggregates obtained from two sources in the vicinity of the Waterway were tested at rapid- and slow-loading rates. Results showed that concrete (mixture D) containing coarse aggregate with the lower modulus of elasticity (7.3×10^6 psi) produced 1.29 times higher rapid-load strain capacity at seven-days age than did concrete of the same proportions (mixture A) produced using aggregate having a higher modulus of elasticity (11.3×10^6 psi). Increase in tensile strain capacity for seven-days age slow-loading tests was 1.23 and 1.89 times that obtained in rapid loading at seven-days age for mixtures A and D, respectively. The strain capacities of rapid-loaded beams tested on the same day that the slow-loaded beams failed represented an increase in strain capacity, slow versus rapid loading, of 0.98 and 1.37 for mixtures A and D, respectively. The results showed both higher rapid-load strain capacities and a greater increase in strain capacity under slow loading for the mixture with the lower modulus of elasticity coarse aggregate.

63. High strength (4000-psi) concrete (mixture B) with 1-1/2-in. nominal maximum size aggregate produced the highest rapid-loading strain capacity as well as the greatest increase, 1.58 times, under slow loading.

64. Creep tests conducted on concrete mixtures A and D showed that mixture D produced approximately 1.30 times the creep that mixture A did in tests begun at seven-days age.

65. Lower strength (3000-psi) concrete (mixture A) without pozzolan and mixture AF with 25 percent pozzolan replacement produced adiabatic temperature rise of 46°F and 39°F, respectively, when the tests began

at 55°F. When tested at 80°F, mixture A resulted in the same adiabatic temperature rise at 28 days, but was 8°F higher at 4-days age.

66. Temperature calculations simulating construction beginning in April, June, and September were conducted on three structural features, i.e., dam, spillway, and lock wall monoliths. Concrete placement was simulated at normal mean daily temperature except for 85°F placement in July and August. Peak temperatures near 140°F were indicated in the lower section of the spillway pier for June construction start. Peak temperature in the dam monolith was 108°F and in the lock wall was 121°F when the culvert was constructed of 4000-psi concrete and approximately 110°F when constructed of 3000-psi concrete.

67. When calculated temperatures were used as input to thermal strain calculations, maximum tensile strain reached 1.39 times that of rapid-load strain capacity at one-day age in the lower portion of the spillway pier. Excessive strain remained through the second day after placement. Maximum tensile strains reached 1.01 times strain capacity on the top surface of the spillway overflow section in the first several days after placement.

68. Maximum tensile strains did not exceed strain capacity in the dam or lock wall monoliths. The maximum ratios of tensile strain versus strain capacity in these monoliths were 0.78 and 0.60, respectively.

69. The results of this study for the most part substantiate the maximum placement temperature of 80°F selected for Aliceville and Columbus Locks and Dams when exposure temperatures are not varying widely. It is recommended, however, that consideration be given to reducing the maximum placement temperatures of 4000-psi concrete by 20°F to 60°F and of 3000-psi concrete by 10°F to 70°F in succeeding waterway projects to protect against rapid ambient temperature drops. This is more strongly recommended if no pozzolan replacement is used. The placement temperatures recommended above should prevent the majority of damaging thermal strains under most exposure conditions. These recommendations can be modified based upon the performance of the Aliceville and Columbus structures that are now well along in construction.

70. It is strongly recommended that all concrete make use of pozzolan replacement up to at least 25 percent by volume, since this is the most beneficial single measure that can be taken to reduce the initial rate of temperature rise and the resulting thermal strains.

71. If design considerations permit, some reduction in cement content especially in the 4000 psi used in the spillway pier would lower excessive strains seen in this study. Reduction in cement content would allow higher placement temperature above the $60^{\circ}F$ recommended in para 69.

72. The piers and overflow surfaces of the spillway monoliths should be closely monitored during the first several days after placement to verify that these areas are not cracking as indicated by the results of this investigation.

73. In order to control transverse, periodic cracking common to the top surfaces of lock wall monoliths, the following procedure is recommended. First this area should be closely monitored after completion of initial lock wall monoliths in the Aliceville and Columbus projects. In the event that cracks are discovered and the placement rate is not greater than 10 days per lift, the topmost three lifts should be insulated at one day after placement of the topmost lift. The conductance of insulation should not be greater than 0.5 Btu/hr-ft^2-$^{\circ}F$ and the insulation should remain in place for a minimum of 45 days. If the placement rate of the top three lifts exceeds 10 days per lift, insulation of these lifts should be considered between placements. The effect of this insulation will be to allow the entire top third of the lock wall to cool at a constant rate.

74. It is suggested that interior and exterior temperatures be continuously monitored for later reference in various areas of the structures especially during the first several days after placement.

75. Use of lower modulus of elasticity coarse aggregate ($<7 \times 10^6$ psi) should produce concrete that is more resistant to thermal cracking. Thus, if a choice exists between aggregates it is probable that coarse aggregates with moduli greater than 7×10^6 psi will be less desirable from the standpoint of thermal strain.

REFERENCES

1. U. S. Army Engineer Waterways Experiment Station, CE, "Handbook for Concrete and Cement," Aug 1949, with quarterly supplements, Vicksburg, Miss.

2. Wilson, E. L., "The Determination of Temperatures Within Mass Concrete Structures," Report No. 68-17, Dec 1968, Structural Engineering Laboratory, University of California, Berkeley, Calif.

3. Sandhu, R. S., Wilson, E. L., and Raphael, J. M., "Two-Dimensional Stress Analysis with Incremental Construction and Creep," Report No. 67-34, Dec 1967, Structural Engineering Laboratory, University of California, Berkeley, Calif.

4. McHenry, D., "A New Aspect of Creep in Concrete and Its Application to Design," Proceedings, American Society for Testing and Materials, Vol 43, 1943, pp 1069-1087.

5. King, I. P., "Finite Element Analysis of Two-Dimensional Time-Dependent Stress Problems," Report No. 65-1, Jan 1965, Structural Engineering Laboratory, University of California, Berkeley, Calif.

6. Bombich, A. A., "Thermal Strain Studies of Cracking in Martins Fork Lake Dam, Kentucky" (in preparation), U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

7. ACI Committee 207, "Effect of Restraint, Volume Change, and Reinforcement on Cracking of Massive Concrete," Journal, American Concrete Institute, July 1973, pp 443-470.

Table 1

Coarse Aggregate Properties - Rock Cores

	Compressive Strength, psi	Modulus of Elasticity $\times 10^6$ psi	Thermal Diffusivity ft^2/hr
CRD G-31 (Alabama source)	30,620*	12.17*	
	22,780*	8.86*	
	30,900*	12.28*	
	23,510*	10.88*	
	15,840*	11.91*	
	30,900*	11.62*	
			0.0514
			0.0548
Average	25,760*	11.29*	0.0531
MOB-39 G-1 (Mississippi source)	12,800	6.59	
	13,730	7.24	
	10,210	8.00	
	11,990	7.50	
			0.0396
			0.0391
Average	12,180	7.33	0.0394

* Results of tests conducted in 1972.

Table 2

Chemical and Physical Properties

of Cements and Pozzolan

Test	MOB-39 C-1	MOB-39 C-2	MOB-39 C-3	MOB-5 AD-1
		Chemical Data		
CaO, %	64.0	64.6	65.7	
SiO_2, %	20.6	22.2	21.9	
Al_2O_3, %	6.7	4.3	4.5	
Fe_2O_3, %	3.2	5.1	3.2	
MgO, %	0.7	0.3	0.9	1.5
SO_3, %	2.5	2.1	2.2	0.3
Loss on ignition, %	1.4	0.4	1.2	3.9
Alkalies - total as Na_2O, %	0.45	0.39	0.42	0.66
Na_2O, %	0.17	0.14	0.40	
K_2O, %	0.42	0.38	0.03	
Insoluble residue, %	0.39	0.12	0.41	
C_3S, %	47.7	52.0	60.1	
C_3A, %	12.4	2.6	6.5	
C_2S, %	23.1	24.6	17.5	
$C_3A + C_3S$, %	60.1	54.6	66.6	
C_4AF, %	9.6	15.6	9.7	
$SiO_2 + R_2O_3$, %				89.1
		Physical Data		
Moisture, %	--	--	--	0.3
Surface area, cm^2/g	3120	3080	3430	

Table 3

Heat of Hydration of Cements

Cement*	Storage Temperature, Degrees F	Heat of Hydration Calories per Gram 1-Day	3-Day	7-Day	28-Day
A	73	45.47	60.20	71.50	83.55
	100	55.56	70.73	80.40	--
	130	64.66	75.32	79.86	--
B	73	--	--	75.68	90.69
C	73	46.45	67.47	80.79	90.78
Blend	73	40.35	53.53	66.61	76.76
	100	53.55	63.72	69.84	--
	130	55.90	66.95	66.82	--

* Cement "A" - Type II, low alkali, with heat of hydration limit (MOB-39 C-2).
Cement "B" - Type I (MOB-39 C-1)
Cement "C" - Type II (MOB-39 C-3)
Blend - Type II (MOB-39 C-3) + 25 percent by volume, fly ash (MOB-5 AD-1)

Table 4

Concrete Mixture Proportions

Material	Saturated Surface Dry Weight, lb	Solid Volume, cu ft	Saturated Surface Dry Weight, lb	Solid Volume, cu ft
	3000-psi with coarse aggregate (CRD-G-31)			
	Mixture A		Mixture AF	
Portland cement (MOB-39 C-2)	282.0	1.435	211.5	1.076
Fly ash (MOB-5 AD-1)	--	--	51.5	0.359
Fine aggregate (MOB-39 S-1)	848.5	5.291	848.5	5.291
Coarse aggregate (#4 - 3/4)	711.0	4.189	711.0	4.189
Coarse aggregate (3/4 - 1-1/2)	623.3	3.686	623.3	3.686
Coarse aggregate (1-1/2 - 3)	564.6	3.351	564.6	3.351
Coarse aggregate (3 - 4-1/2)	928.1	5.529	928.1	5.529
Water	157.44	2.523	157.44	2.523
Air	--	0.996	--	0.996
	4114.94	27.000	4095.94	27.000
	4000-psi with coarse aggregate (CRD-G-31)			
	Mixture B		Mixture B-ATR	
Portland cement (MOB-39 C-2)	517.0	2.630	573.4	2.917
Fine aggregate (MOB-39 S-1)	1084.3	6.761	1036.8	6.465
Coarse aggregate (#4 - 3/4)	1193.4	7.031	1192.7	7.027
Coarse aggregate (3/4 - 1-1/2)	934.3	5.525	933.8	5.522
Water	222.31	3.568	223.63	3.584
Air	--	1.485	--	1.485
	3951.31	27.000	3960.33	27.000
	2000-psi with coarse aggregate (CRD-G-31)			
	Mixture C			
Portland cement (MOB-39 C-2)	244.4	1.243		
Fine aggregate (MOB-39 S-1)	891.3	5.558		
Coarse aggregate (#4 - 3/4)	707.6	4.169		
Coarse aggregate (3/4 - 1-1/2)	620.4	3.669		
Coarse aggregate (1-1/2 - 3)	561.9	3.335		
Coarse aggregate (3 - 4-1/2)	923.9	5.504		
Water	157.44	2.523		
Air	--	0.999		
	4106.94	27.000		

(Continued)

Table 4 (Concluded)

Material	Saturated Surface Dry Weight, lb	Solid Volume, cu ft	Saturated Surface Dry Weight, lb	Solid Volume, cu ft
	3000-psi with coarse aggregate (MOB-39 G-1)			
	Mixture D			
Portland cement (MOB-39 C-2)	282.0	1.435		
Fine aggregate (MOB-39 S-1)	848.5	5.291		
Coarse aggregate (#4 - 3/4)	541.6	3.351		
Coarse aggregate (3/4 - 1-1/2)	806.2	5.028		
Coarse aggregate (1-1/2 - 3)	676.9	4.188		
Coarse aggregate (3 - 4-1/2)	679.5	4.188		
Water	157.44	2.523		
Air	--	0.996		
	3992.14	27.000		

Table 5

Concrete Properties

Mixture No.	Age, Days	Compressive Strength, psi	Splitting Tensile, psi	Elastic Modulus,* psi x 10^6	Poisson's Ratio**
A	3	1630	205	3.69	--
	7	2170	235	4.22	--
	28	3640	430	5.48	0.23
	90	4630	470	6.15†	0.22
	180	4770	480	5.72†	0.22
B	3	2500	260	4.59	--
	7	3280	360	5.32	--
	28	4860	430	5.95	0.22
	90	6410	495	6.78	0.23
C	3	1180	180	3.52	--
	7	2040	245	3.92	--
	28	3480	400	5.00	0.21
	90	4720	455	5.18	0.21
D	3	1580	170	3.01	0.16
	7	2140	270	3.22	0.16
	28	3830	405	3.88	0.18
	90	4880	490	4.24	0.19
	180	5610	510	4.48	0.16

* Secant modulus - 50 percent ultimate strength.

** Determined at 50 percent ultimate strength.

† Examination of the individual data items does not show any reason for the apparent loss in E between 90 and 180 days.

TABLE 6

SPECIFIC CREEP 7-DAYS LOADING

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SPECIMEN MIXTURE A

FUNCTION	INDEX OF DETERMINATION	A	B
Y=A+B*ALOG(X)	0.98268121	0.37466E-01	0.42549E-01
Y=A*X**B	0.96084510	0.70807E-01	0.26319E 00

****

I	DATA POINTS X	DATA POINTS Y	CALCULATED Y FROM Y=A+B*ALOG(X)	CALCULATED Y FROM Y=A*X**B
1	0.18000E 01	0.75000E-01	0.68476E-01	0.82654E-01
2	0.28300E 01	0.84000E-01	0.81279E-01	0.92347E-01
3	0.58000E 01	0.10400E 00	0.11226E 00	0.11248E 00
4	0.70000E 01	0.11300E 00	0.12026E 00	0.11317E 00
5	0.80000E 01	0.11600E 00	0.12534E 00	0.12240E 00
6	0.90000E 01	0.12100E 00	0.13076E 00	0.12035E 00
7	0.10000E 02	0.13000E 00	0.13544E 00	0.12860E 00
8	0.15000E 02	0.14600E 00	0.15267E 00	0.14442E 00
9	0.16000E 02	0.15100E 00	0.15544E 00	0.14659E 00
10	0.17000E 02	0.15500E 00	0.15702E 00	0.14925E 00
11	0.21000E 02	0.16400E 00	0.16701E 00	0.15779E 00
12	0.23000E 02	0.17600E 00	0.17059E 00	0.16161E 00
13	0.27000E 02	0.18100E 00	0.17770E 00	0.16653E 00
14	0.28000E 02	0.18200E 00	0.17925E 00	0.17020E 00
15	0.29000E 02	0.18400E 00	0.18074E 00	0.17170E 00
16	0.32000E 02	0.18800E 00	0.18216E 00	0.17333E 00
17	0.31200E 02	0.18700E 00	0.18358E 00	0.17482E 00
18	0.34200E 02	0.19600E 00	0.18751E 00	0.17912E 00
19	0.36000E 02	0.19400E 00	0.18874E 00	0.18124E 00
20	0.42000E 02	0.22422E 00	0.19570E 00	0.18937E 00
21	0.49000E 02	0.19300E 00	0.22218E 00	0.19614E 00
22	0.51000E 02	0.20700E 00	0.22476E 00	0.19727E 00
23	0.56000E 02	0.21600E 00	0.20874E 00	0.20432E 00
24	0.62000E 02	0.22100E 00	0.21307E 00	0.20961E 00
25	0.67200E 02	0.22200E 00	0.21637E 00	0.21413E 00
26	0.78000E 02	0.23600E 00	0.22234E 00	0.22287E 00
27	0.87000E 02	0.23000E 00	0.22749E 00	0.22737E 00
28	0.94000E 02	0.23600E 00	0.23076E 00	0.23403E 00
29	0.10400E 03	0.23400E 00	0.23523E 00	0.24041E 00
30	0.11100E 03	0.24500E 00	0.23785E 00	0.24456E 00
31	0.11800E 03	0.23400E 00	0.24046E 00	0.24853E 00
32	0.12500E 03	0.23300E 00	0.24291E 00	0.25233E 00
33	0.13200E 03	0.24600E 00	0.24523E 00	0.25597E 00
34	0.14100E 03	0.24500E 00	0.24803E 00	0.26046E 00
35	0.14600E 03	0.24000E 00	0.24951E 00	0.26256E 00
36	0.15300E 03	0.24600E 00	0.25151E 00	0.26612E 00
37	0.16700E 03	0.26100E 00	0.25523E 00	0.27232E 00
38	0.18200E 03	0.24900E 00	0.25889E 00	0.27855E 00

*

TABLE 7

SPECIFIC CREEP 7-DAYS LOADING

....

SPECIMEN MIXTURE B

FUNCTION	INDEX OF DETERMINATION	A	B
Y=A+B*ALOG(X)	0.99310814	0.28112E-01	0.32312E-01
Y=A*X**B	0.94812757	0.46571E-01	0.28839E 00

****

I	DATA POINTS X	DATA POINTS Y	CALCULATED Y FROM Y=A+B*ALOG(X)	CALCULATED Y FROM Y=A*X**B
1	0.17000E 01	0.40000E-01	0.39258E-01	0.54272E-01
2	0.30000E 01	0.64300E-01	0.57610E-01	0.63931E-01
3	0.70000E 01	0.81000E-01	0.84099E-01	0.81627E-01
4	0.80000E 01	0.91000E-01	0.93103E-01	0.87762E-01
5	0.13000E 02	0.10300E 00	0.10423E 00	0.97561E-01
6	0.14000E 02	0.10300E 00	0.10735E 00	0.99602E-01
7	0.15000E 02	0.10400E 00	0.10961E 00	0.10163E 00
8	0.20000E 02	0.11900E 00	0.11691E 00	0.11045E 00
9	0.21000E 02	0.12200E 00	0.12045E 00	0.11205E 00
10	0.22000E 02	0.12300E 00	0.12199E 00	0.11357E 00
11	0.23000E 02	0.13300E 00	0.12976E 00	0.12175E 00
12	0.34000E 02	0.13400E 00	0.13605E 00	0.12676E 00
13	0.35000E 02	0.13800E 00	0.13929E 00	0.12994E 00
14	0.36000E 02	0.13900E 00	0.13729E 00	0.13052E 00
15	0.42000E 02	0.14300E 00	[illegible]	0.13685E 00
16	0.48000E 02	0.15000E 00	0.14720E 00	0.14223E 00
17	0.51000E 02	0.14600E 00	0.14916E 00	0.14473E 00
18	0.56000E 02	0.15200E 00	0.15212E 00	0.14865E 00
19	0.62000E 02	0.15500E 00	0.15547E 00	0.15312E 00
20	0.67000E 02	0.15400E 00	0.15833E 00	0.15731E 00
21	0.77000E 02	0.16300E 00	0.16247E 00	0.16299E 00
22	0.84000E 02	0.17600E 00	0.16528E 00	0.16713E 00
23	0.91000E 02	0.17200E 00	0.16787E 00	0.17103E 00
24	0.93000E 02	0.17300E 00	0.17026E 00	0.17473E 00
25	0.10300E 03	0.17300E 00	0.17340E 00	0.17865E 00
26	0.11800E 03	0.17800E 00	0.17626E 00	0.18434E 00
27	0.12700E 03	0.17500E 00	0.17864E 00	0.18823E 00
28	0.13900E 03	0.17400E 00	0.18155E 00	0.19326E 00
29	0.15300E 03	0.18400E 00	0.18465E 00	0.19863E 00
30	0.16800E 03	0.18600E 00	0.18788E 00	0.20411E 00
31	0.18100E 03	0.19000E 00	0.19002E 00	0.20855E 00

*

TABLE 8

SPECIFIC CREEP 7-DAYS LOADING

....

SPECIMEN MIXTURE C

FUNCTION	INDEX OF DETERMINATION	A	B
Y=A+B*ALOG(X)	0.99152273	0.37759E-01	0.44373E-01
Y=A*X**B	0.97130156	0.76185E-01	0.25457E 00

****

I	DATA POINTS X	DATA POINTS Y	CALCULATED Y FROM Y=A+B*ALOG(X)	CALCULATED Y FROM Y=A*X**B
1	0.19000E 01	0.78000E-01	0.66249E-01	0.89708E-01
2	0.58000E 01	0.11300E 00	0.11576E 00	0.11918E 00
3	0.70000E 01	0.12500E 00	0.12410E 00	0.12503E 00
4	0.90000E 01	0.12500E 00	0.13526E 00	0.13329E 00
5	0.10000E 02	0.13400E 00	0.13993E 00	0.13691E 00
6	0.13000E 02	0.14900E 00	0.15157E 00	0.14637E 00
7	0.14000E 02	0.15500E 00	0.15486E 00	0.14916E 00
8	0.15000E 02	0.15400E 00	0.15792E 00	0.15180E 00
9	0.16000E 02	0.15500E 00	0.16079E 00	0.15431E 00
10	0.20000E 02	0.16700E 00	0.17069E 00	0.16333E 00
11	0.21000E 02	0.17600E 00	0.17285E 00	0.16537E 00
12	0.23000E 02	0.17900E 00	0.17689E 00	0.16925E 00
13	0.27000E 02	0.18400E 00	0.18400E 00	0.17633E 00
14	0.28000E 02	0.18500E 00	0.18562E 00	0.17794E 00
15	0.29000E 02	0.19100E 00	0.18717E 00	0.17954E 00
16	0.30000E 02	0.18300E 00	0.18868E 00	0.18102E 00
17	0.35000E 02	0.19200E 00	0.19552E 00	0.18834E 00
18	0.36000E 02	0.19300E 00	0.19677E 00	0.18970E 00
19	0.43000E 02	0.20300E 00	0.20465E 00	0.19847E 00
20	0.55000E 02	0.21500E 00	0.21556E 00	0.21131E 00
21	0.64000E 02	0.21900E 00	0.22230E 00	0.21962E 00
22	0.72000E 02	0.23400E 00	0.22753E 00	0.22630E 00
23	0.78000E 02	0.23300E 00	0.23108E 00	0.23096E 00
24	0.87000E 02	0.24600E 00	0.23592E 00	0.23747E 00
25	0.94000E 02	0.24300E 00	0.23936E 00	0.24220E 00
26	0.10700E 03	0.24800E 00	0.24511E 00	0.25232E 00
27	0.12000E 03	0.25100E 00	0.25019E 00	0.25773E 00
28	0.13200E 03	0.25100E 00	0.25442E 00	0.26406E 00
29	0.14600E 03	0.25700E 00	0.25890E 00	0.27093E 00
30	0.16100E 03	0.26000E 00	0.26323E 00	0.27775E 00
31	0.17400E 03	0.26400E 00	0.26665E 00	0.28330E 00
32	0.18100E 03	0.26600E 00	0.26843E 00	0.28616E 00

*

TABLE 9

SPECIFIC CREEP 7-DAYS LOADING

....

