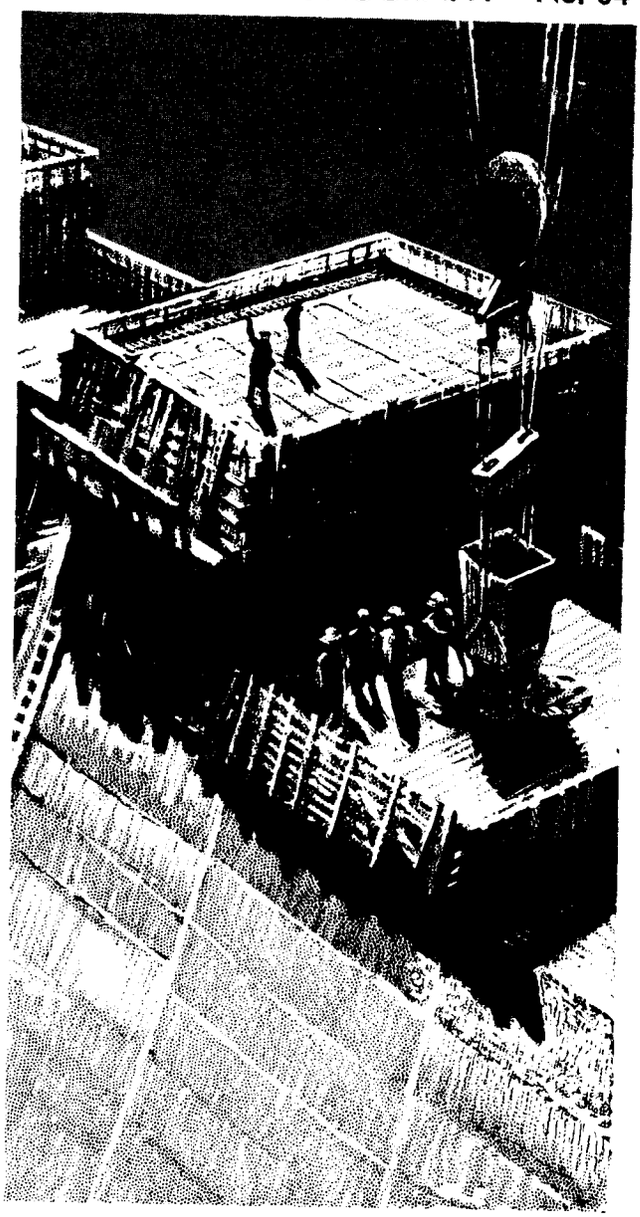
A WATER RESOURCES TECHNICAL PUBLICATION

ENGINEERING MONOGRAPH No. 34



Control of Cracking in Mass Concrete Structures

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UNITED STATES DEPARTMENT
OF THE INTERIOR
BUREAU OF RECLAMATION

A WATER RESOURCES TECHNICAL PUBLICATION

Engineering Monograph No. 34

Control of Cracking in Mass Concrete Structures

by

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Dams Branch
Division of Design
Engineering and Research Center

UNITED STATES DEPARTMENT OF THE INTERIOR
Bureau of Reclamation

As the Nation's principal conservation agency, the Department of the Interior has responsibility for most of our nationally owned public lands and natural resources. This includes fostering the wisest use of our land and water resources, protecting our fish and wildlife, preserving the environmental and cultural values of our national parks and historical places, and providing for the enjoyment of life through outdoor recreation. The Department assesses our energy and mineral resources and works to assure that their development is in the best interests of all our people. The Department also has a major responsibility for American Indian reservation communities and for people who live in Island Territories under U.S. administration.

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Preface

Various Bureau of Reclamation papers and memoranda on heat flow and temperature control of mass concrete have been written during the past 25 years. Several of these writings were concerned only with the mathematical relationships of heat flow. Others either gave general considerations to be taken into account in design and construction, or described a particular instance of artificial cooling and the results obtained. Because of the limited availability of these data and memoranda and because additional experiences in cooling of mass concrete structures have modified, in some instances, the application of temperature control measures, this Engineering Monograph was written to summarize studies and procedures as they relate to present day practice.