SPECIMEN MIXTURE D

FUNCTION	INDEX OF DETERMINATION	A	B
Y=A+B*ALOG(X)	0.99018292	0.56706E-01	0.47838E-01
Y=A*X**B	0.97284669	0.91238E-01	0.24557E 00

****

I	DATA POINTS X	DATA POINTS Y	CALCULATED Y FROM Y=A+B*ALOG(X)	CALCULATED Y FROM Y=A*X**B
1	0.18000E 01	0.94000E-01	0.84825E-01	0.10541E 00
2	0.25000E 01	0.11200E 00	0.10596E 00	0.11740E 00
3	0.35000E 01	0.11900E 00	0.12057E 00	0.12664E 00
4	0.70000E 01	0.13900E 00	0.14370E 00	0.14713E 00
5	0.80000E 01	0.14900E 00	0.15618E 00	0.15204E 00
6	0.90000E 01	0.15700E 00	0.16122E 00	0.15650E 00
7	0.10000E 02	0.16300E 00	0.16626E 00	0.16062E 00
8	0.11000E 02	0.16700E 00	0.17140E 00	0.16441E 00
9	0.15000E 02	0.18400E 00	0.18025E 00	0.17742E 00
10	0.16000E 02	0.19000E 00	0.18734E 00	0.18025E 00
11	0.17000E 02	0.19800E 00	0.19224E 00	0.18255E 00
12	0.18000E 02	0.19100E 00	0.19478E 00	0.18554E 00
13	0.22000E 02	0.20100E 00	0.20459E 00	0.19491E 00
14	0.24000E 02	0.22700E 00	0.20574E 00	0.19912E 00
15	0.28000E 02	0.21900E 00	0.21611E 00	0.20681E 00
16	0.31000E 02	0.22800E 00	0.22223E 00	0.21234E 00
17	0.35000E 02	0.24200E 00	0.22670E 00	0.21345E 00
18	0.37000E 02	0.23900E 00	0.22745E 00	0.22146E 00
19	0.42000E 02	0.24200E 00	0.23351E 00	0.22346E 00
20	0.46000E 02	0.24500E 00	0.23976E 00	0.23362E 00
21	0.51000E 02	0.24500E 00	0.24430E 00	0.23961E 00
22	0.56000E 02	0.24900E 00	0.24927E 00	0.24519E 00
23	0.62000E 02	0.25100E 00	0.25257E 00	0.24937E 00
24	0.67000E 02	0.26000E 00	0.25725E 00	0.25622E 00
25	0.72000E 02	0.26300E 00	0.26122E 00	0.26075E 00
26	0.77000E 02	0.26300E 00	0.26451E 00	0.26512E 00
27	0.87000E 02	0.27000E 00	0.27035E 00	0.27319E 00
28	0.95000E 02	0.27600E 00	0.27456E 00	0.27916E 00
29	0.10200E 03	0.27600E 00	0.27790E 00	0.28406E 00
30	0.10900E 03	0.28100E 00	0.28113E 00	0.28875E 00
31	0.11600E 03	0.28200E 00	0.28411E 00	0.29319E 00
32	0.12300E 03	0.28100E 00	0.28691E 00	0.29744E 00
33	0.13300E 03	0.28100E 00	0.29065E 00	0.30321E 00
34	0.14700E 03	0.29800E 00	0.29544E 00	0.31075E 00
35	0.17100E 03	0.29900E 00	0.30267E 00	0.32251E 00
36	0.18200E 03	0.31900E 00	0.30666E 00	0.32748E 00

*

TABLE 10

SPECIFIC CREEP 90-DAYS LOADING

....

SPECIMEN MIXTURE A

FUNCTION	INDEX OF DETERMINATION	A	B
Y=A+B*ALOG(X)	0.96768696	0.50032E-01	0.27068E-01
Y=A*X**B	0.99620928	0.69444E-01	0.20135E 00

****

I	DATA POINTS X	Y	CALCULATED Y FROM Y=A+B*ALOG(X)	Y=A*X**B
1	0.18000E 01	0.80000E-01	0.65349E-01	0.78163E-01
2	0.28000E 01	0.88000E-01	0.77209E-01	0.85442E-01
3	0.35000E 01	0.91000E-01	0.85175E-01	0.90961E-01
4	0.49000E 01	0.95000E-01	0.92493E-01	0.95237E-01
5	0.58000E 01	0.10100E 00	0.97621E-01	0.98936E-01
6	0.72000E 01	0.12200E 00	0.10271E 00	0.10275E 00
7	0.10000E 02	0.11800E 00	0.11237E 00	0.11040E 00
8	0.11000E 02	0.11280E 00	0.11495E 00	0.11254E 00
9	0.13000E 02	0.11600E 00	0.11947E 00	0.11639E 00
10	0.14000E 02	0.11830E 00	0.12147E 00	0.11814E 00
11	0.17000E 02	0.12300E 00	0.12673E 00	0.12235E 00
12	0.18000E 02	0.12300E 00	0.12828E 00	0.12428E 00
13	0.19000E 02	0.12480E 00	0.12974E 00	0.12564E 00
14	0.24000E 02	0.13200E 00	0.13606E 00	0.13163E 00
15	0.25000E 02	0.13030E 00	0.13717E 00	0.13277E 00
16	0.28000E 02	0.13500E 00	0.14024E 00	0.13594E 00
17	0.33000E 02	0.13800E 00	0.14463E 00	0.14041E 00
18	0.38000E 02	0.14200E 00	0.14850E 00	0.14445E 00
19	0.42000E 02	0.14600E 00	0.15121E 00	0.14735E 00
20	0.49000E 02	0.15200E 00	0.15533E 00	0.15204E 00
21	0.60000E 02	0.15600E 00	0.16083E 00	0.15837E 00
22	0.66000E 02	0.15800E 00	0.16344E 00	0.16144E 00
23	0.73000E 02	0.16200E 00	0.16617E 00	0.16475E 00
24	0.80000E 02	0.16700E 00	0.16865E 00	0.16781E 00
25	0.87000E 02	0.17300E 00	0.17092E 00	0.17067E 00
26	0.10200E 03	0.17700E 00	0.17523E 00	0.17623E 00
27	0.11500E 03	0.17800E 00	0.17845E 00	0.18034E 00
28	0.12700E 03	0.18600E 00	0.18153E 00	0.18476E 00
29	0.13300E 03	0.19000E 00	0.18341E 00	0.18723E 00
30	0.15400E 03	0.19600E 00	0.18638E 00	0.19147E 00
31	0.16400E 03	0.20100E 00	0.18806E 00	0.19391E 00
32	0.17400E 03	0.20200E 00	0.18968E 00	0.19623E 00

*

TABLE 11

SPECIFIC CREEP 28-DAYS LOADING

....

SPECIMEN MIXTURE D

FUNCTION	INDEX OF DETERMINATION	A	B
Y=A+B*ALOG(X)	0.97697848	0.20634E-01	0.40047E-01
Y=A*X**B	0.99337507	0.55805E-01	0.28433E 00

****

I	DATA POINTS X	Y	CALCULATED Y FROM Y=A+B*ALOG(X)	Y=A*X**B
1	0.18000E 01	0.64000E-01	0.44173E-01	0.65958E-01
2	0.29000E 01	0.72000E-01	0.63272E-01	0.75539E-01
3	0.70000E 01	0.94000E-01	0.95561E-01	0.97051E-01
4	0.10000E 02	0.10300E 00	0.11234E 00	0.10741E 00
5	0.14000E 02	0.12200E 00	0.12632E 00	0.11822E 00
6	0.16000E 02	0.12800E 00	0.13167E 00	0.12277E 00
7	0.21000E 02	0.13600E 00	0.14256E 00	0.13264E 00
8	0.25000E 02	0.14500E 00	0.14954E 00	0.13932E 00
9	0.30000E 02	0.14600E 00	0.15634E 00	0.14697E 00
10	0.35000E 02	0.15400E 00	0.16301E 00	0.15335E 00
11	0.37000E 02	0.15400E 00	0.16735E 00	0.15218E 00
12	0.46000E 02	0.16700E 00	0.17376E 00	0.16575E 00
13	0.51000E 02	0.17300E 00	0.17829E 00	0.17072E 00
14	0.56200E 02	0.17600E 00	0.18194E 00	0.17532E 00
15	0.66000E 02	0.18300E 00	0.18842E 00	0.18378E 00
16	0.74000E 02	0.18700E 00	0.19300E 00	0.18978E 00
17	0.81000E 02	0.19300E 00	0.19662E 00	0.19472E 00
18	0.88000E 02	0.20300E 00	0.19974E 00	0.19936E 00
19	0.95000E 02	0.22400E 00	0.20300E 00	0.20375E 00
20	0.10200E 03	0.20600E 00	0.20525E 00	0.20791E 00
21	0.11200E 03	0.20400E 00	0.20955E 00	0.21352E 00
22	0.12600E 03	0.21300E 00	0.21431E 00	0.22075E 00
23	0.14000E 03	0.22700E 00	0.21853E 00	0.22750E 00
24	0.15000E 03	0.22920E 00	0.22120E 00	0.23201E 00
25	0.16100E 03	0.23700E 00	0.22413E 00	0.23673E 00
26	0.16800E 03	0.23100E 00	0.22583E 00	0.23961E 00
27	0.17500E 03	0.23600E 00	0.22747E 00	0.24241E 00

*

Table 12

Strain Capacity Tests

Mixture No.	Beam No.	Testing Age, Days	Loading Rate	Stress Capacity,* psi	Strain Capacity,** millionths	
					Tensile	Compressive
A	4	7	40 psi/min	225	52	47
	3	7-139	25 psi/week	425	64	86
	6	139	40 psi/min	295	65	58
	5	90	40 psi/min	280	62	64
	2	90-241	25 psi/week	420	71	92
	1	241	40 psi/min	495	54	59
B	2	7	40 psi/min	280	75	74
	3	7-178	25 psi/week	560	103	142
	1	178	40 psi/min	495	65	71
C	2	7	40 psi/min	140	53	50
	1	7-134	25 psi/week	425	89	100
	3	134	40 psi/min	405	57	62
D	5	7	40 psi/min	220	55	51
	2	7-133	25 psi/week	425	104	104
	6	133	40 psi/min	390	76	76
	3	28	40 psi/min	320	67	61
	4	28-134	25 psi/week	360	78	90
	1	134	40 psi/min	425	74	70

* Determined at 90 percent of ultimate load.

** Extrapolated outside fiber strain at 90 percent of ultimate load.

Table 13

Summary of Temperature and Thermal Strain Calculations

Run No.	Model Type	Start Date	Placement Duration (days)	Placement Temperature* (°F)	Placement Rate Days/Lift	Placement Rate Lifts	Peak Temperature, Node No., and Day °F	No.**	Day†	Tensile Strain/Strain Capacity Ratio, Maximum ε/S.C. Ratio	Ele. No	Day†
1	Dam	1 Apr	100	Ambient	5	1-14	100	321	55	0.70	230	33
2	Dam	1 Jun	100	Mod. Ambient	5	1-14	108	257	43	0.78	195	28
3	Dam	1 Sep	100	Ambient	5	1-4	104	122	22	0.72	230	33
4	Spillway Monolith	1 Apr	90	Ambient	5 3	1-8 9-15	100 132	203 298	42 57	0.95 1.35	240 272	43 49
5	Spillway	1 Jun	90	Mod. Ambient	5 3	1-8 9-15	110 140	203 303	41 55	1.01 1.39	240 272	43 49
6	Spillway	1 Sep	90	Ambient	5 3	1-8 9-15	101 118	89 298	38 51	0.97 1.37	240 272	43 49
7	Spillway	1 Jun	100	Mod. Ambient	5	1-15	137	303	55	1.34	272	53
8	Lock wall	1 Apr	100	Ambient	5	1-14	102	124	19	0.47	148	23
9	Lock wall	1 Jun	100	Mod. Ambient	5	1-14	119	150	24	0.55	148	23
10	Lock wall	1 Sep	100	Ambient	5	1-14	110	105	14	0.49	148	23
11	Lock wall	1 Apr	252	Mod. Ambient	18	1-14	110	150	75	0.46	148	34
12	Lock wall	1 Jun	252	Mod. Ambient	18	1-14	121	150	76	0.48	148	34
13	Lock wall	1 Sep	252	Ambient	18	1-14	95	44	4	0.46	148	34

* Modified ambient (Mod. Ambient) placement temperature curve during July and August in which peak placement temperature (85°F) is several degrees F above ambient temperature.
** Position of nodes and elements found in Figures 17, 19, and 21.
† Elapsed time (days) into the computer simulation.

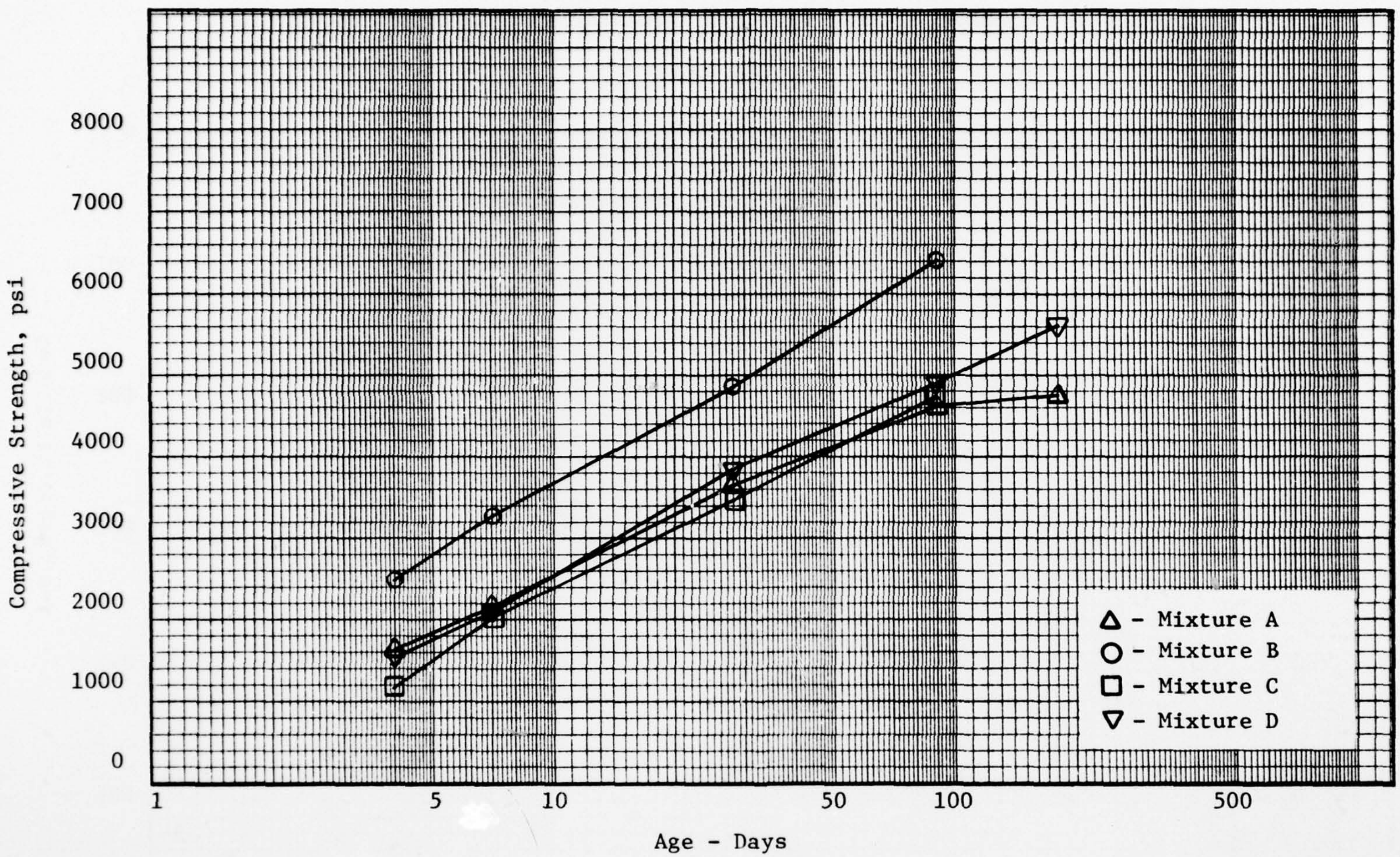

Figure 1. Compressive strength gain of concrete

WES FORM NO. 1780
JULY 1968

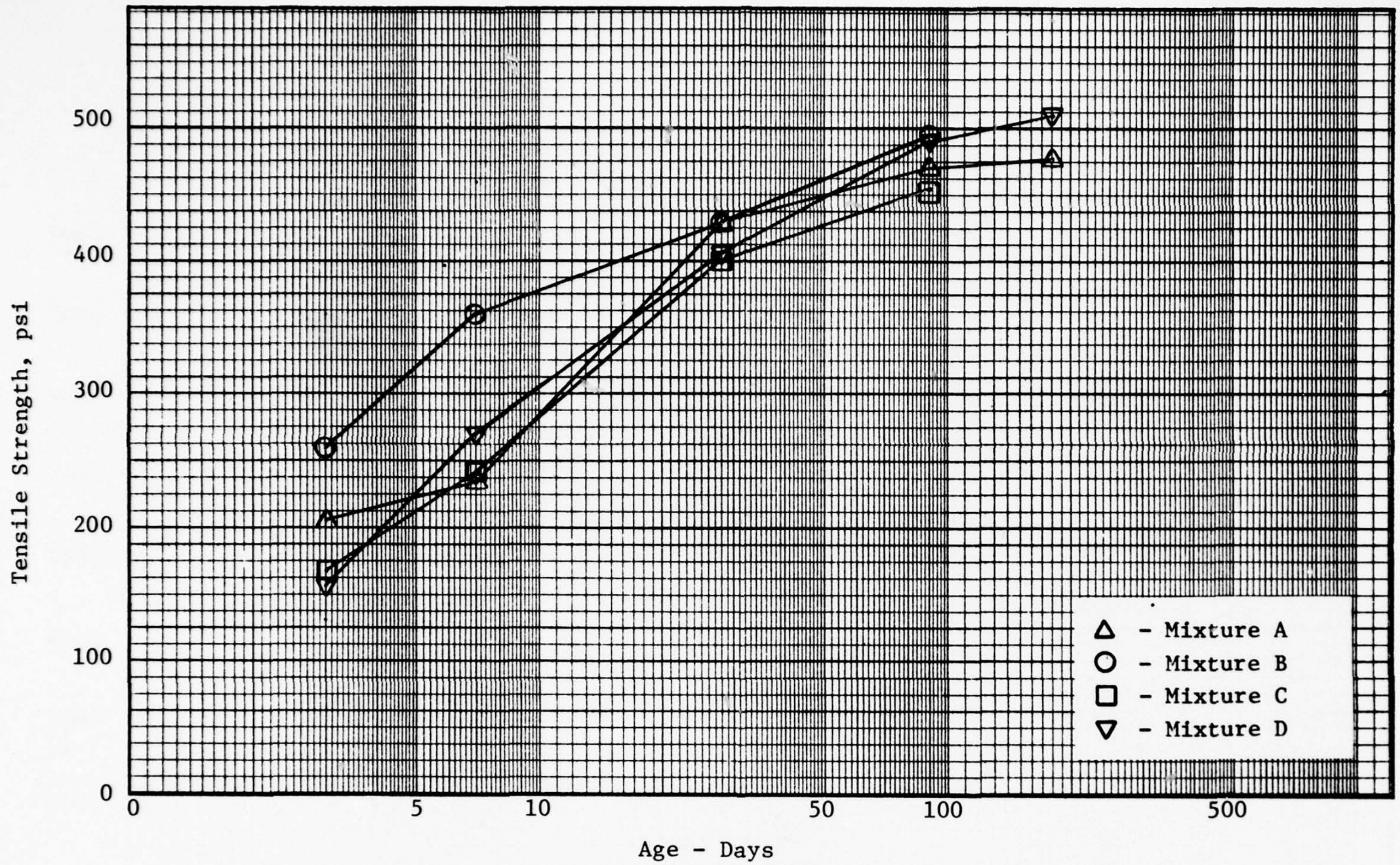

Figure 2. Compressive strength gain of concrete

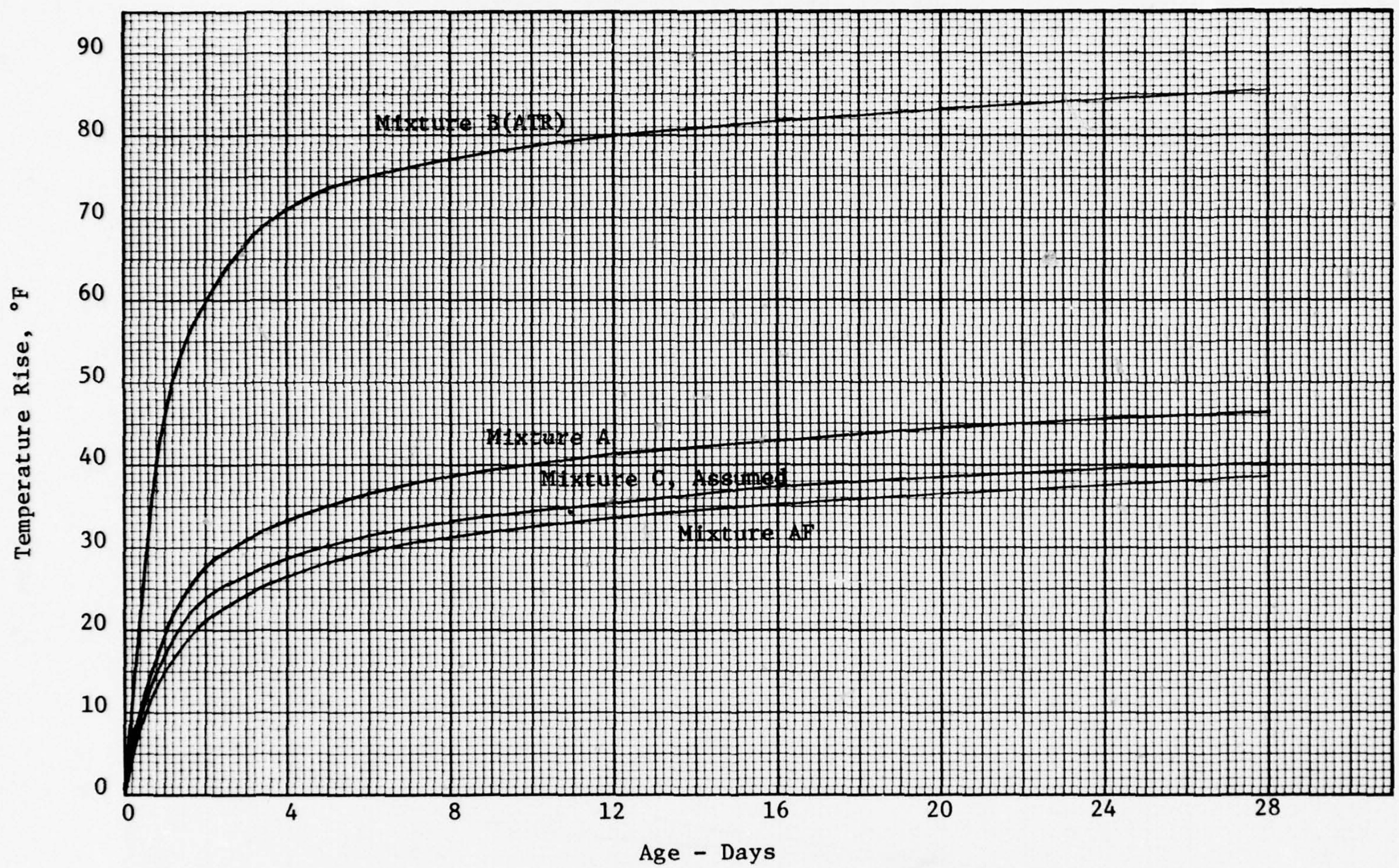

Figure 3. Adiabatic temperature rise--initial temperature 55°F

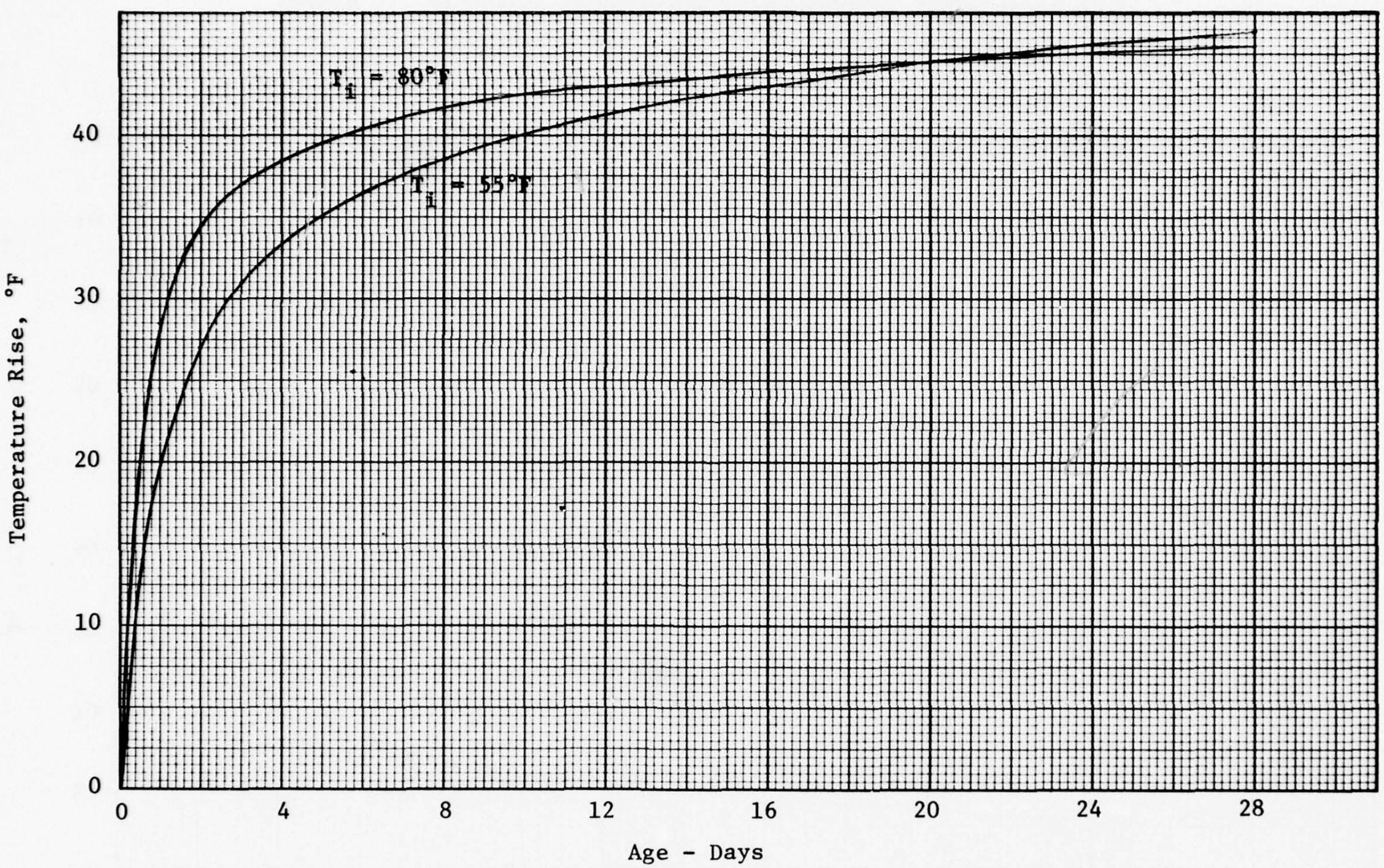

Figure 4. Adiabatic temperature rise of Mixture A at initial temperatures of 55°F and 80°F