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Introduction

CRACKING in mass concrete structures is undesirable as it affects the water-tightness, durability, appearance, and internal stresses of the structures. Cracking in mass concrete will normally occur when tensile stresses are developed which exceed the tensile strength of the concrete. These tensile stresses may occur because of imposed loads on the structure, but more often occur because of restraint against volumetric change. The largest volumetric change in mass concrete results from change in temperature. Owing to the heat of hydration of the cementing materials in mass concrete, this change in temperature may be relatively large. Control of the subsequent temperature drop may be necessary if cracking is to be prevented or minimized.

Temperature control measures are adopted, as necessary, to minimize cracking and/or to control the size and spacing of cracks. As referred to here, cracks include the controlled crack, which is more commonly referred to as a contraction joint. These cracks occur when various parts of the mass concrete are subjected to volumetric changes due to temperature drop, drying shrinkage, and/or autogenous shrinkage. Such cracks vary from extremely small or hairline surface cracks which penetrate only a few inches into the mass, to irregular structural cracks of varying width which completely cross construction blocks, to the regular contraction joint with a relatively uniform opening which separates the construction blocks in a concrete monolith.

Trial-load analyses are used to shape and proportion the dam properly to the site for the prescribed loading conditions. Temperature control studies furnish data for these trial-load analyses. Basically, to function in the manner designed, an arch dam must be a continuous structure from abutment to abutment. Similarly, a gravity dam with longitudinal joints must have a monolithic section in an upstream-downstream direction. Provision for the construction of these structures must include measures by which the concrete is cooled and measures by which the transverse and longitudinal contraction joints are permanently closed by grouting before the reservoir loads are applied. The more widely-used temperature control measures to assure continuity within a structure are precooling of the concrete, artificial cooling of the concrete by means of embedded pipe systems, use of a construction slot, or a combination of these methods. The most commonly used of these measures for dams designed by the Bureau of Reclamation is the

pipe-cooling method, supplemented where necessary by the precooling method.

Early studies are based on existing data and on a possible construction schedule. Actual exposure conditions, water temperatures, and construction progress may vary widely from the conditions assumed, and adjustments are made during the construction period. Construction procedures and practices are anticipated during the design and specifications stage. Revisions and amendments are made during construction, when necessary, to obtain the best structure possible consistent with economy and good construction practices.

Volumetric Changes in Mass Concrete

Volume change, as referred to herein, applies to expansion and contraction of hardened concrete. Wetting, drying, and temperature variations are the principal causes of these expansions and contractions, although the chemical interaction between certain minerals and the alkalies inherent in cement may also contribute to volume change.

In the past, relatively large volumetric changes in several structures have resulted from the interaction between the aggregates and the alkalies in cement. Volume change in concrete due to alkali-aggregate reaction has been reduced in present day structures as a result of laboratory investigations of the concrete aggregates. Selection of nonreactive aggregates and/or the use of low-alkali cements now controls this type of volume change.

Autogenous volume change is usually a shrinkage and is entirely a result of chemical reaction within the concrete itself. Autogenous shrinkage is highly dependent upon the characteristics and amount of the cementing materials. The magnitude of the shrinkage varies widely. At Norris Dam, for example, measurements showed up to 90 millionths of an inch per inch autogenous shrinkage. At Grand Coulee Dam, about 60 millionths of an inch per inch was noted. Although of unknown magnitude, the autogenous shrinkage at Friant Dam was of sufficient magnitude and continued over such a period of time as to require regrouting of the contraction joints.

The volumetric change due to temperature can be controlled within reasonable limits and can be made a part of the design of a structure. Although the final state of temperature equilibrium depends on site conditions, a degree of control over these volume changes can be effected by limiting the amount of temperature rise which occurs immediately after placement and/or by controlling the placing temperature of the concrete. The temperature rise may be controlled by embedded pipe cooling systems, placement in shallow lifts, or by the use of a concrete mix designed to limit the heat of hydration. The last procedure is normally accomplished through a reduction in the amount of cement, use of a low-heat cement, use of a pozzolan to replace part of the cement, use of admixtures, or a combination of these factors. The placing temperature can also be varied, within limits, by precooling measures which lower the temperatures of one or more of the ingredients of the mix before batching. This will proportionately reduce the overall volume change.