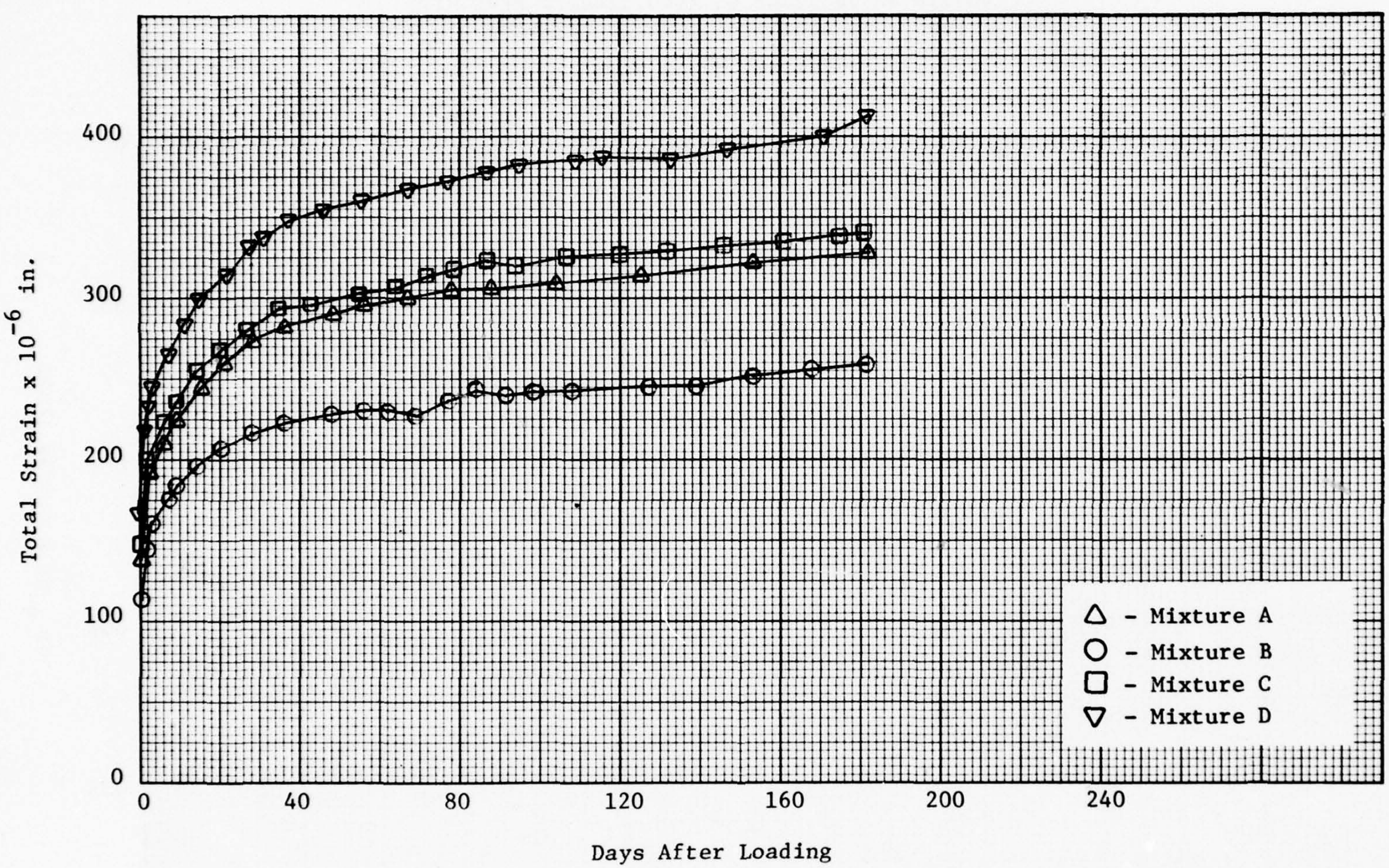

Figure 5. Uniaxial creep of concrete at 7-days age

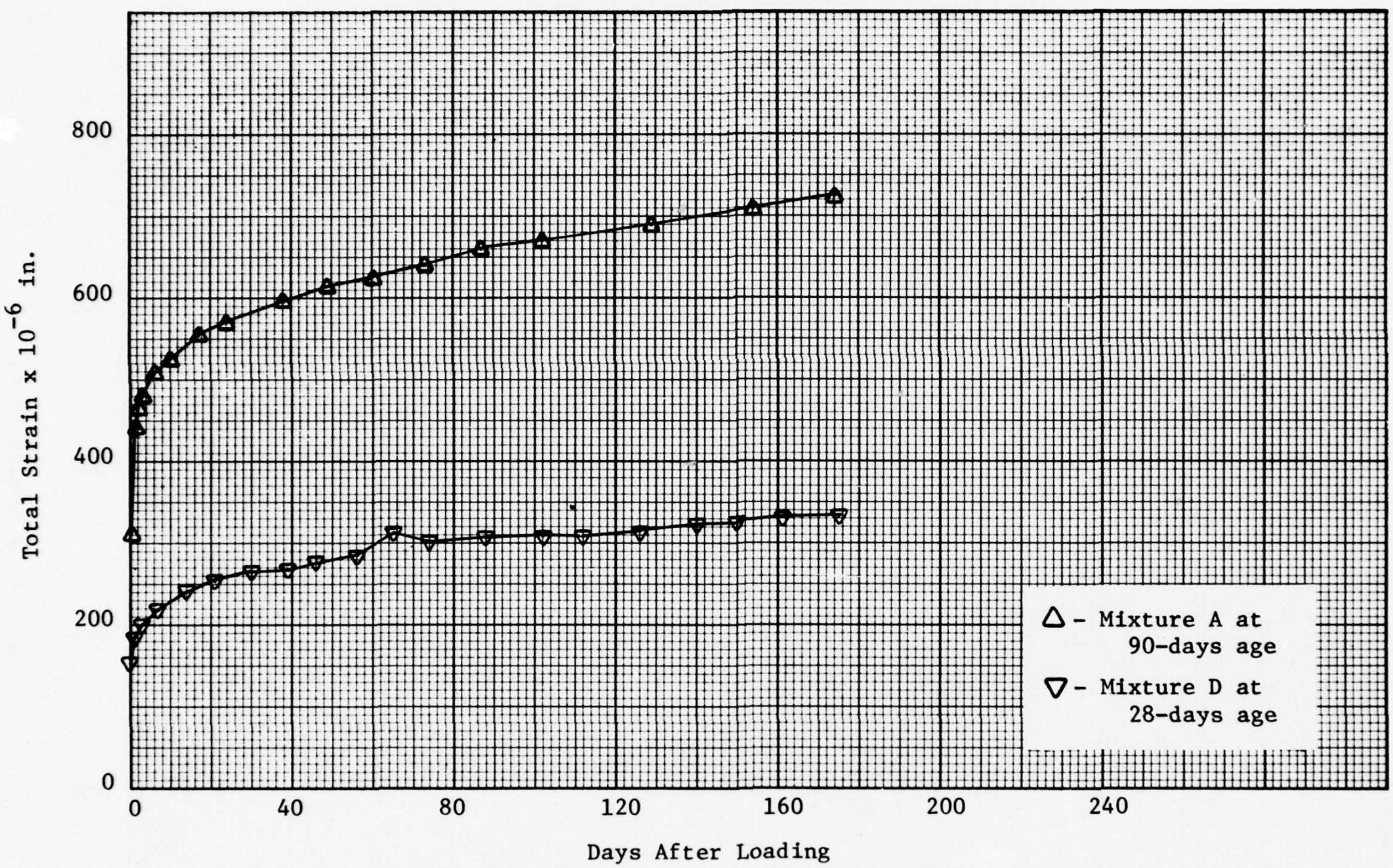

Figure 6. Uniaxial creep of concrete at 28 and 90 days with load proportional to compressive strength

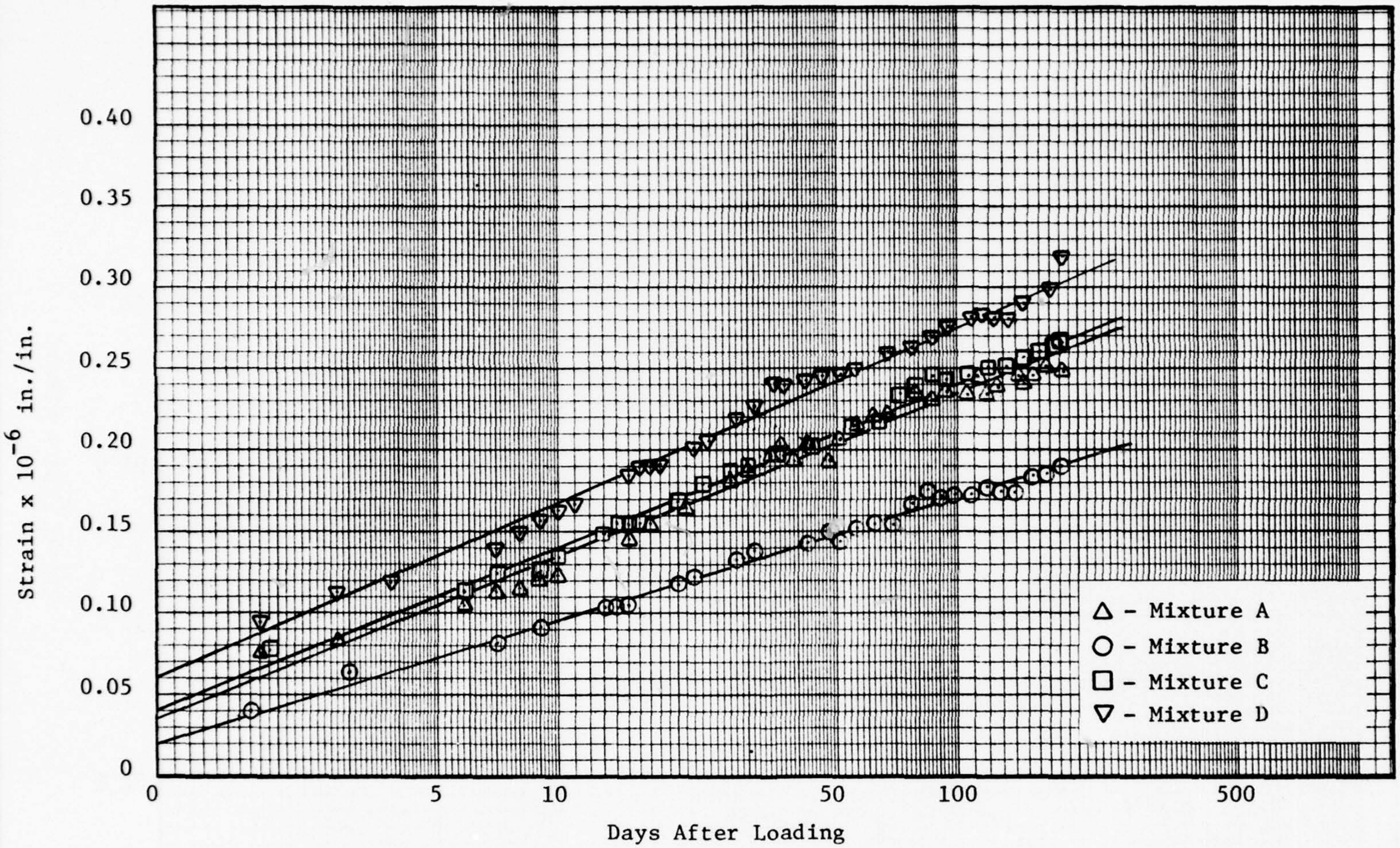

Figure 7. Specific creep of concrete at 7-days age

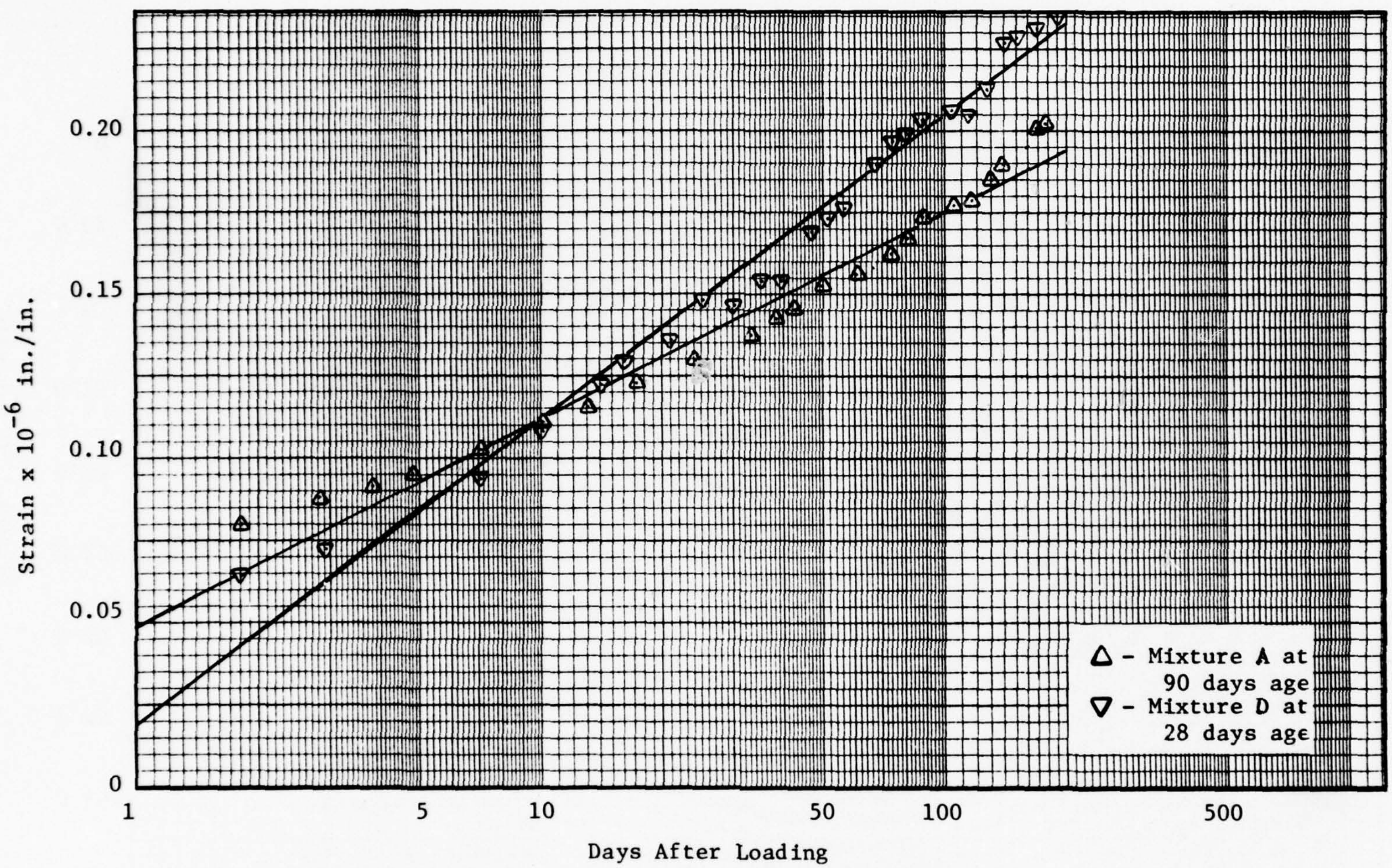

Figure 8. Specific creep of concrete at 28 and 90 days age

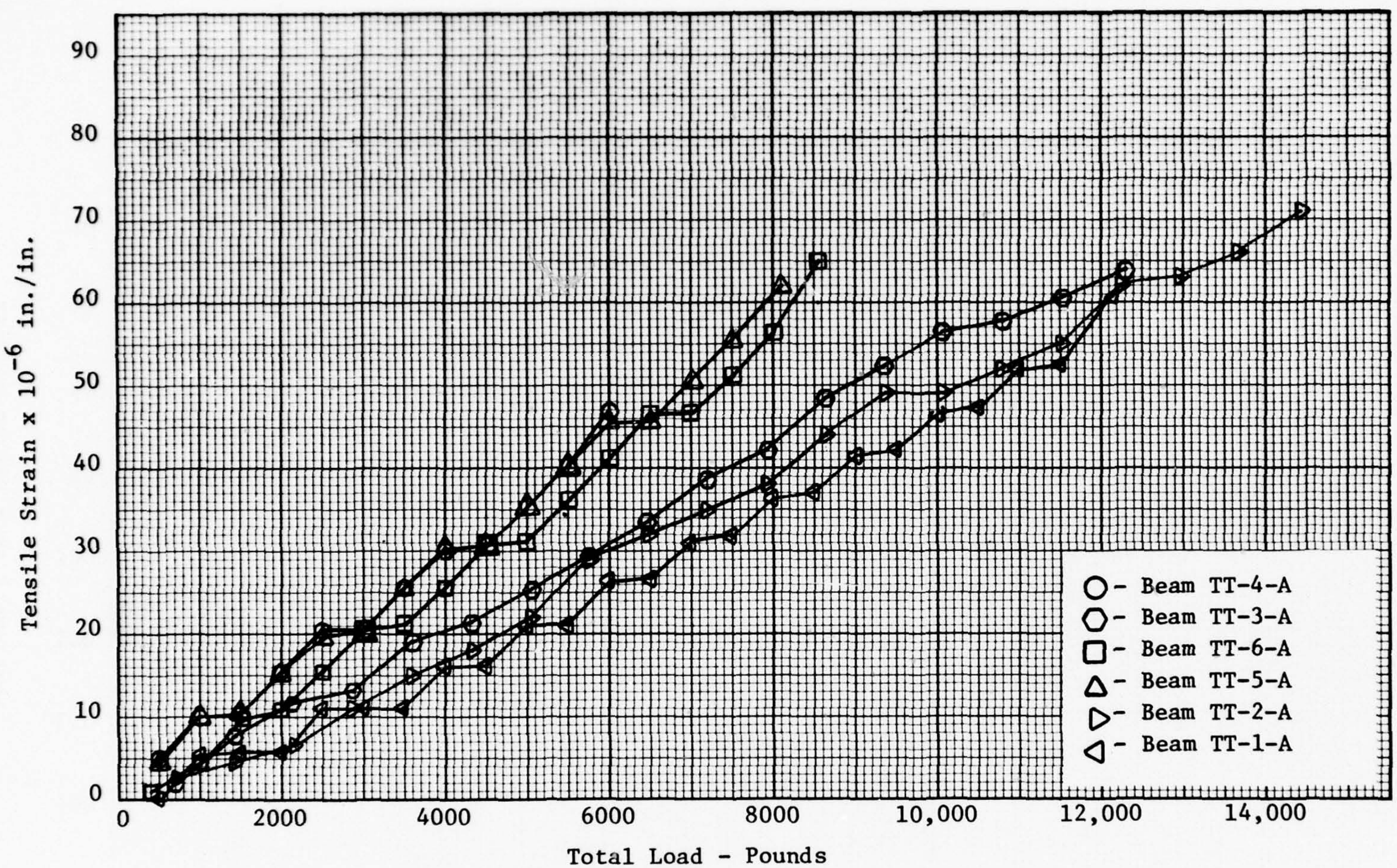

Figure 9. Tensile strain capacity - Mixture A

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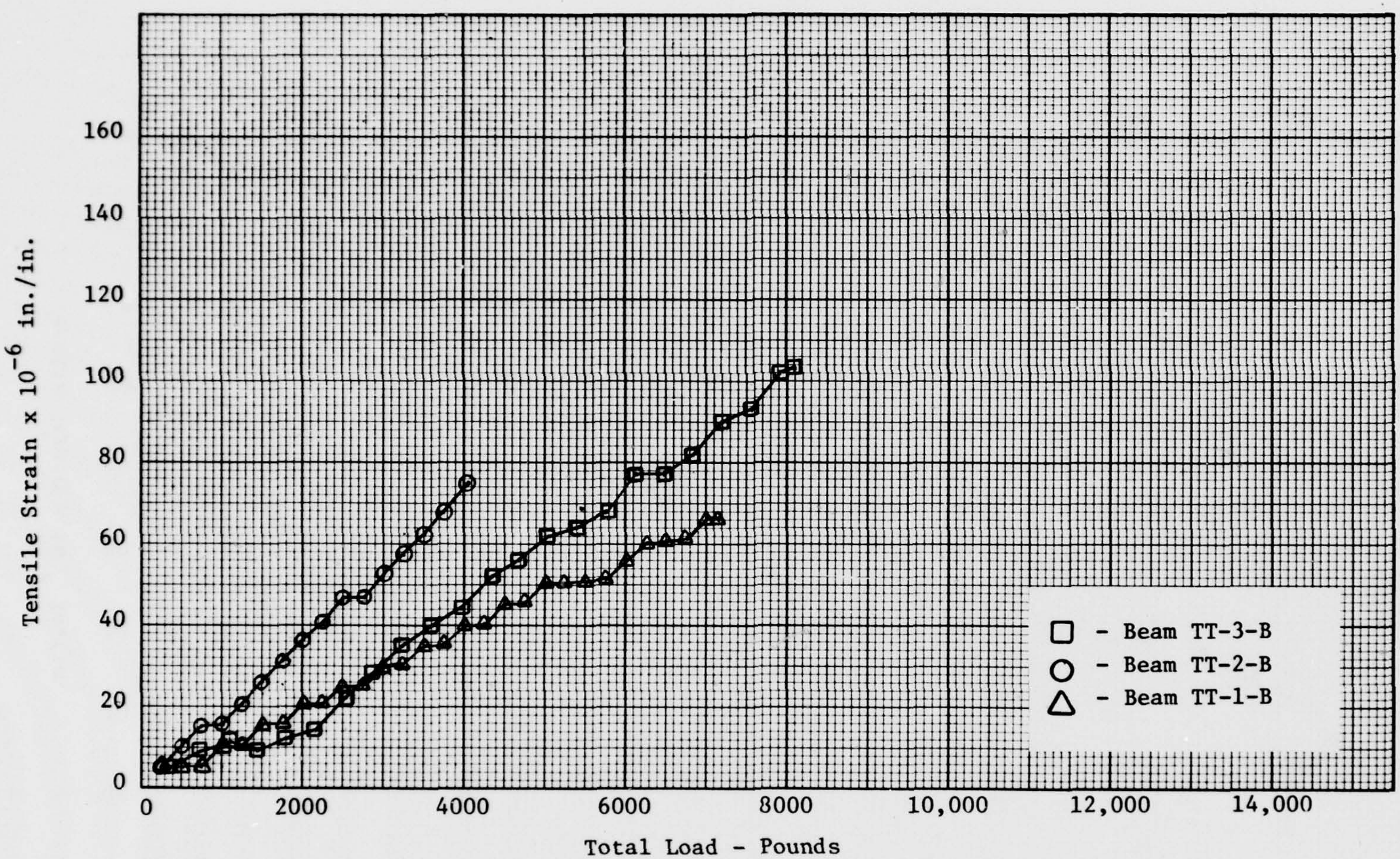

Figure 10. Tensile strain capacity - Mixture B

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JULY 1968

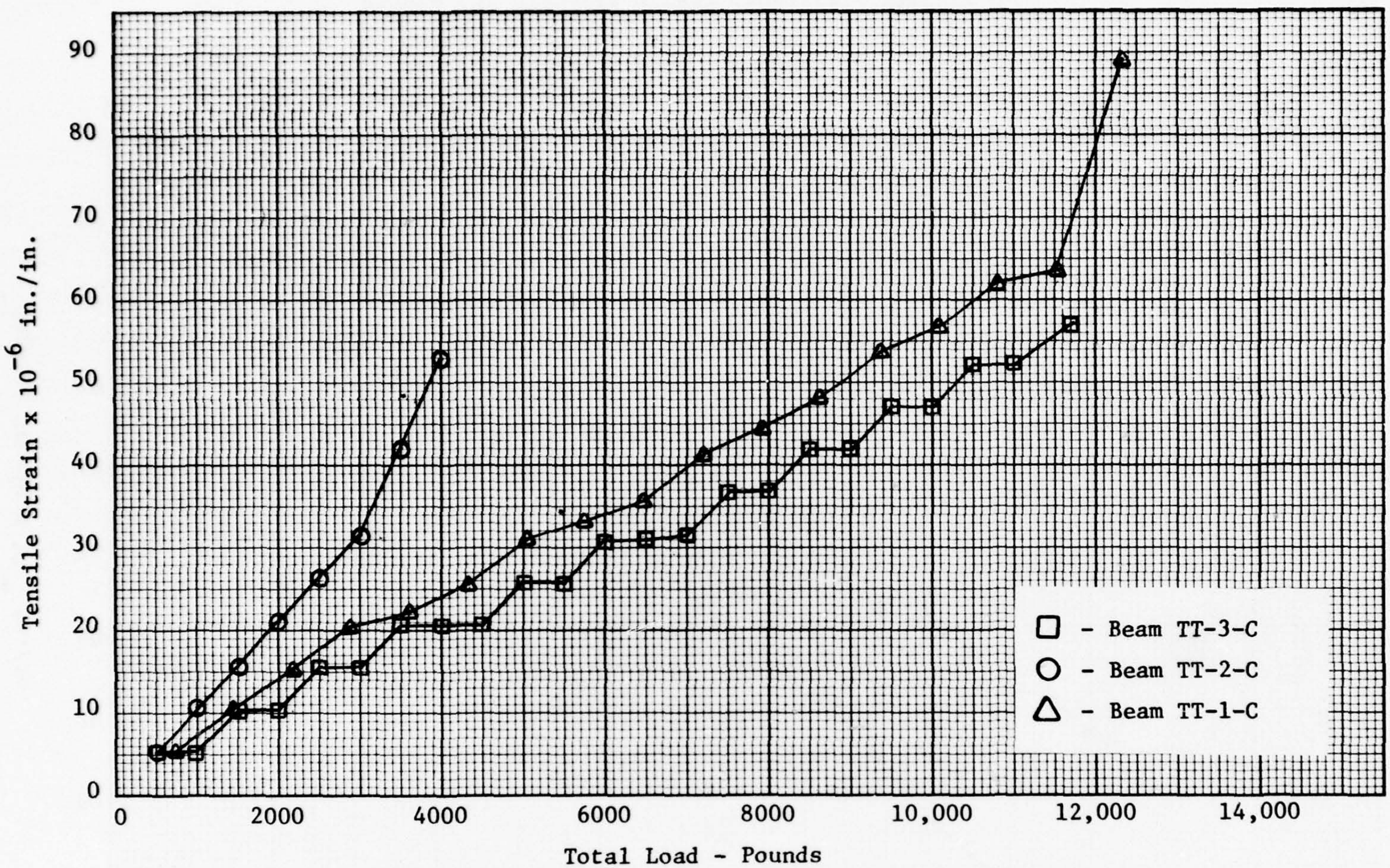

Figure 11. Tensile strain capacity - Mixture C

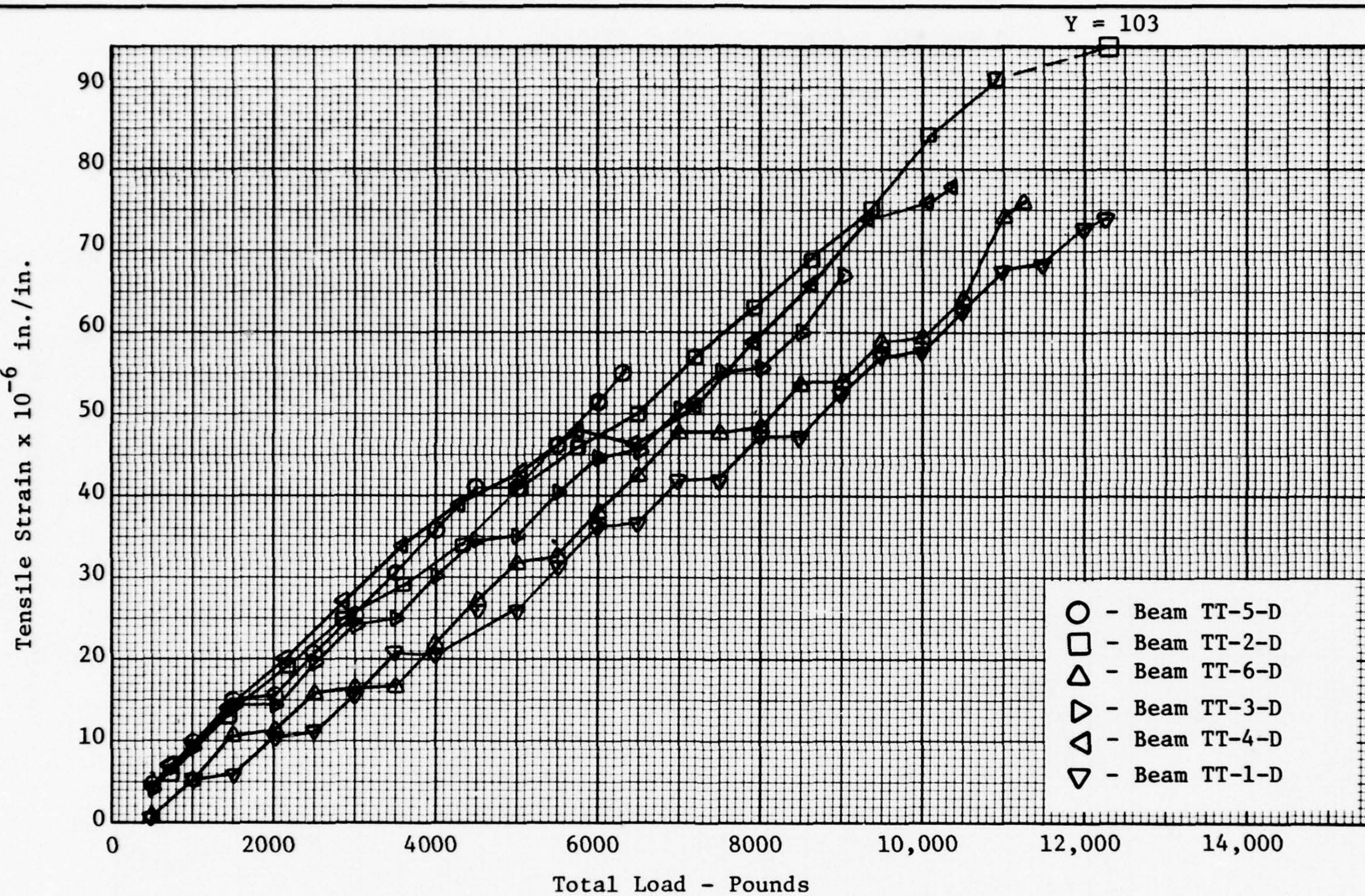

Figure 12. Tensile strain capacity - Mixture D

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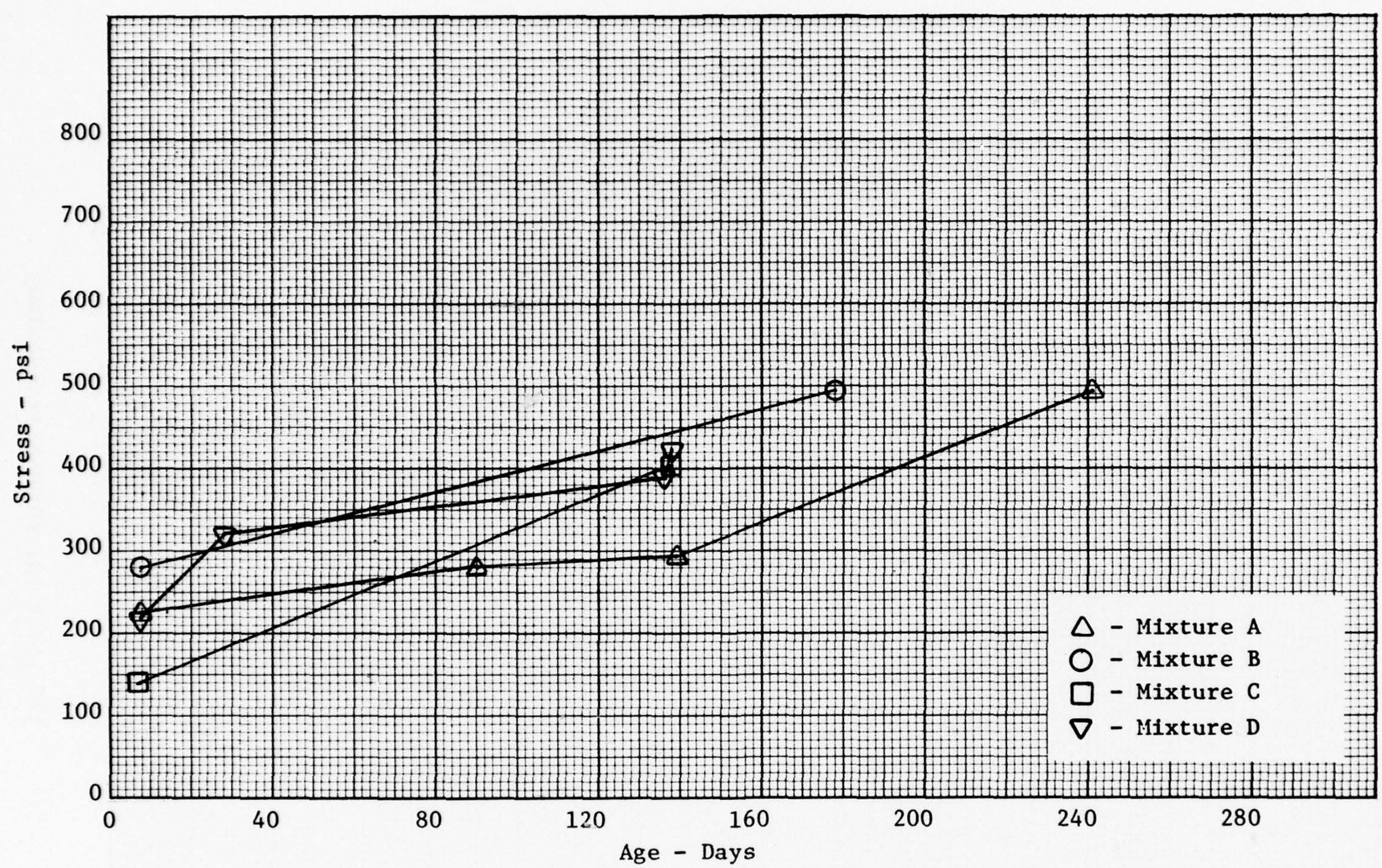

Figure 13. Flexural stress capacity of concrete

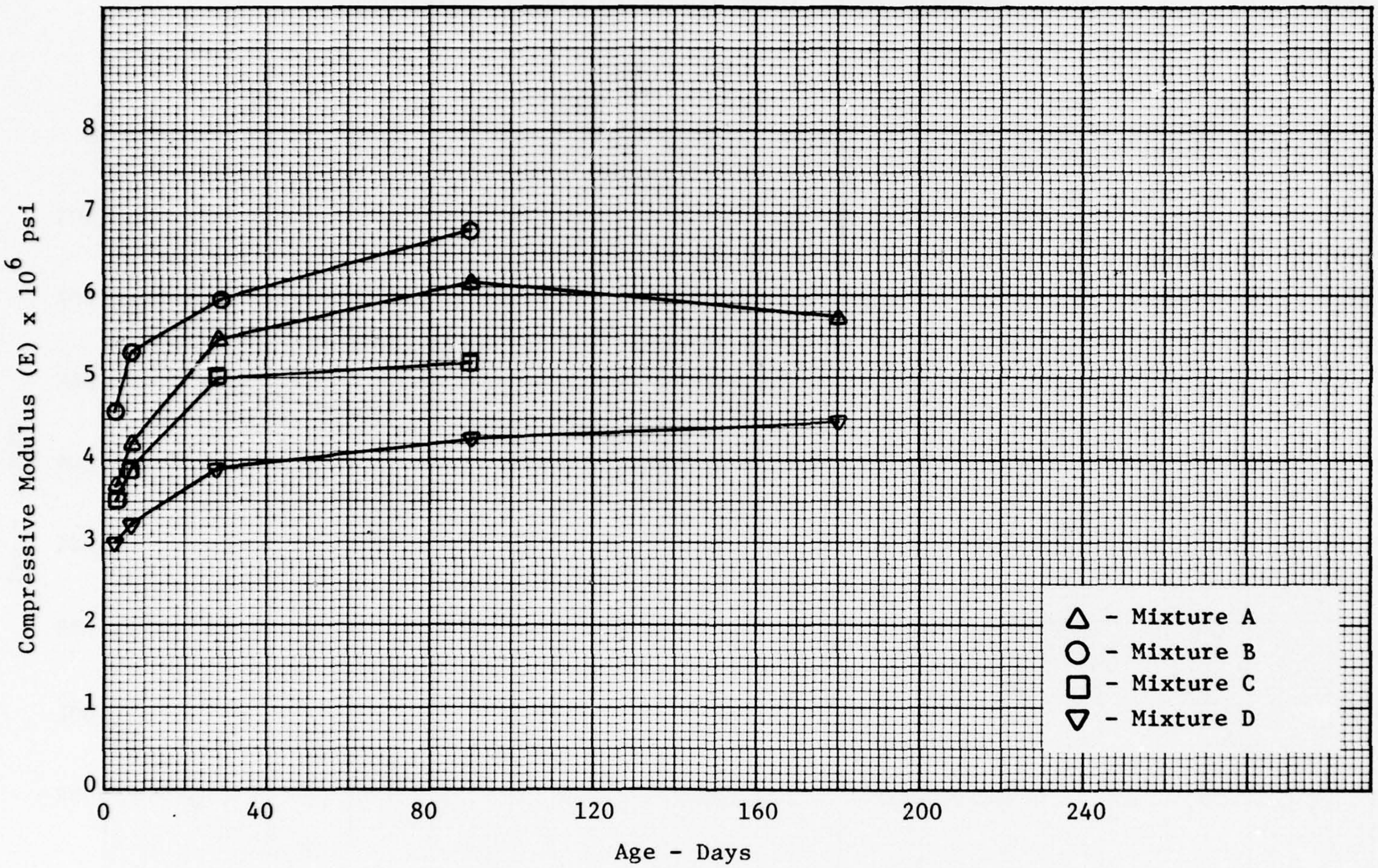

Figure 14. Modulus of elasticity of concrete

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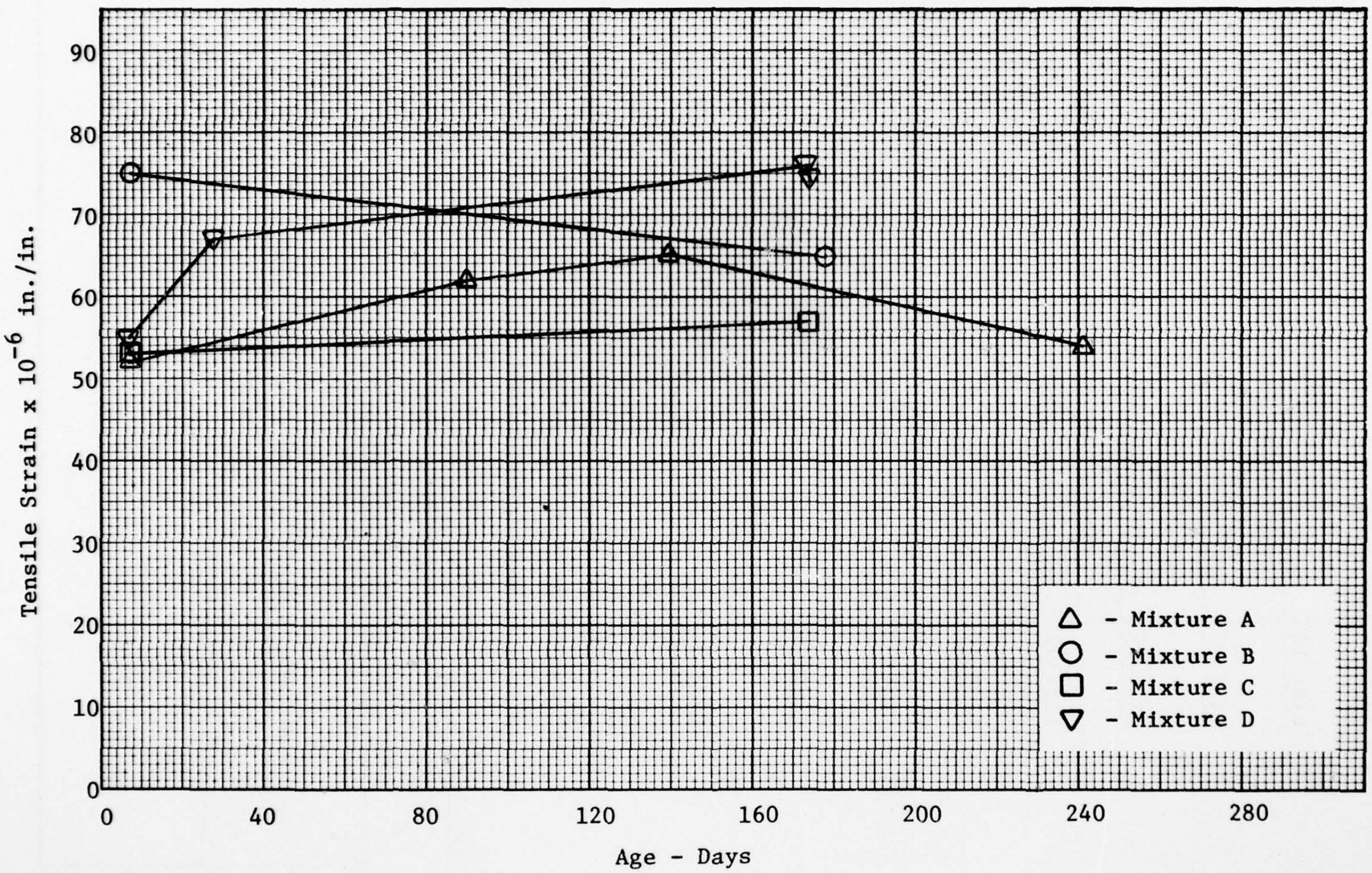