Drying shrinkage can cause, as a skin effect, hairline cracks on the surface of a massive structure. This occurs because of the restraint the interior mass exerts on the surface layers. It is, however, negligible as a cause of volumetric change in mass concrete. The primary objection to these random hairline cracks of limited depth is that they may provide the entry for further and more extensive cracking and spalling under adverse exposure conditions. Proper curing applied during the early age of the concrete should therefore be used and thought of as a crack prevention measure.

The Temperature Control Problem

The method and extent of temperature control is governed by site conditions and the structure itself. The ideal condition would be to place the concrete at such a temperature that the subsequent hydration of the cement combined with exposure temperatures would cause a rise in temperature as the structure assumes its final stable temperature.[1]

[1] Numerous references are made, as above, to the final stable temperature of a concrete structure. Such a condition does not exist as a single, definable temperature. A final state of temperature equilibrium will eventually exist in which the temperature at any given point in the structure, or the mean temperature across a given section, will fluctuate between certain limits. Because of these varying temperatures and ever-changing temperature distributions, the term "final stable temperature" should be thought of as an approximate or average condition of temperature.

This would result in no volumetric temperature shrinkage, and any subsequent cracking would be associated with foundation settlements, loadings, or localized stresses. This condition was closely approximated by precooling measures at Vaitarna Dam in India and Klang Gates Dam in Malaya where relatively high final stable temperature conditions exist. Temperature conditions in the United States, however, are not conducive to such a temperature treatment. Most of the sites in this country have a final stable temperature in the order of 40° F. to 60° F. To allow for a moderate temperature rise in these instances would require below-freezing placing temperatures. Since the ideal condition cannot be realized, maximum concrete temperatures will be above the final stable temperature and a drop in temperature must be considered. Unless otherwise noted, all statements made herein will be directed toward structures which cannot be placed or constructed under the ideal condition.

In practically all mass concrete structures, contraction joints are provided to allow for volume changes caused by temperature drops. If the structure is an arch dam, contraction joint grouting will be required if the arches are to be monolithic and are to transfer the reservoir load to the abutments by arch action without excessive deflection. An arch dam, therefore, will normally require that an embedded pipe system be used to cool the concrete artificially, either during the construction of the dam or immediately afterward, so that the contraction joints will be open and can be grouted before the reservoir is filled. In some instances, as with extremely thin arch dams, natural cooling over a winter period will accomplish the same effect.

The gravity-type structure with no longitudinal contraction joints requires only that degree of temperature control necessary to prevent structural cracking circumferentially across the block as the block cools and approaches its final stable temperature. Since open longitudinal joints would prevent a gravity block carrying its load as a monolith, gravity-type dams with longitudinal contraction joints must be cooled by an embedded pipe system and the longitudinal contraction joints grouted before the full reservoir load is applied to the dam. The precooling method of temperature control and the use of low-heat cements, reduced cement content, and pozzolans will normally be used for gravity-type dams containing no longitudinal joints. The degree of precooling and/or the design of the concrete mix is generally based on the size of the block. Normally, a 25° to 30° temperature drop can be allowed to take place in the size of block now being used in gravity dams before tensile stresses great enough to cause cracking across the block will develop. This allowable drop should be less if severe exposure temperatures can be expected while the interior concrete temperatures are relatively high.

Applications

The need for controlling concrete temperatures undoubtedly was recognized by the Bureau of Reclamation soon after the first large concrete structures were built. In subsequent large structures, the fact that the temperature of concrete rises an appreciable amount above its placing temperature was either ignored, which led to unfavorable cracking in many instances, or a construction program was adopted which minimized the effects of the temperature changes. The latter was usually accomplished either by placing the concrete at a reduced rate, by placing in shallow lifts, or by the use of closure slots.

During the construction of Gibson (1929–30) and Ariel (1930) Dams, actual concrete temperatures were obtained which provided data on heat generation and the subsequent rise of concrete temperatures during the early age of concrete. An experimental artificial cooling installation was used in Ariel Dam where closure blocks were cooled by circulating water through vertical cored holes, 5 to 12 inches in diameter. At about the same time, early designs for Hoover Dam indicated that a definite control of concrete temperatures would be required because of the unprecedented size of that structure, and studies were made as to how this could best be accomplished. These studies included the air-cooled slot method, use of precooled aggregates, use of precast concrete blocks, and the embedded pipe method.