Figure 15. Rapid-load strain capacity of concrete

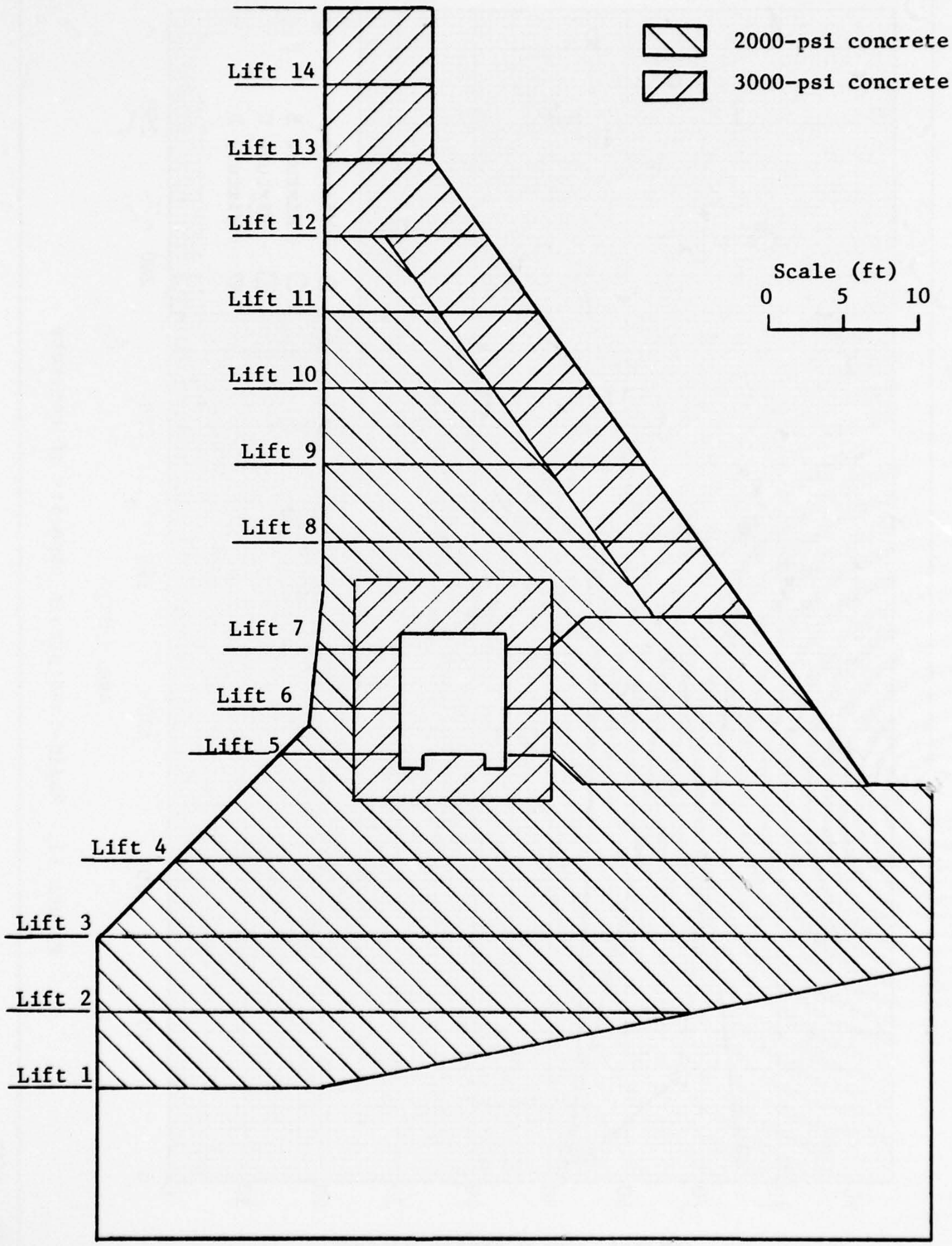
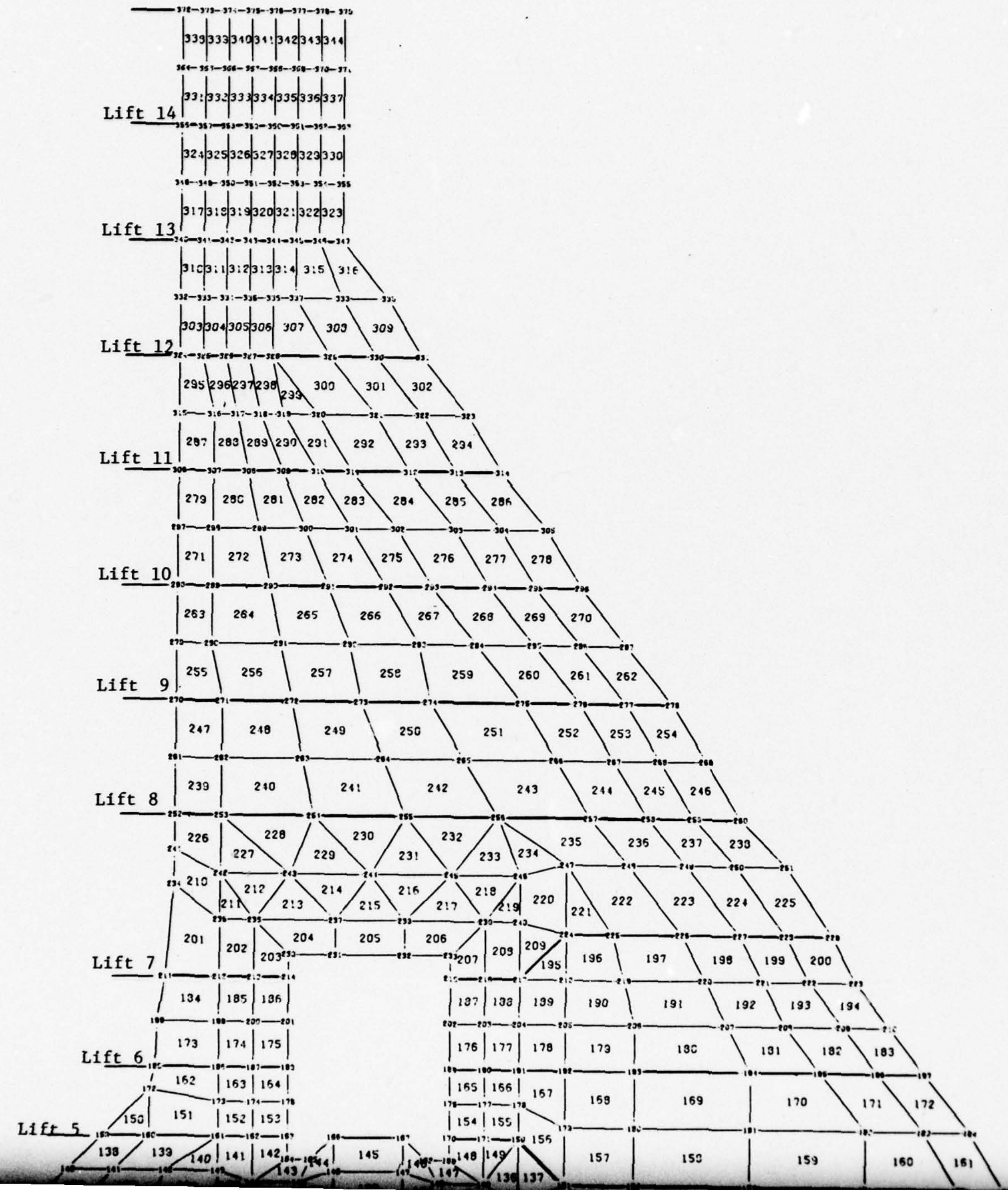


Figure 16. Dam monolith configuration

TENN-TOM WATERWAY DAM MONOLITH



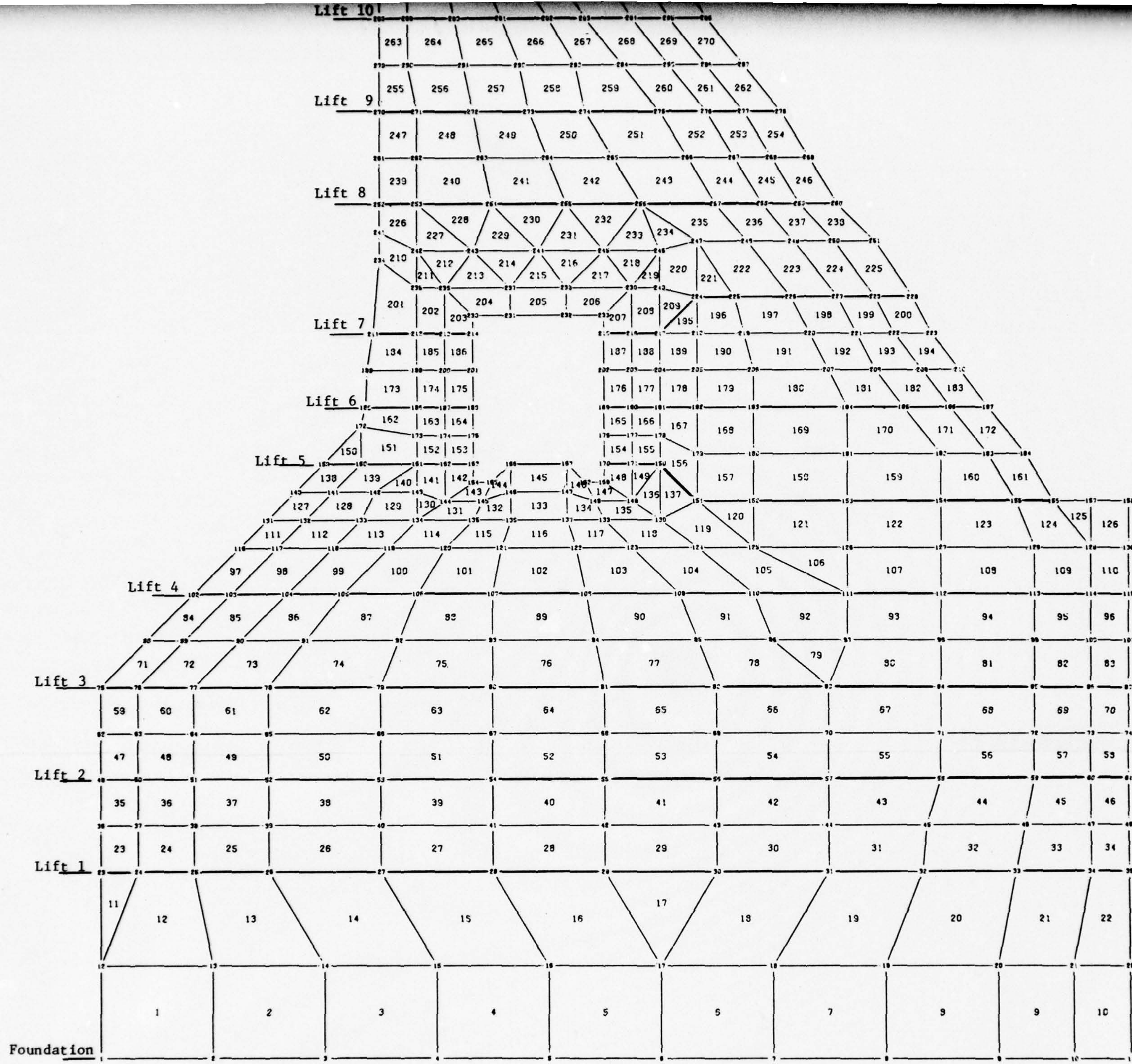


Figure 17. Finite Element Model - Dam Monolith

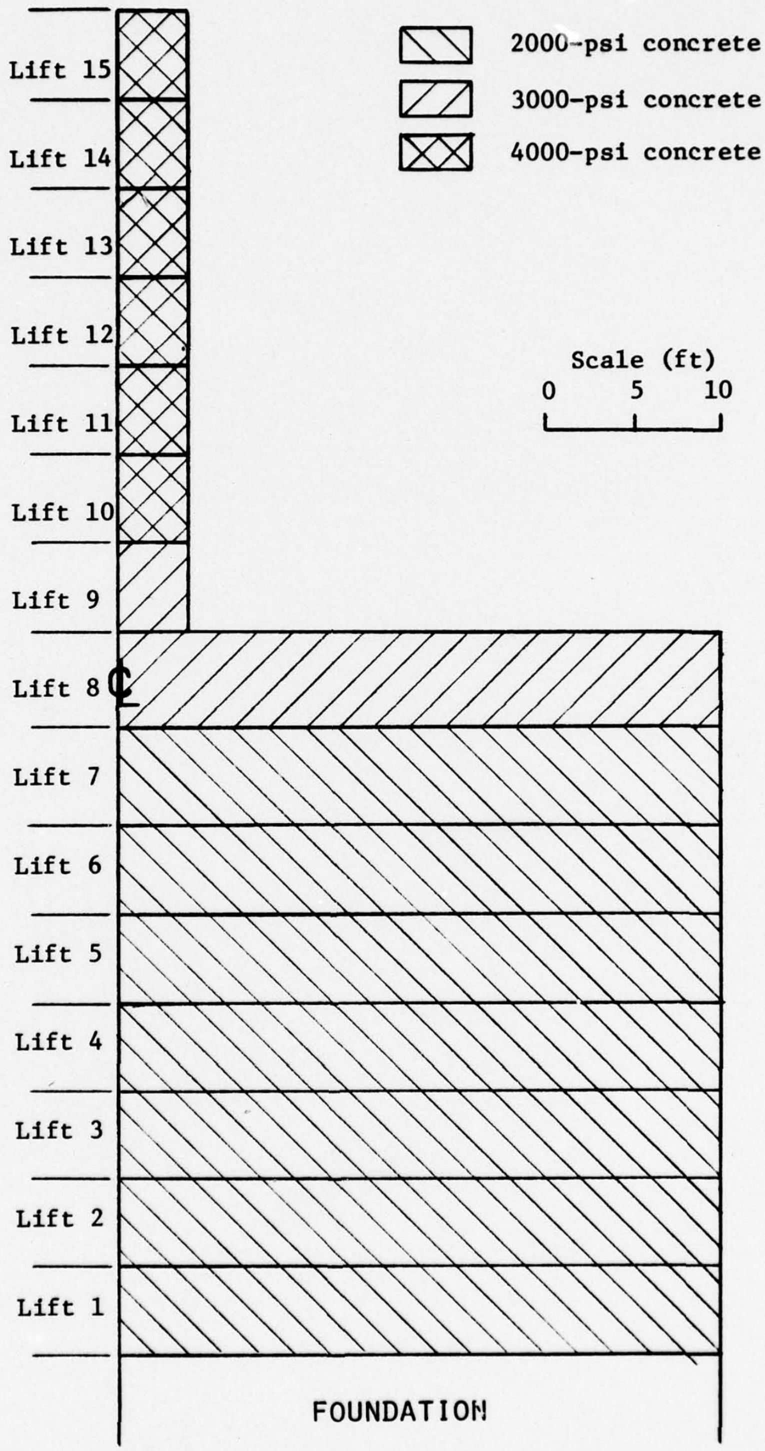
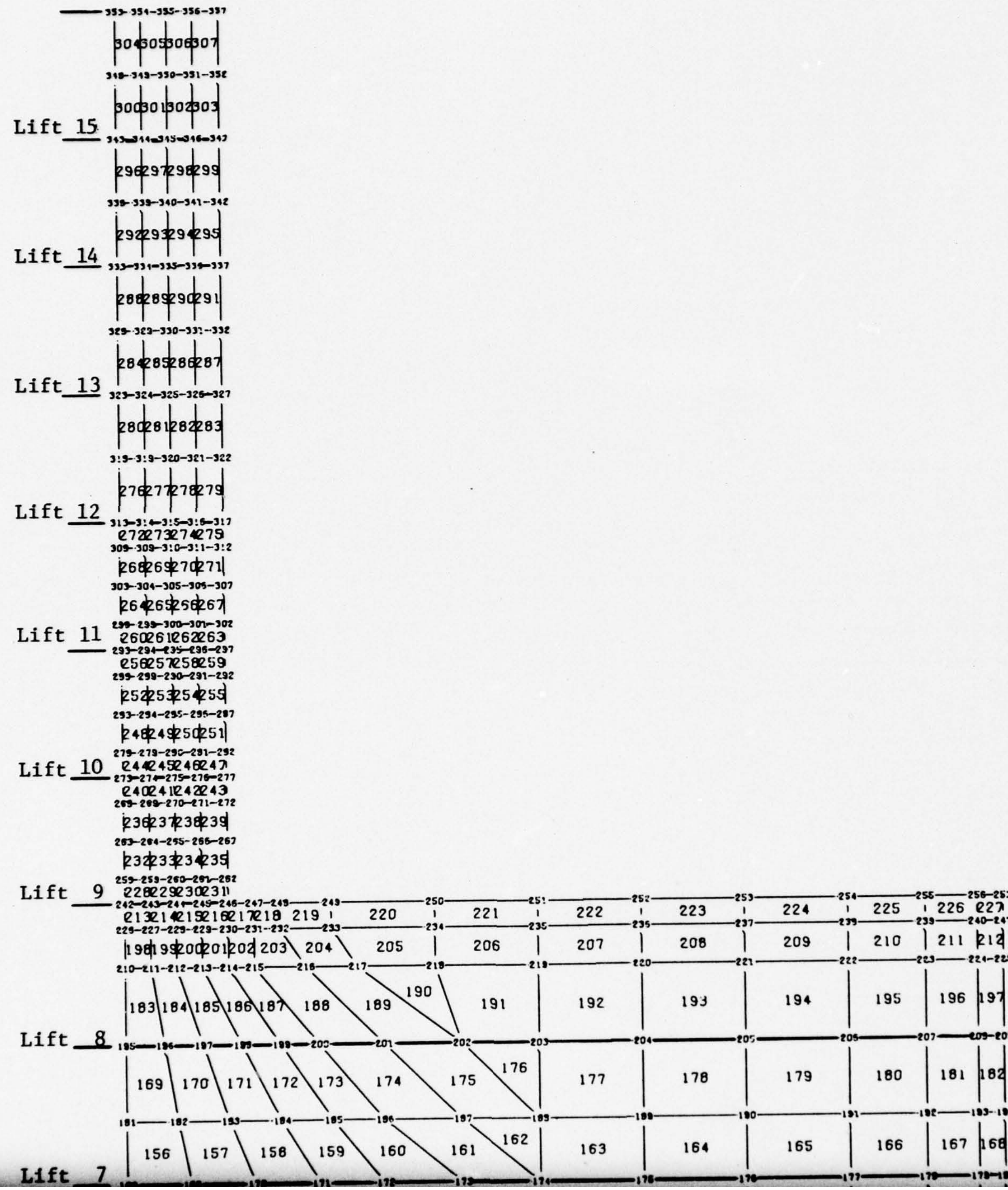
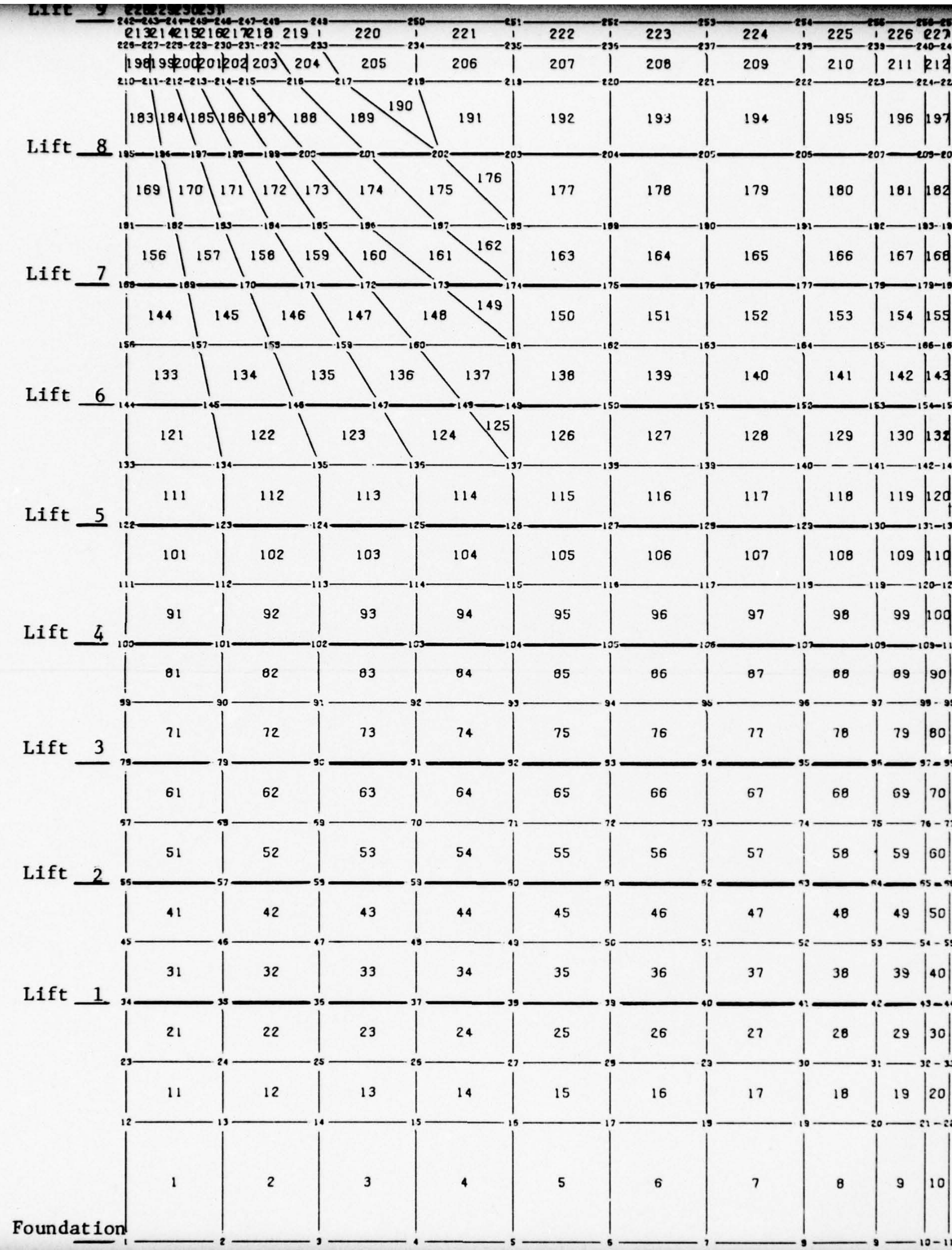