As a part of the Hoover Dam studies, an experimental installation using 1-inch pipes at 4-foot 8-inch horizontal spacing on the tops of 4-foot lifts was made in a small section of Owyhee Dam, then under construction. The results of this artificial cooling test checked the mathematical theory and

proved the embedded cooling coils to be a practical means of controlling concrete temperatures. As a result of the mathematical studies and the Owyhee tests, the pipe cooling method was adopted for the temperature control of Hoover Dam.

Temperature control through artificial cooling proved so successful at Hoover Dam, the Bureau of Reclamation has used essentially the same system on succeeding dams. In addition to reducing the maximum concrete temperatures, these embedded pipe cooling systems provided the means of cooling so that the transverse and longitudinal contraction joints could be grouted during or immediately after the construction period.

The layout of the concrete cooling system as used at Hoover Dam is basically the same as that used today. It consists of pipe or tubing placed in grid-like coils over the entire top surface of each 5- or 7½-foot lift of concrete. Coils are formed by joining together lengths of 1-inch outside diameter, thin-wall metal pipe or tubing; the maximum length of coil is normally between 800 and 1,200 feet in length. The number of coils in a block depends on the size of the block and the horizontal spacing. The horizontal spacing of the pipe or tubing varies between 2½ feet and 6 feet, depending on exposure conditions, length of block, height of lift, and proximity of foundation or abutment. Figures 1 and 2 show cooling details for Glen Canyon Dam, and Figure 3 shows similar details for Monticello Dam.

The velocity of flow of the cooling water through the embedded coils is normally required to be not less than 2 feet per second (about 4 gallons per minute). Cooling water is usually pumped through the coils, although a gravity system may sometimes be used to advantage. When river water is used, the warmed water is usually wasted after passing through the coils. However, there have been instances where to avoid use of river water having a high percentage of solids, the warmed water was cooled and recirculated. When using refrigerated water, the warmed water is normally returned to the water coolers in the refrigerating plant, recooled, and then recirculated.

The temperature of the concrete, for control of the cooling operations, is determined by resistance-type thermometers embedded in the concrete, by resistance-type thermometers inserted into pipes embedded in the concrete, or by thermocouples. Embedded resistance thermometers should be embedded at midlift and the electrical cable extended to a terminal board where readings can be taken whenever desired. Thermometer tubes are normally lengths of cooling tubing with the embedded end crimped shut and which extend from near the center of a block at midlift to the downstream face. Insert-type resistance thermometers are inserted into these tubes to obtain the concrete temperatures. Thermocouples are placed in the fresh concrete near the center of a block at midlift height, and the lead wires from the thermocouples are carried to readily accessible points on the downstream face.

Aside from the embedded pipe cooling system, the most widely used method of temperature control is that which reduces the placing temperature of the concrete. This has varied from the simplest of expedients, such as at Bartlett Dam in Arizona where concrete placement was restricted to nighttime placement during the hot summer months, to a complete precooling treatment where a maximum temperature has been specified for the concrete as it is placed in the forms. The usual maximum temperature specified has been 50° F., but this has varied from a 65° F. maximum temperature limitation at Monticello Dam to a 45° F. temperature limitation at Donnells Dam. These temperature control limitations should not be confused with the 80° to 90° F. limitations on the placing temperature of concrete for some of the older dams. The 80° to 90° F. limitations were put in specifications for controlling the quality of the concrete rather than as a crack prevention measure.

Other measures have been in between that degree of control obtained by restricted time of placement and that obtained by a complete precooling of the materials making up the concrete mix. Placing temperatures at Davis Dam were reduced by the addition of up to 90 pounds of ice per cubic yard of concrete, along with the mix water being cooled to 40° to 45° F. On several projects, sprinkling of the coarse aggregate piles was the only precooling measure employed. Although not solely a cooling measure, water curing has a definite effect on concrete temperatures. Fog spraying was used at Bartlett Dam during the

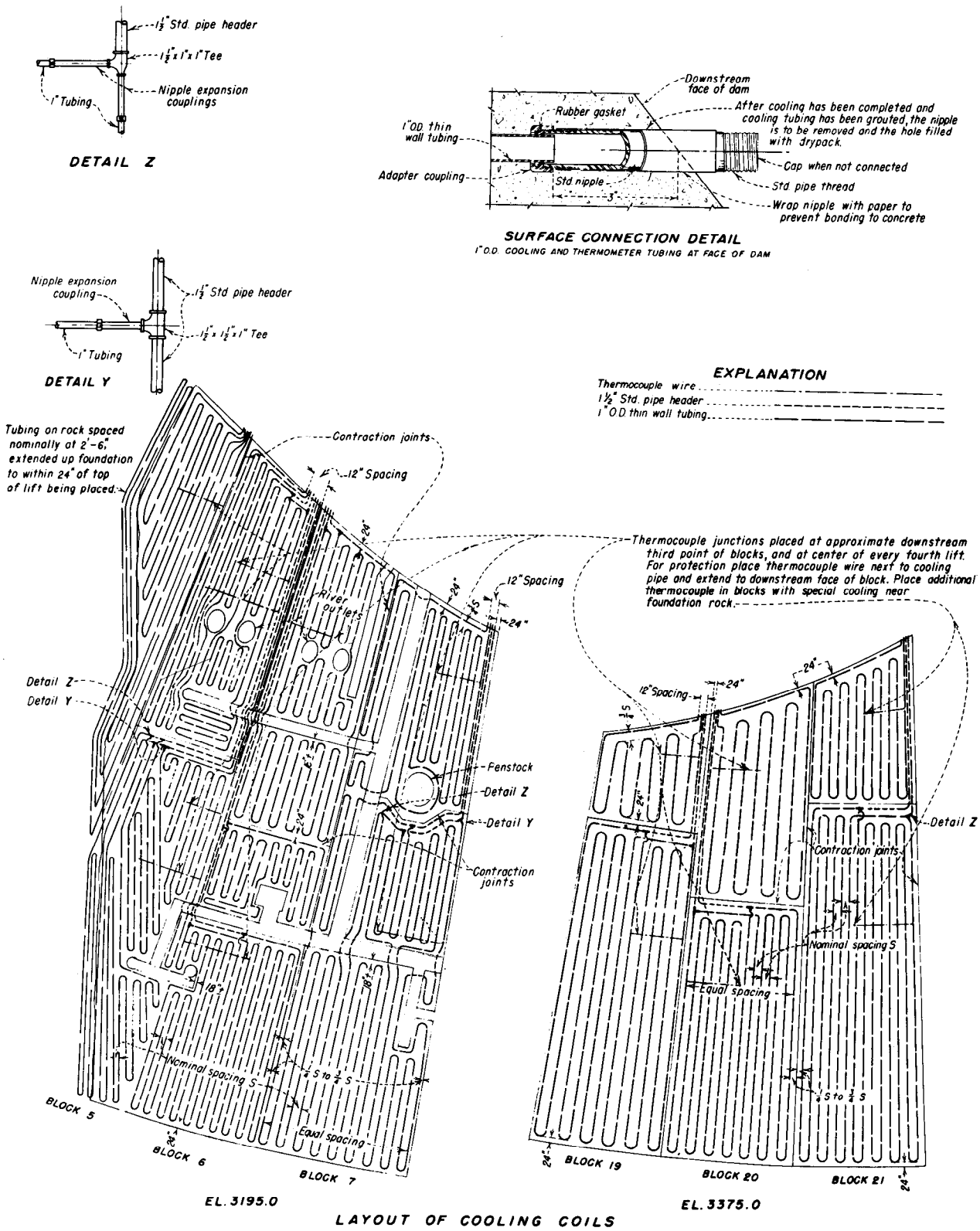
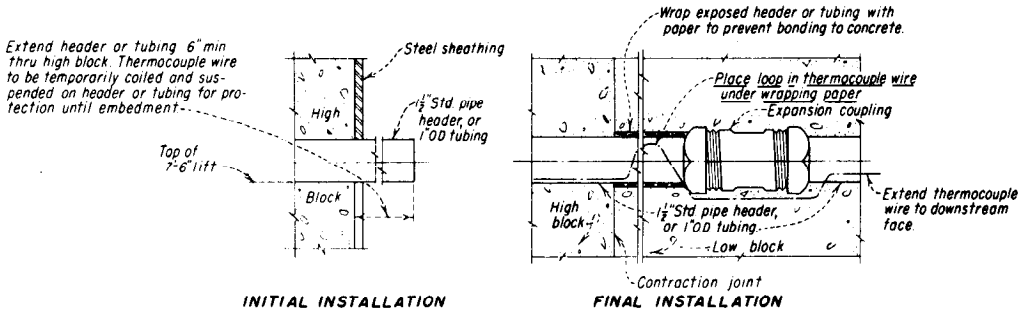