Figure 18. Spillway monolith configuration

TENN-TOM WTRWY SPILLWAY PIER MONOLITH




Lift 9
Lift 8
Lift 7
Lift 6
Lift 5
Lift 4
Lift 3
Lift 2
Lift 1
Foundation

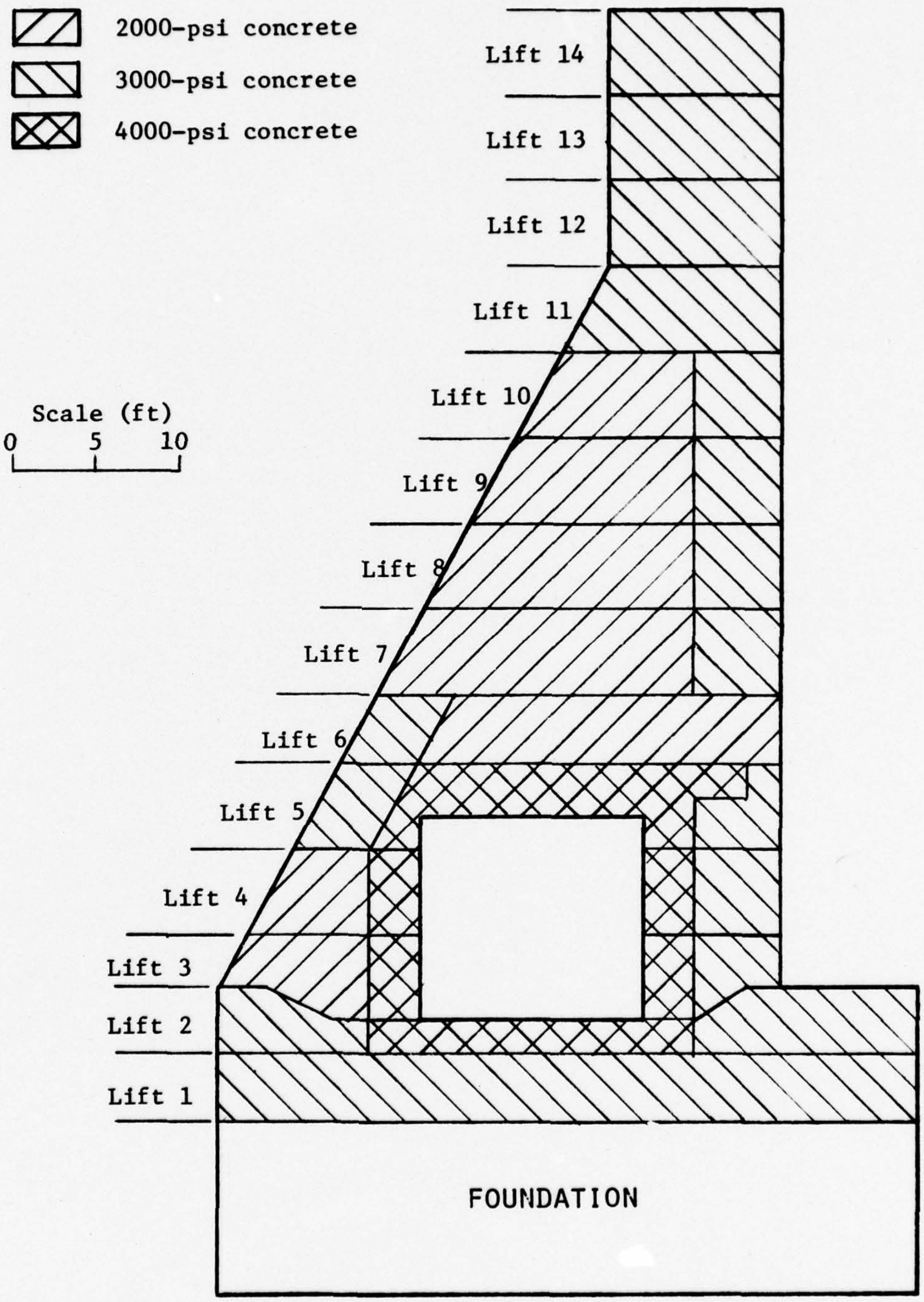
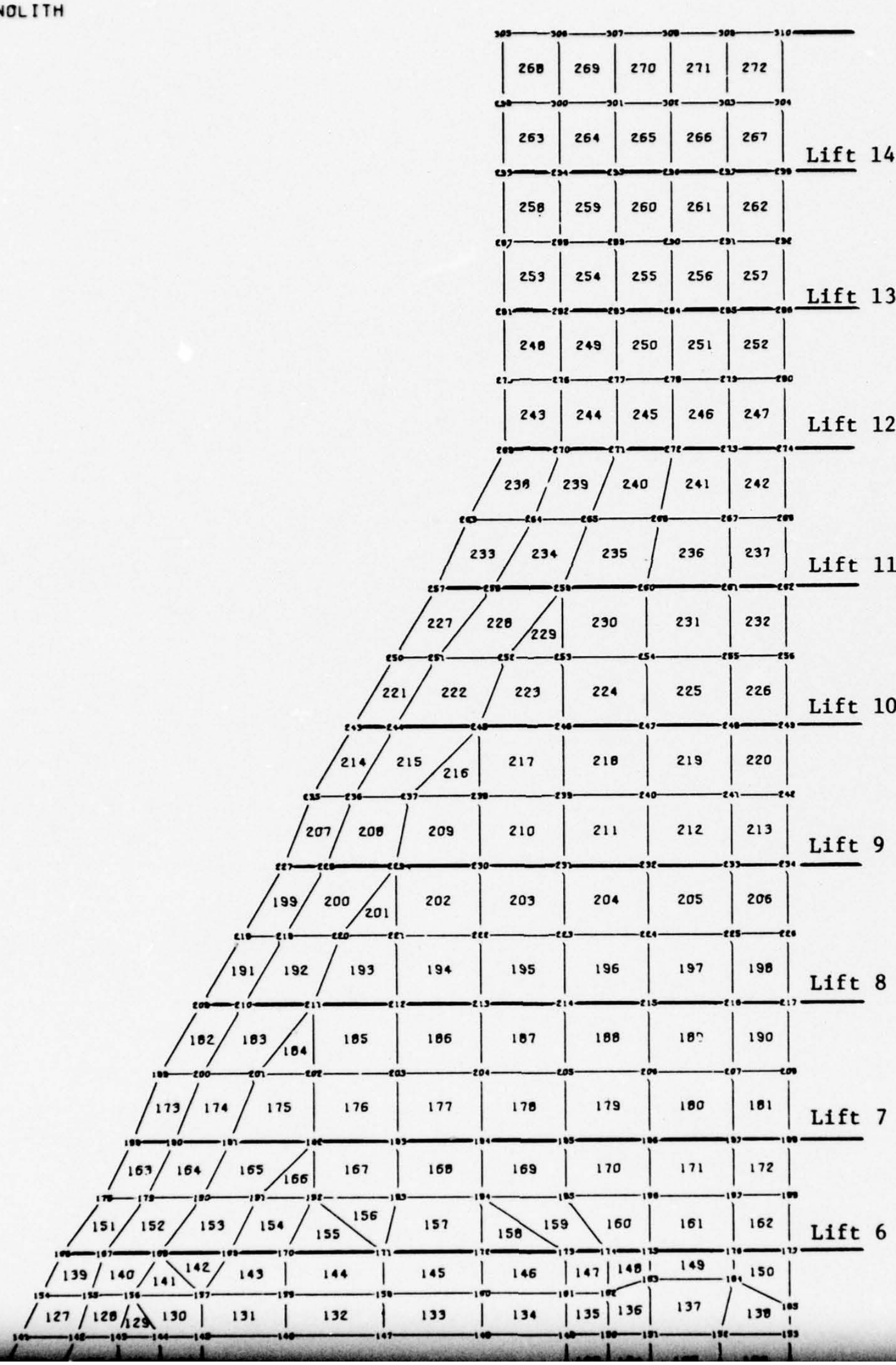


Figure 20. Lock wall monolith configuration

TENN-TOM WTRWY LOCKWALL MONOLITH



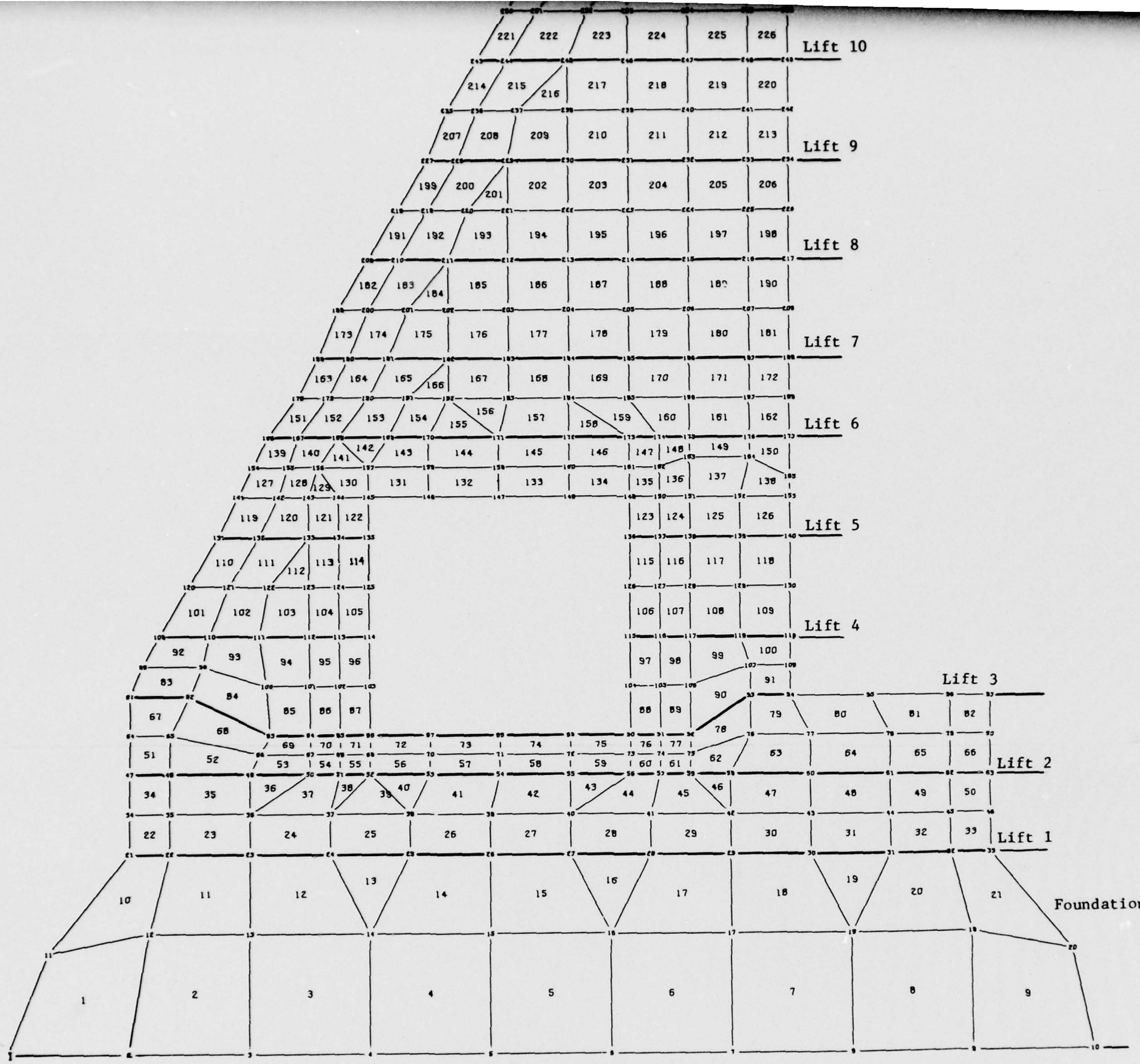


Figure 21. Finite Element Model - Lock Wall Monolith

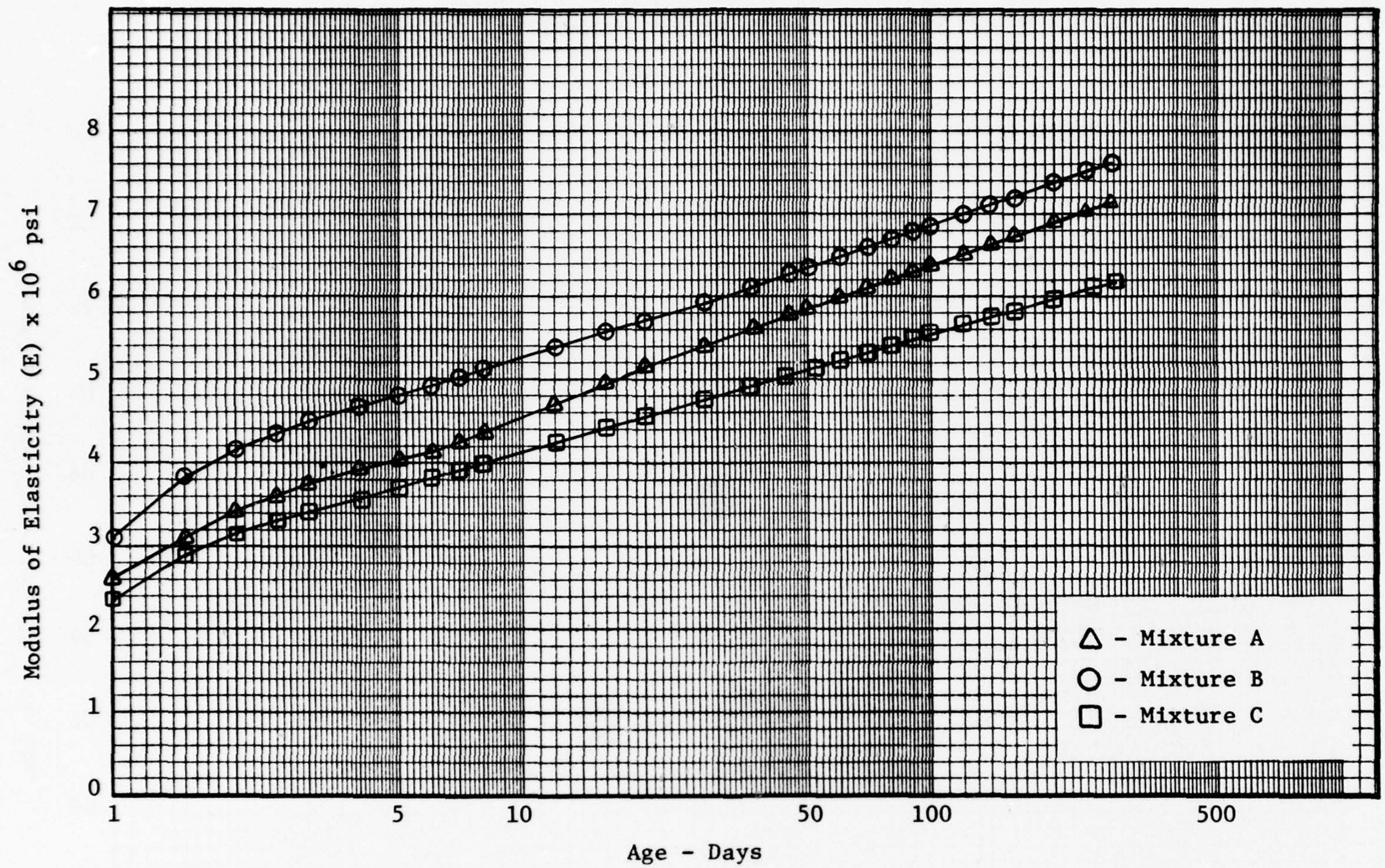


Figure 22. Modulus of elasticity versus age of concrete used for computer input data

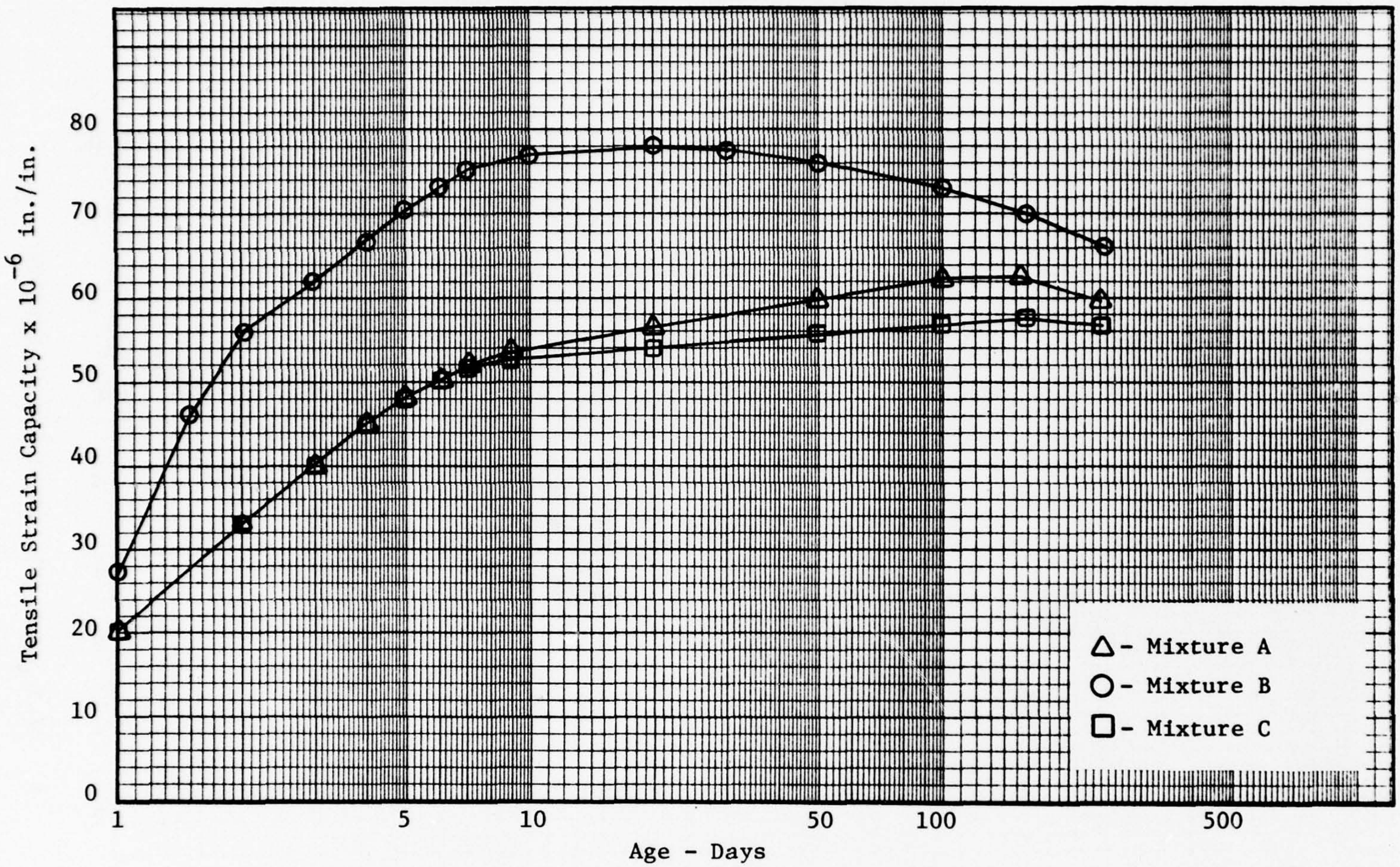


Figure 23. Rapid-load strain capacity versus age of concrete used for computer input data

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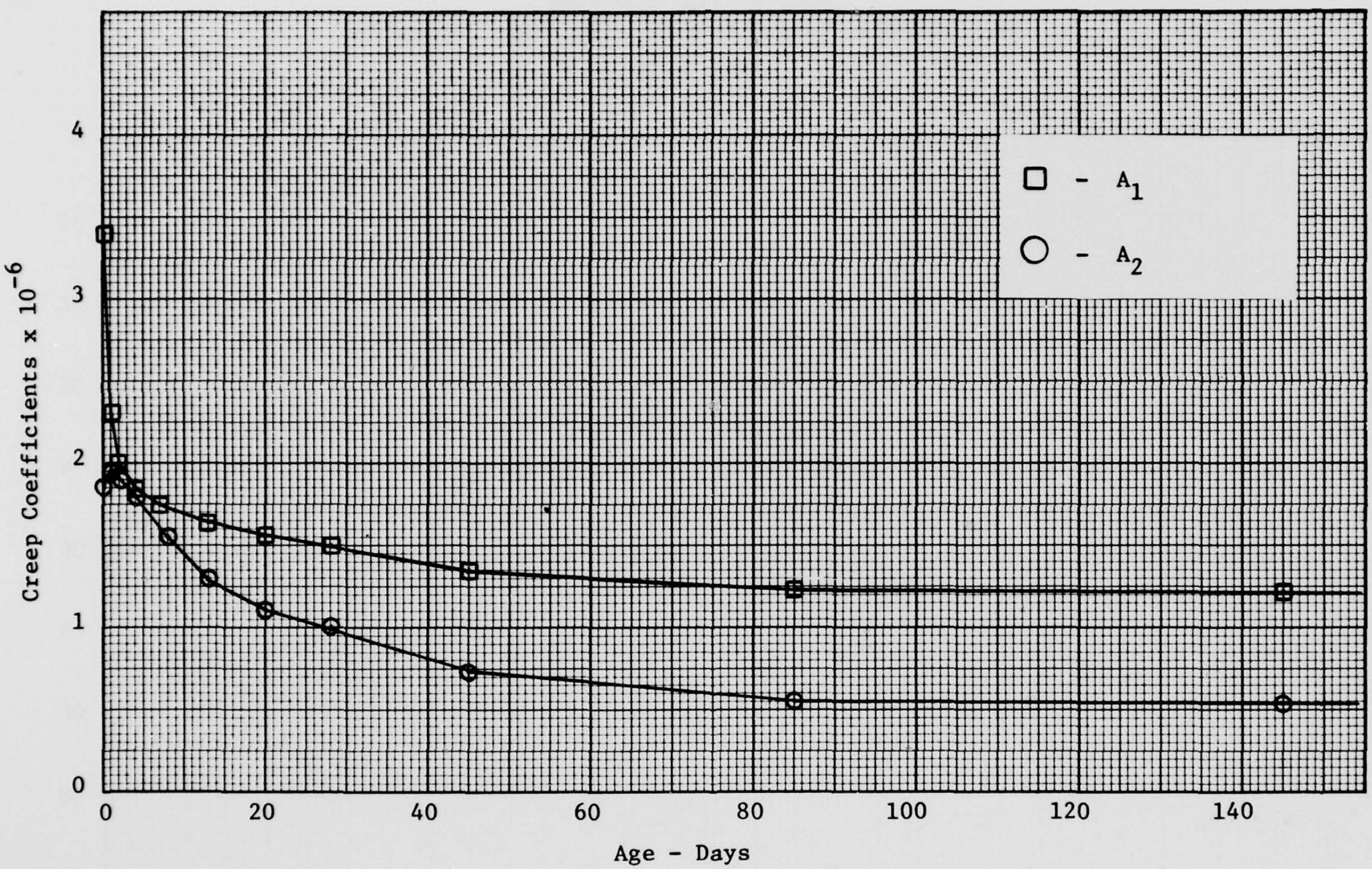

Figure 24. Concrete creep coefficients used for computer input data

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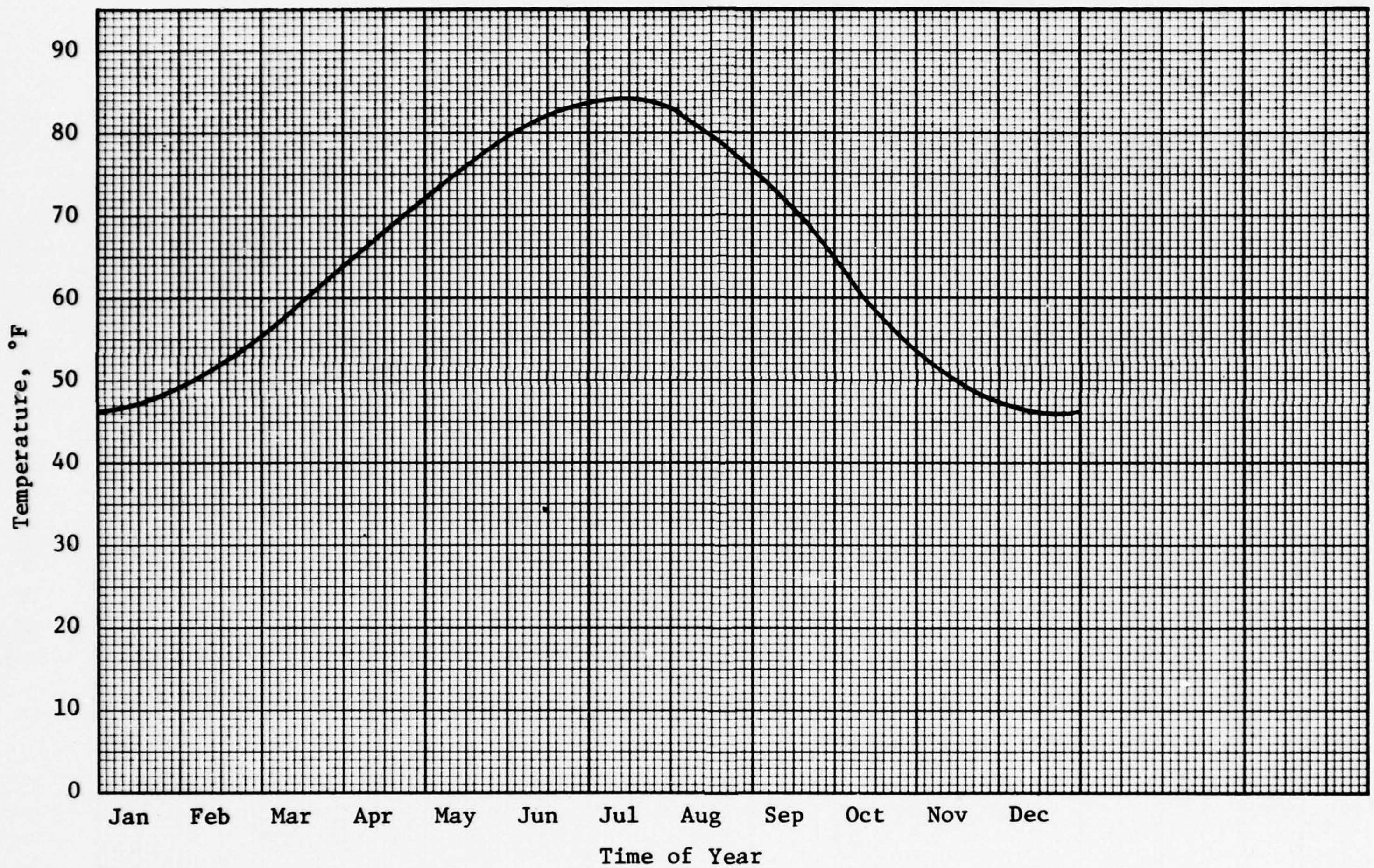

Figure 25. Mean daily normal ambient temperatures, Aberdeen, Mississippi

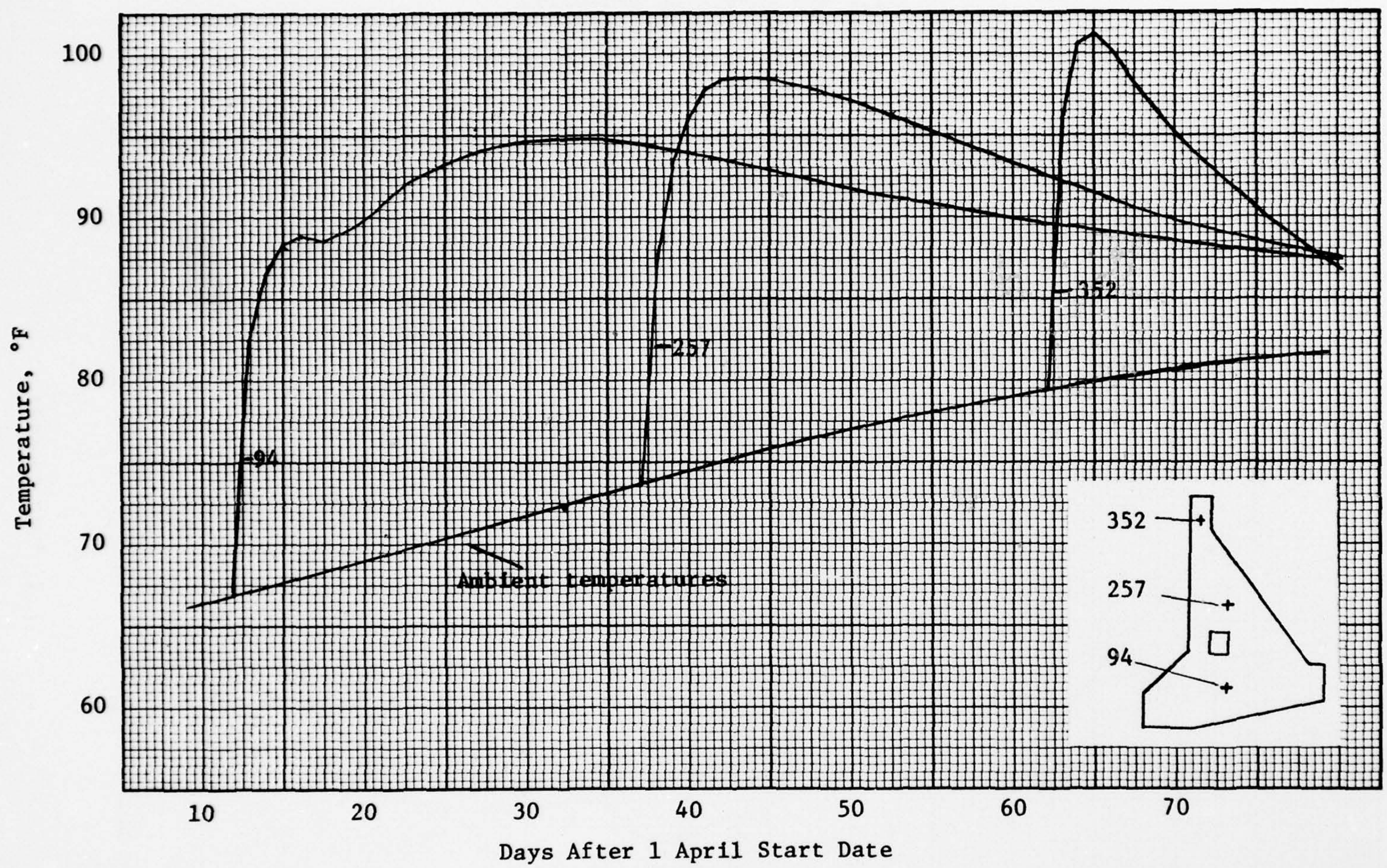

Figure 26. Nodal temperatures in dam monolith for construction at 5 days per lift

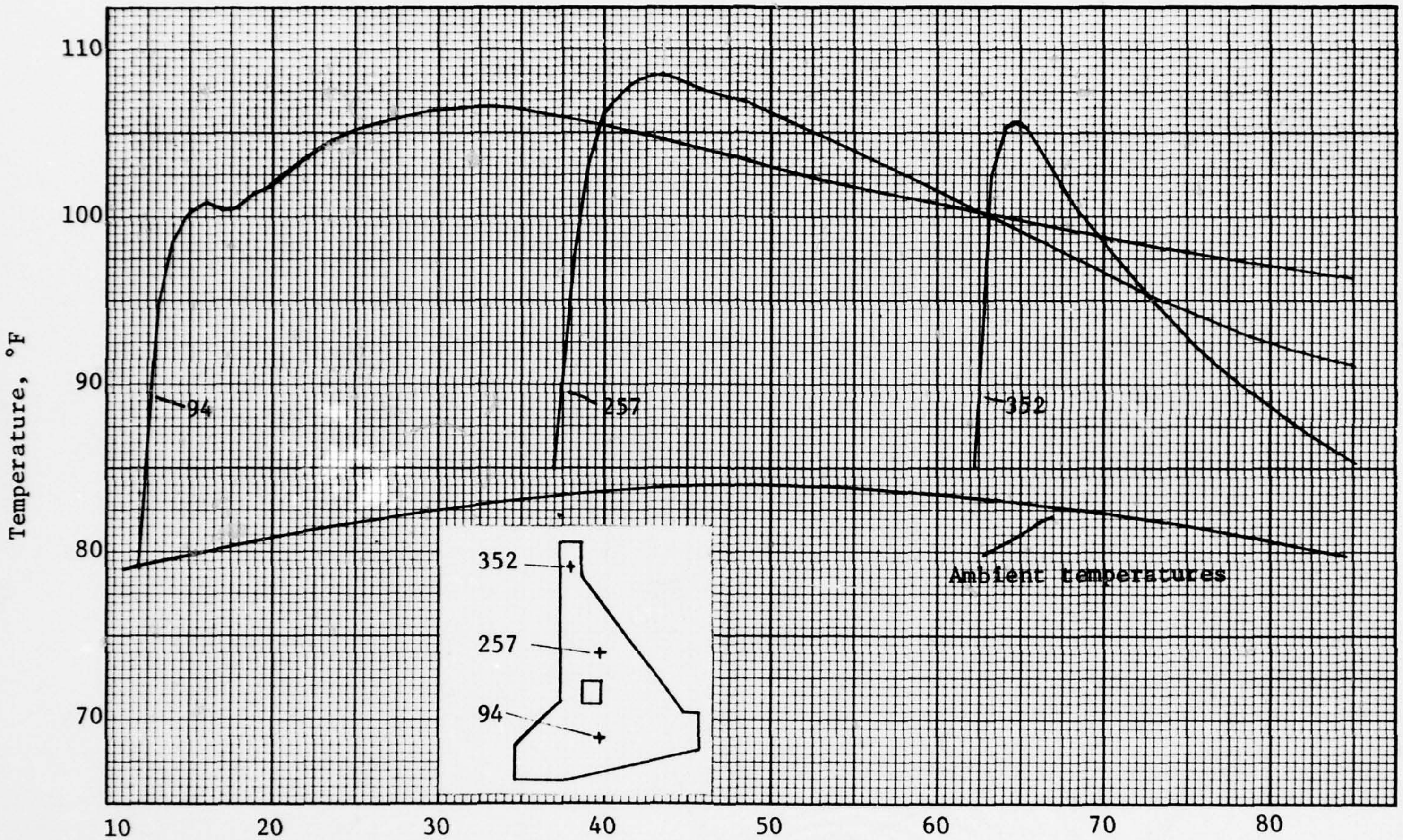

Figure 27. Nodal temperatures in dam monolith constructed at 5 days per lift

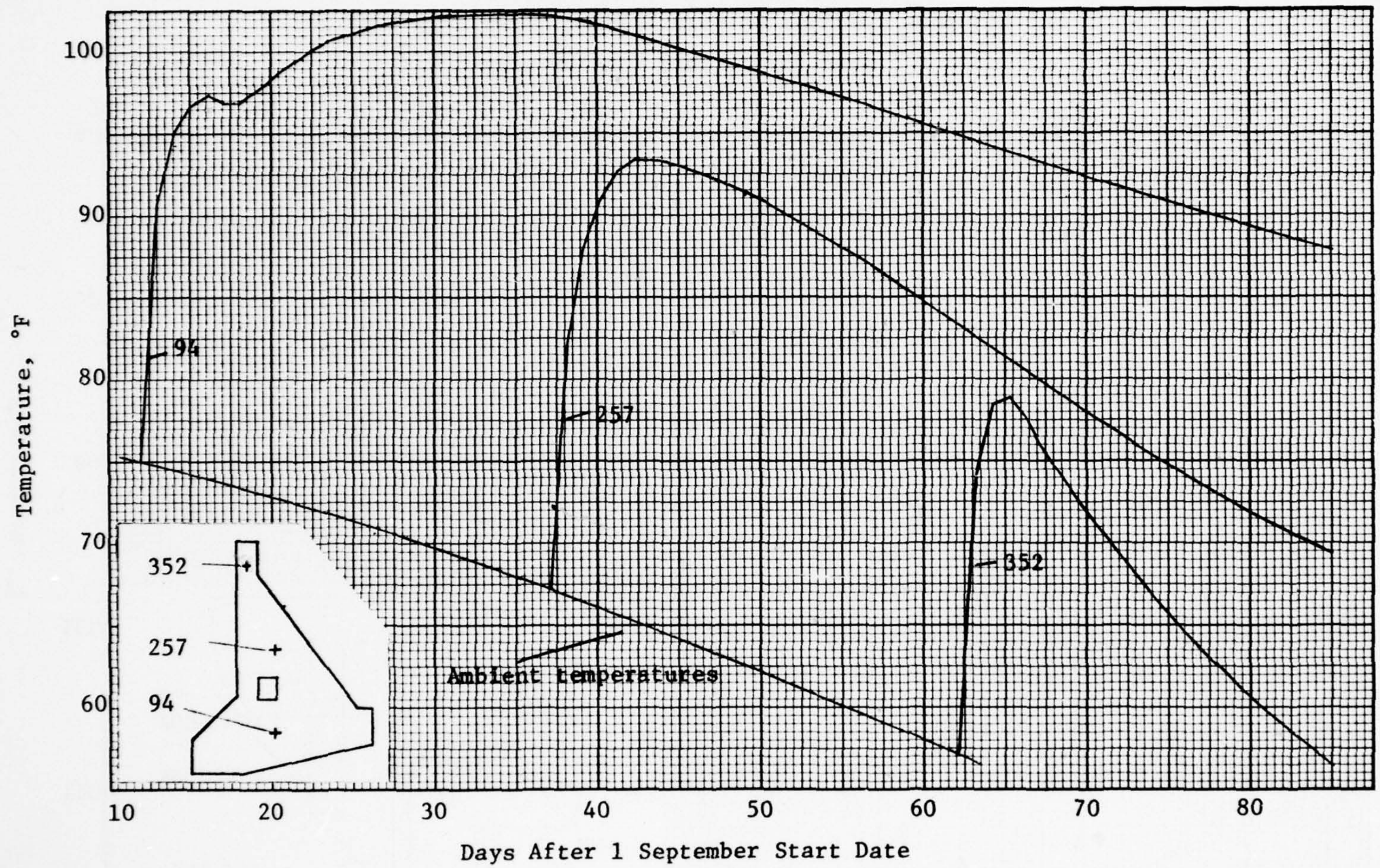

Figure 28. Nodal temperatures in dam monolith constructed at 5 days per lift

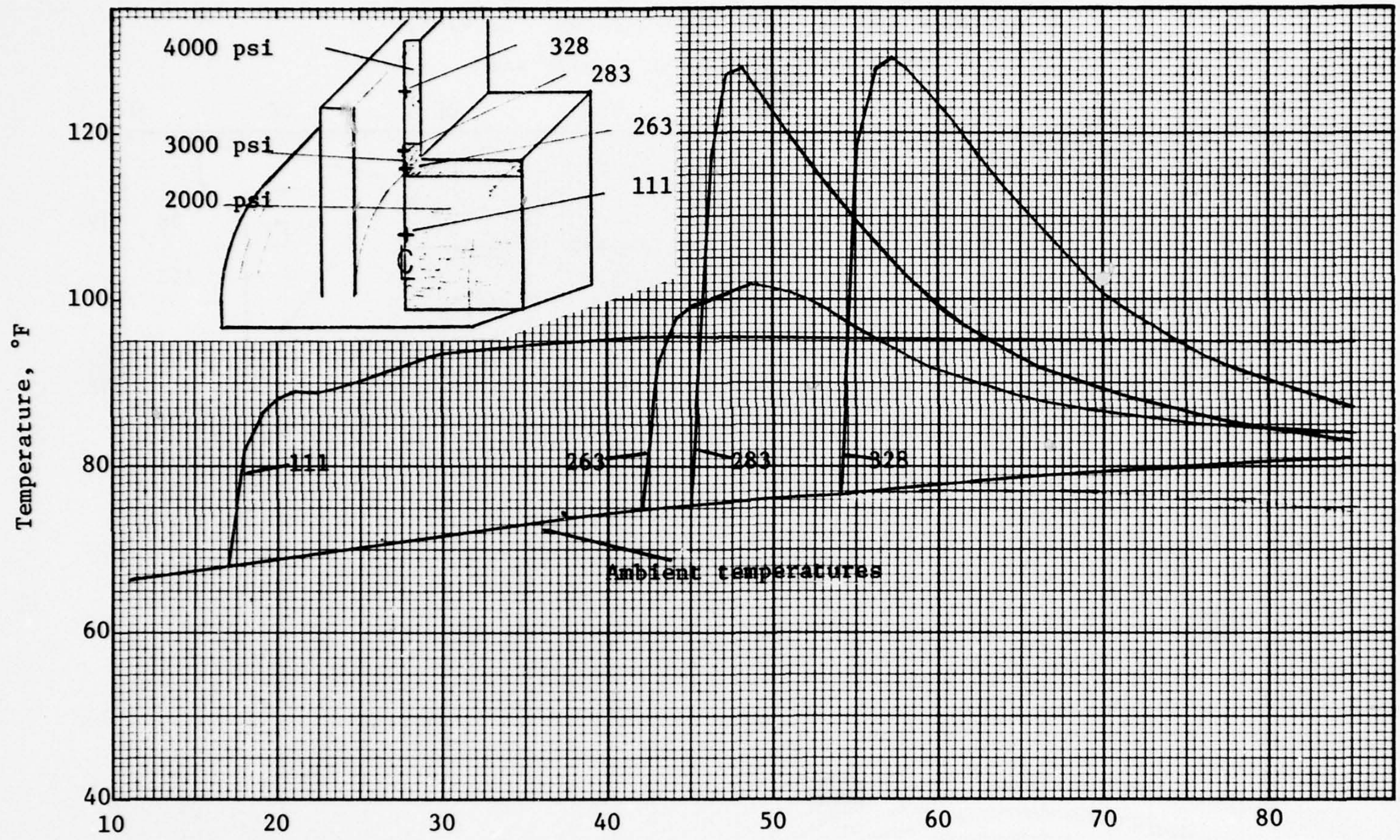

Figure 29. Nodal temperatures in spillway monolith for construction at 5/3 days per lift

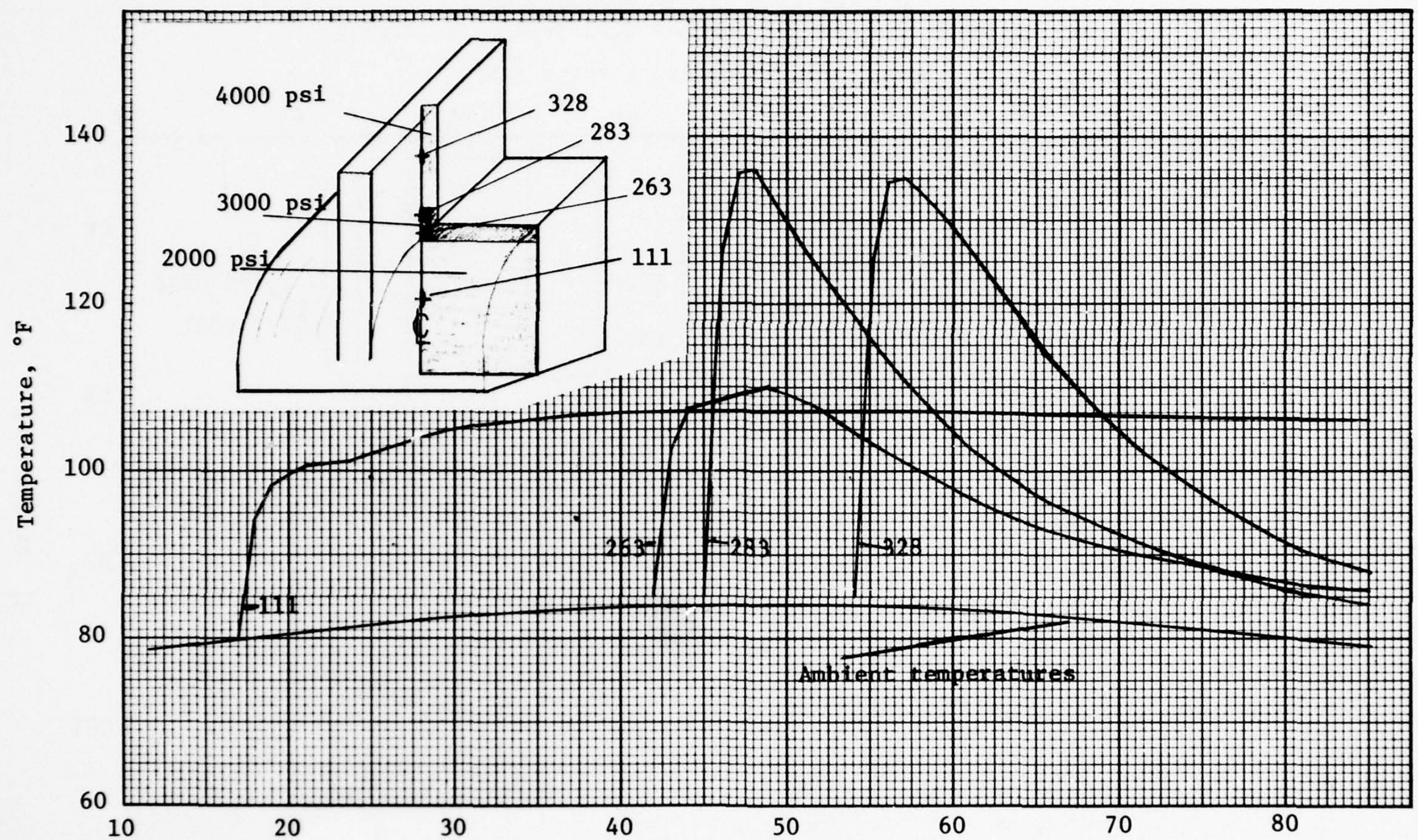

Figure 30. Nodal temperatures in spillway monolith for construction at 5/3 days per lift

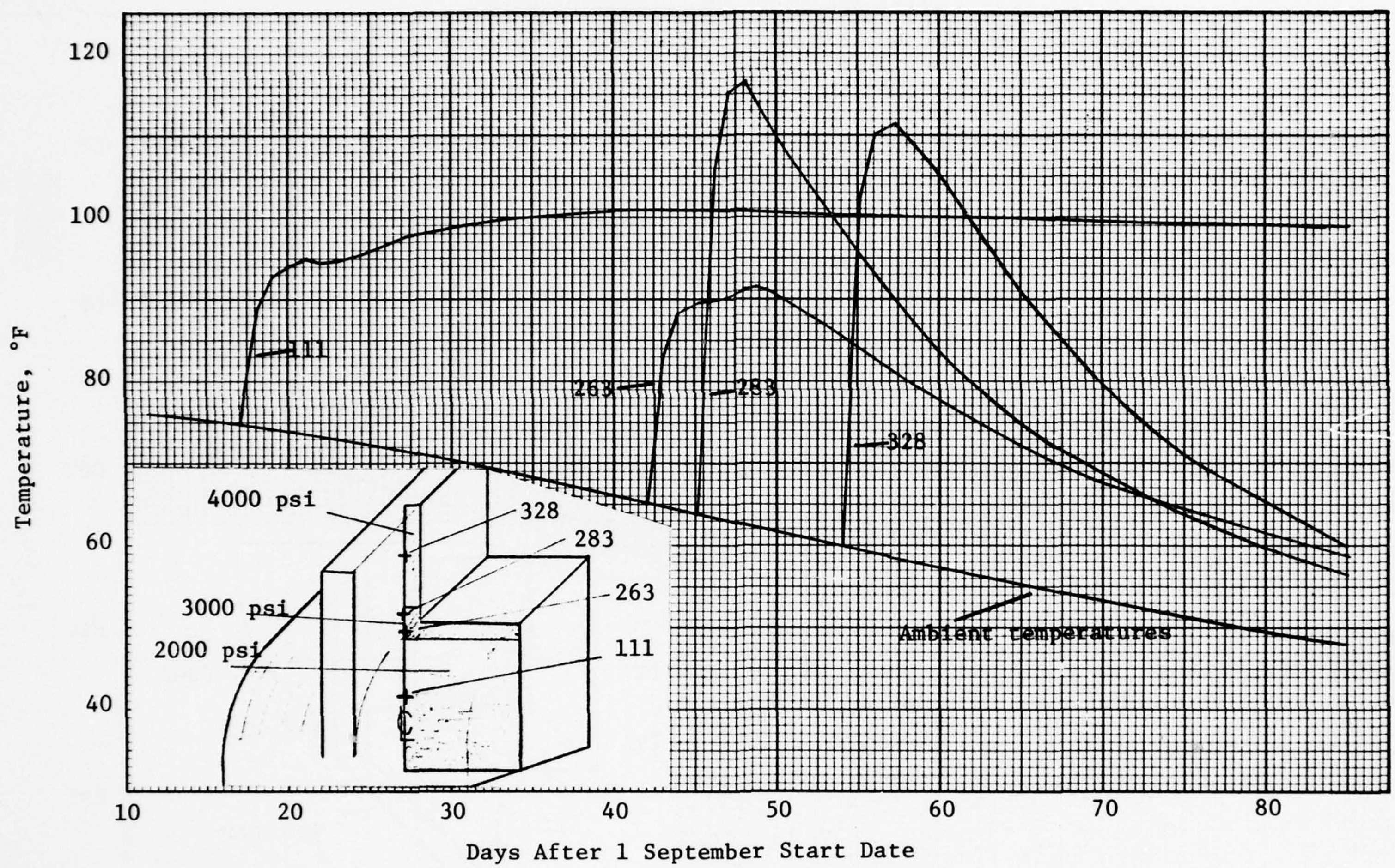

Figure 31. Nodal temperatures in spillway monolith for construction at 5/3 days per lift

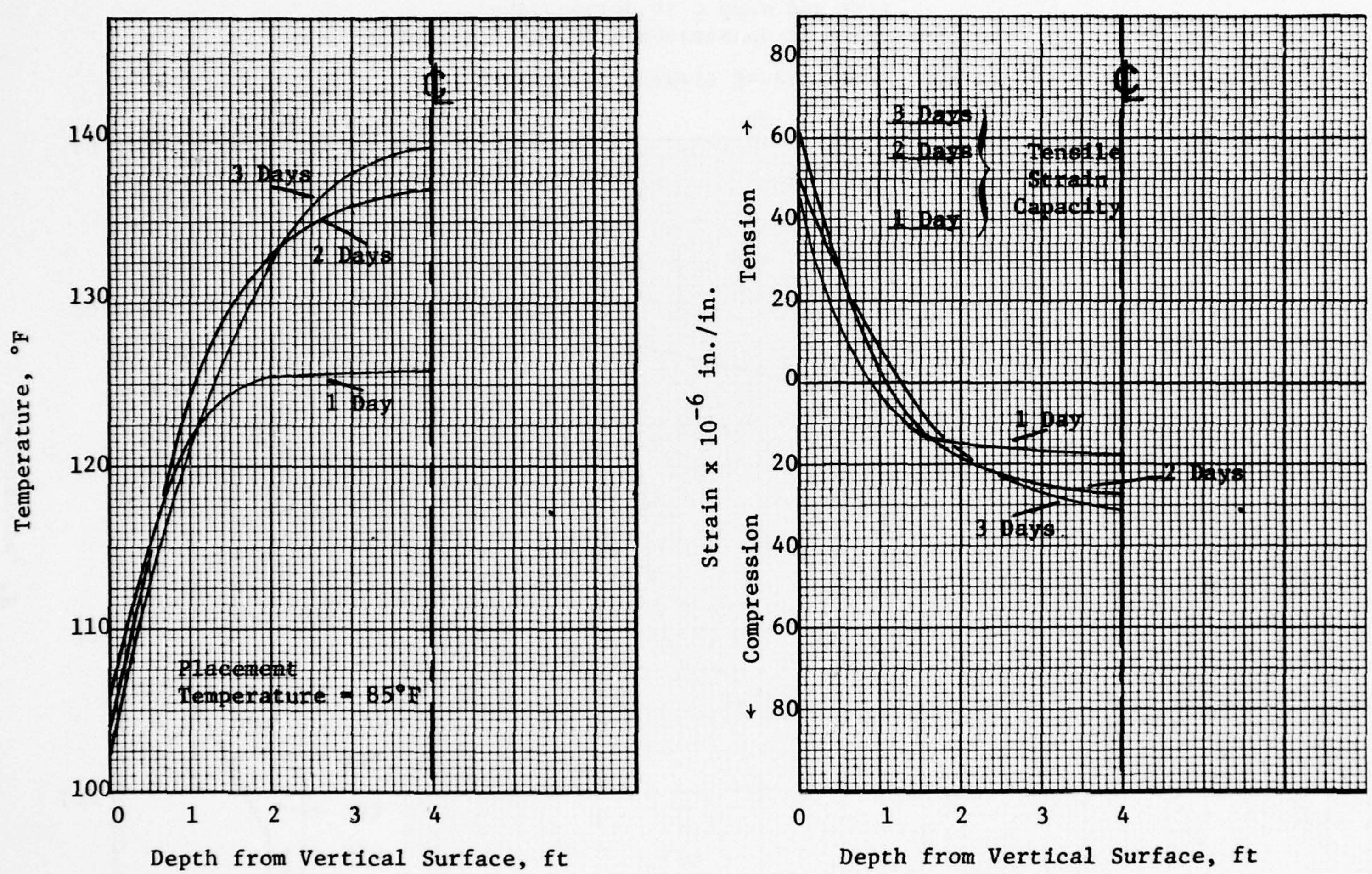

Figure 32. Temperature and vertical normal strain gradients following placement at mid-height of lift 11 in spillway pier

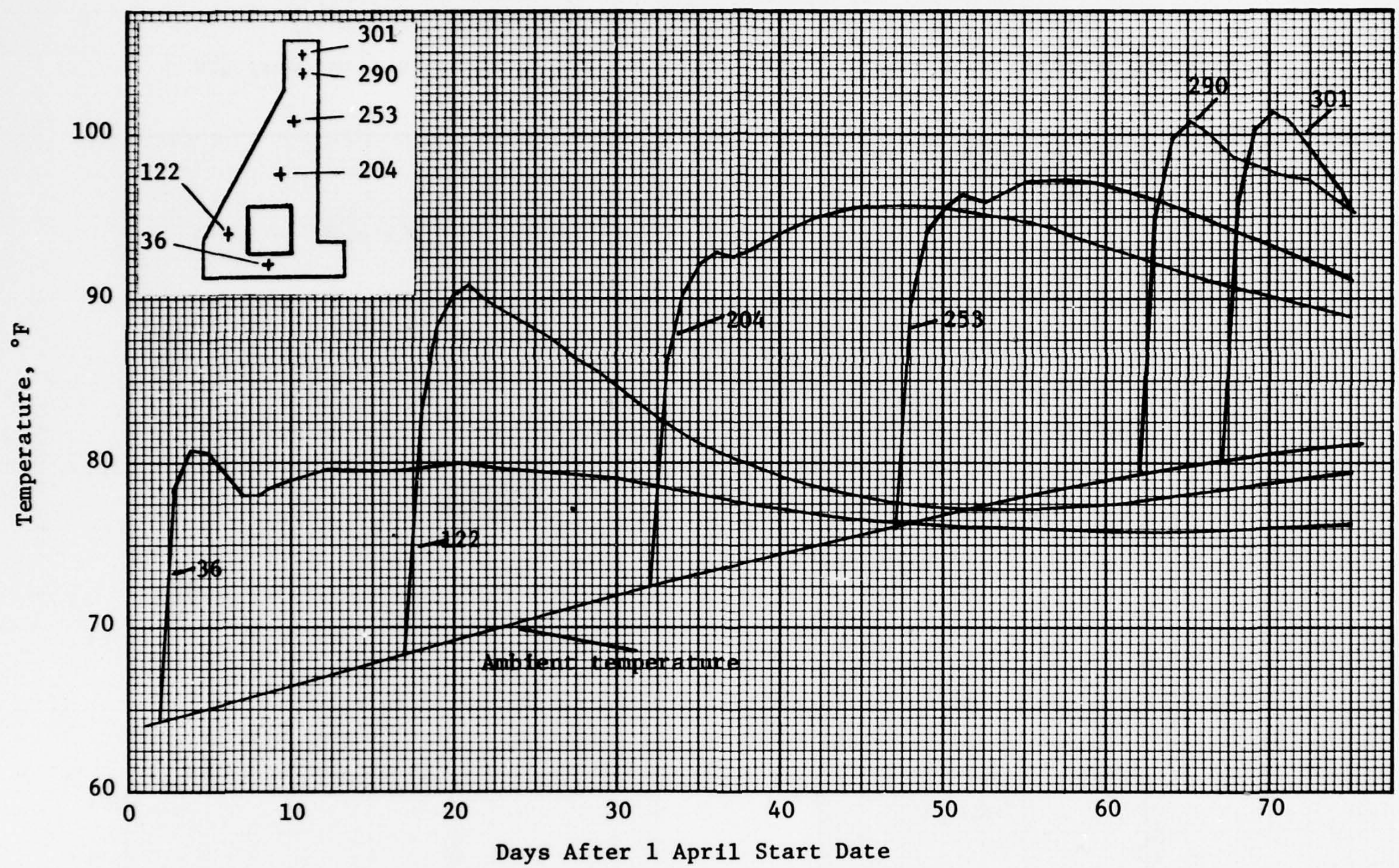

Figure 33. Nodal temperatures in lock wall for construction at 5 days per lift

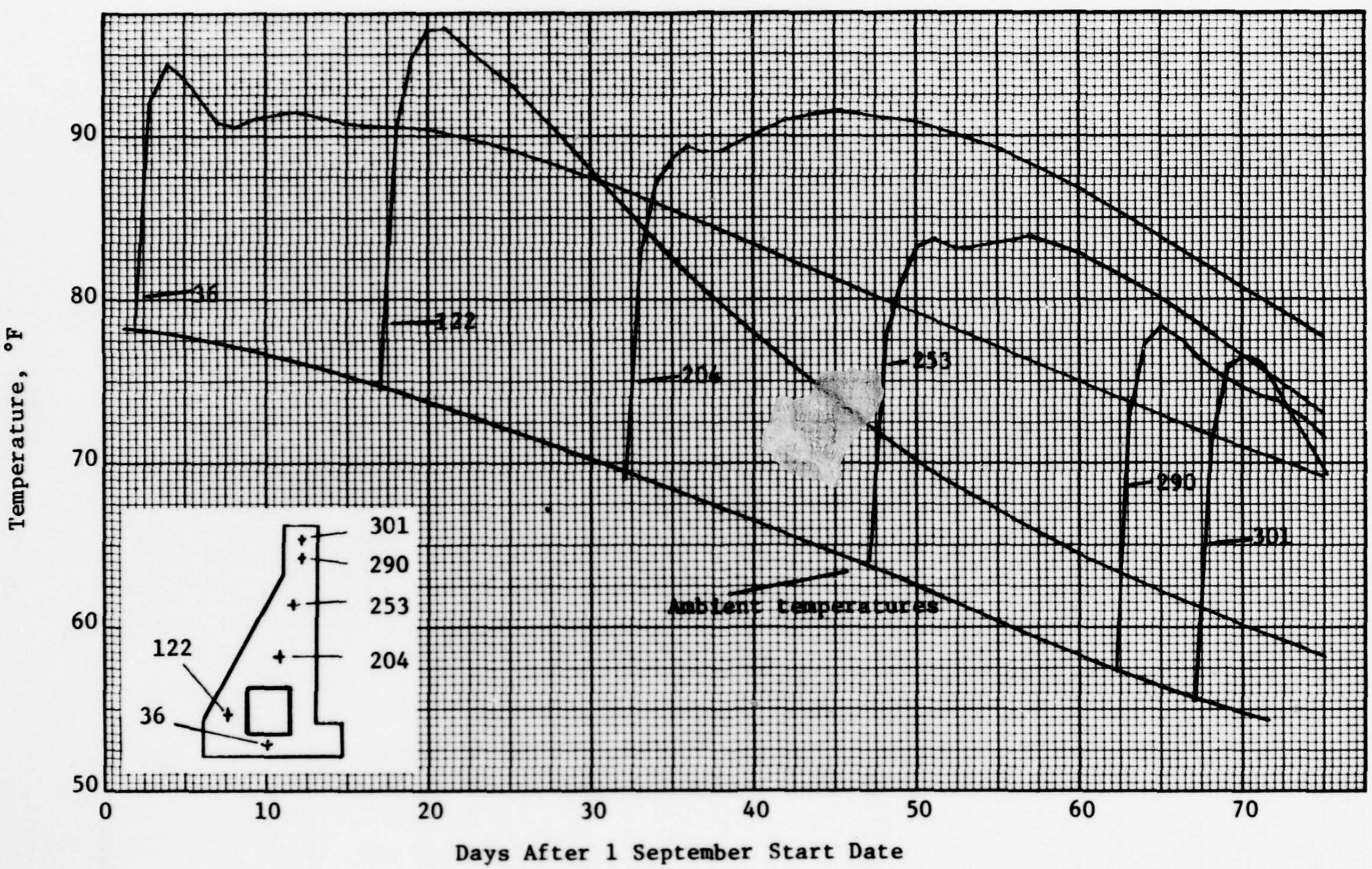

Figure 34. Nodal temperatures in lock wall for construction at 5 days per lift

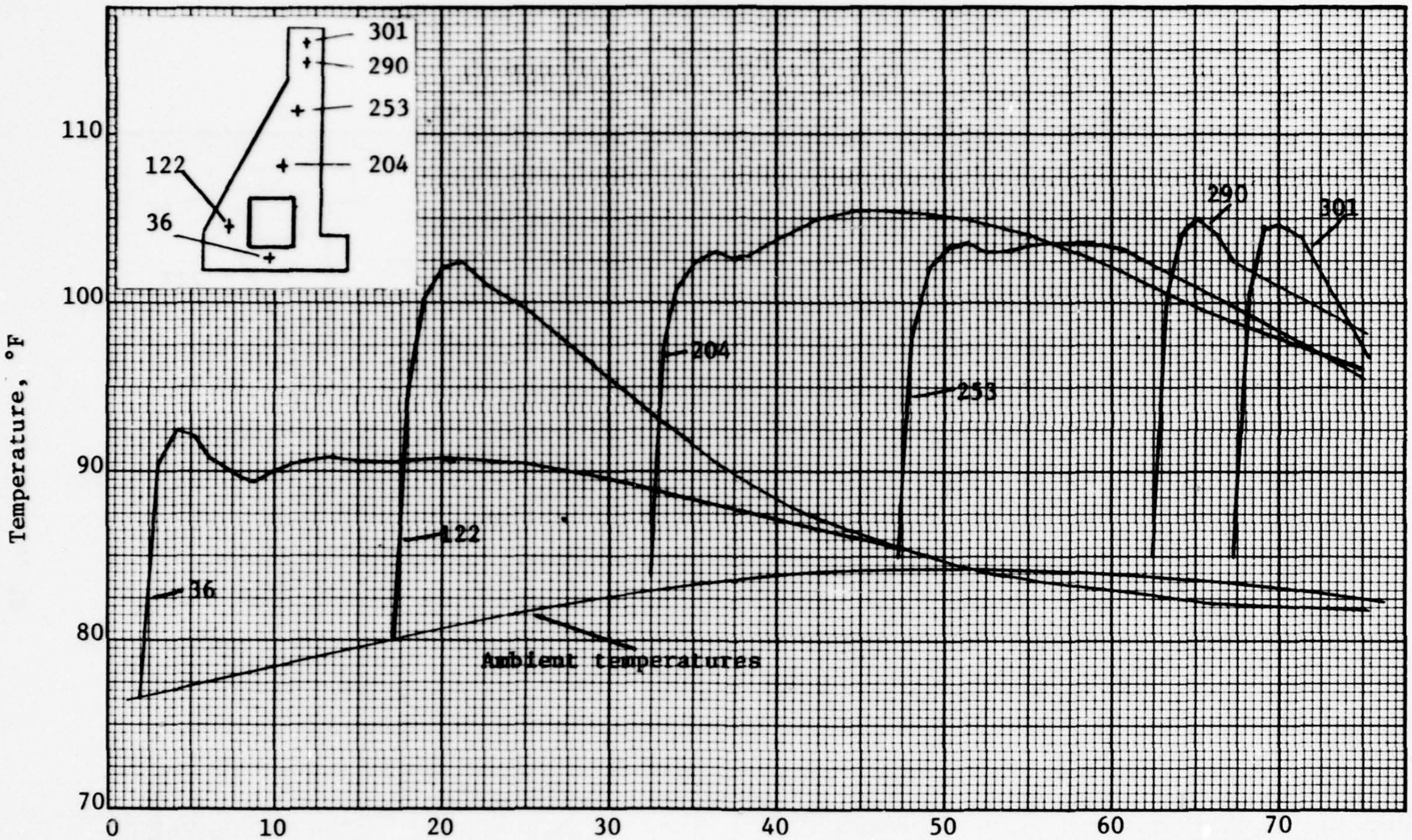

Figure 35. Nodal temperature in lock wall for construction at 5 days per lift

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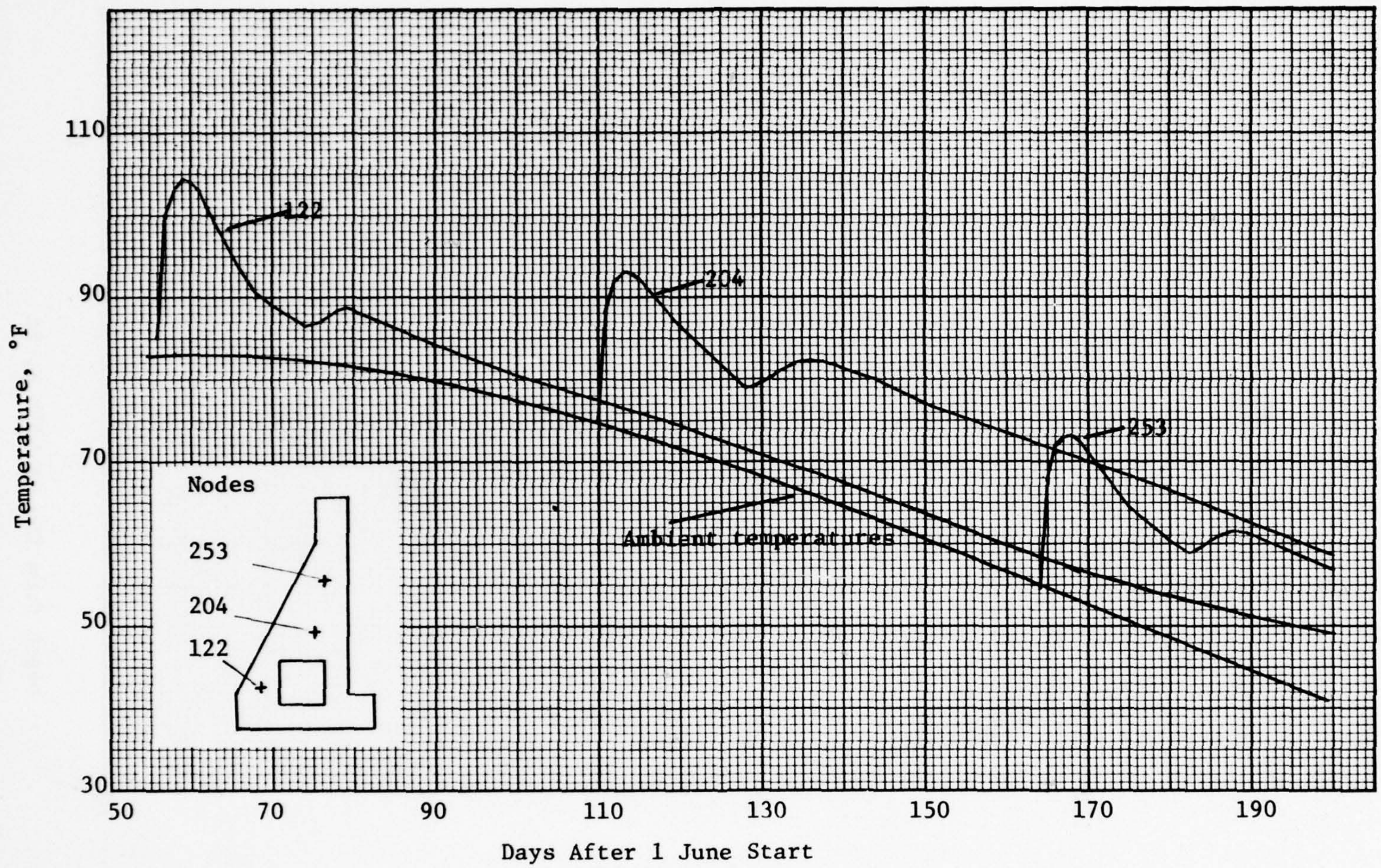

Figure 36. Nodal temperatures in lock wall at 18 days per lift rate

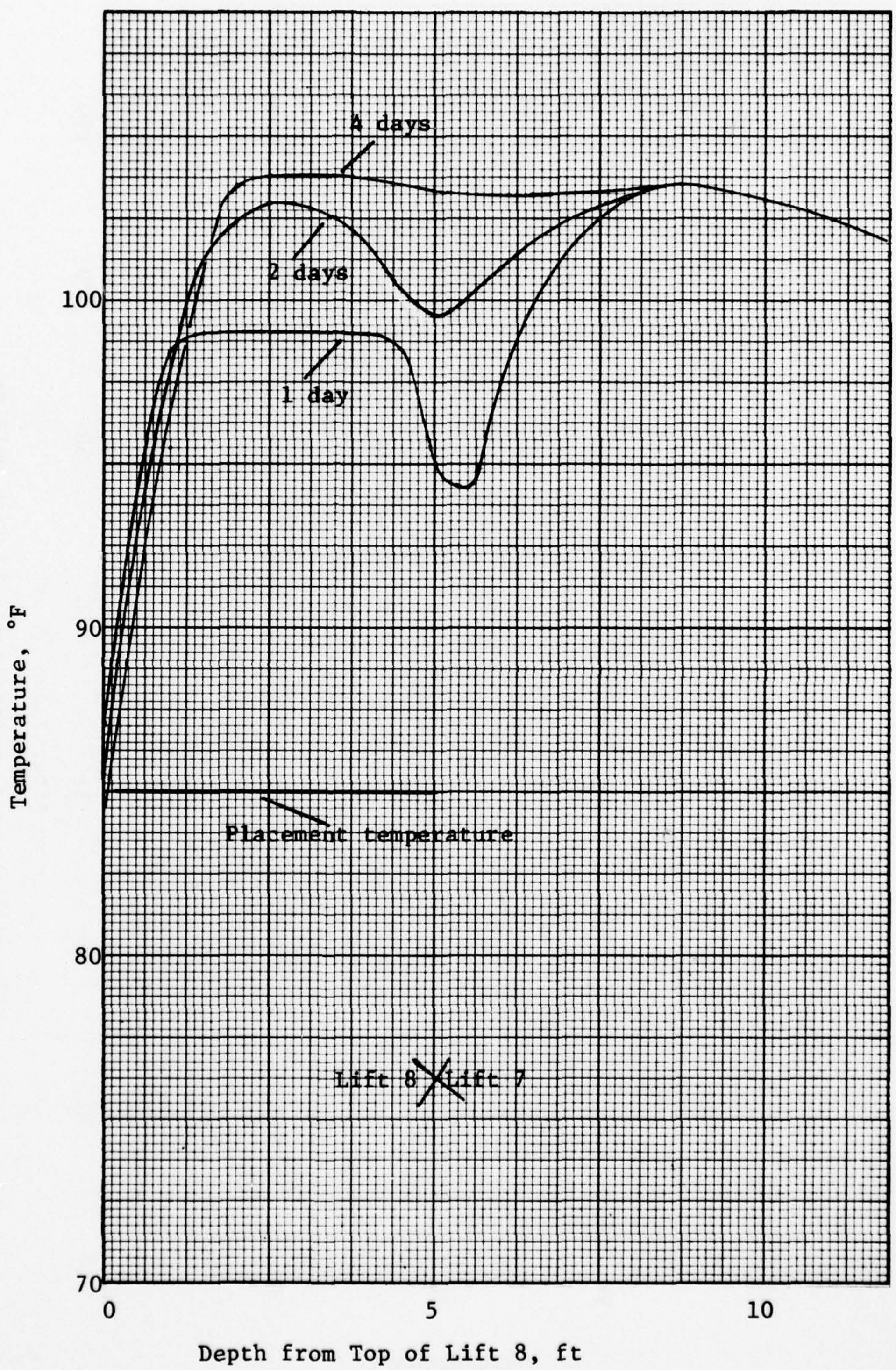

Figure 37. Thermal gradients in lock wall at 1, 2, and 4 days after placement of lift 8 in late July

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Bombich, Anthony A
Concrete temperature control studies - Tennessee-Tombigbee waterway projects / by Anthony A. Bombich, Billy R. Sullivan, James E. McDonald. Vicksburg, Miss. : U. S. Waterways Experiment Station, 1977.
36, [42] p. ; 27 cm. (Miscellaneous paper - U. S. Army Engineer Waterways Experiment Station ; C-77-8)
Prepared for U. S. Army Engineer District, Mobile, Mobile, Alabama.
References: p. 36.

1. Computerized simulation. 2. Concrete mixtures. 3. Concrete temperature. 4. Concrete thermal properties. 5. Finite element method. 6. Temperature control. 7. Tennessee-Tombigbee Waterway. I. Sullivan, Billy R., joint author. II. McDonald, James E., joint author. III. United States. Army. Corps of Engineers. Mobile District. IV. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Miscellaneous paper ; C-77-8.
TA7.W34m no.C-77-8