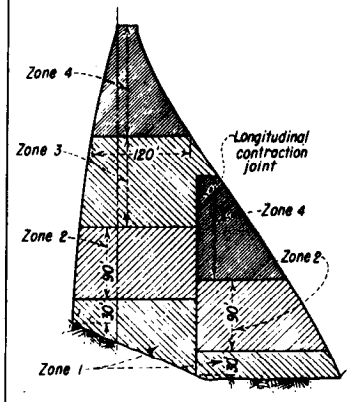
FIGURE 1.—*Glen Canyon Dam—cooling pipe layout.*



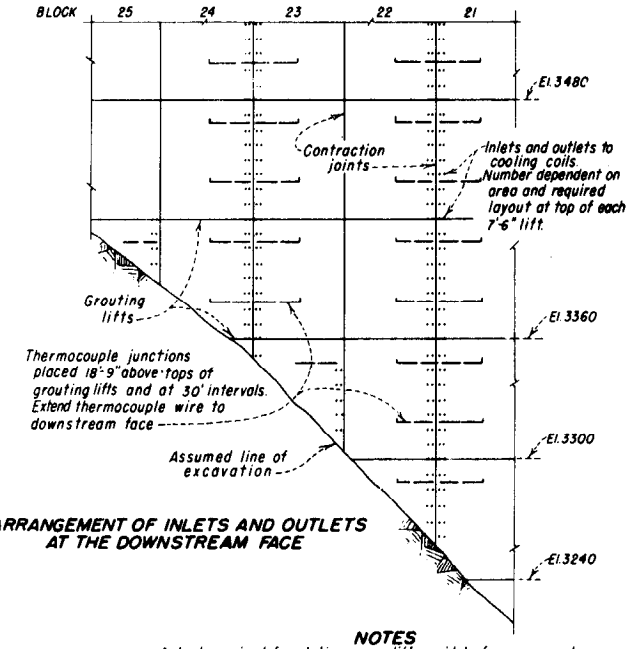
TYPICAL EXPANSION COUPLING
SHOWING CONTRACTION JOINT CROSSING

HORIZONTAL SPACING OF COOLING TUBING (CRITERIA TO BE APPLIED TO EACH BLOCK)

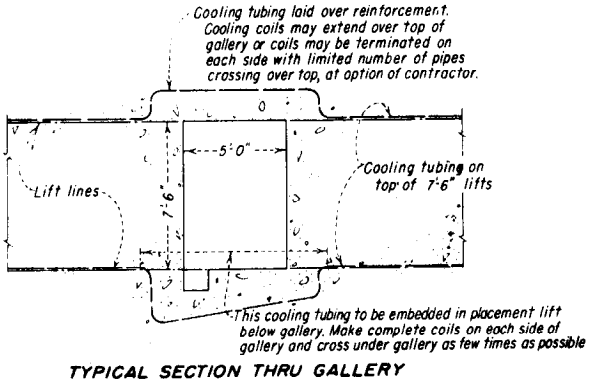
ZONE	DESCRIPTION	MAXIMUM NOMINAL SPACING S — CONCRETE PLACED: NOV. JAN. DEC. FEB.	MAXIMUM NOMINAL SPACING S — CONCRETE PLACED: ALL OTHER MONTHS
1	Between the foundation and 30' above the highest point of the foundation	3'-0" on all concrete, 2'-6" along foundation rock	3'-0" on all concrete, 2'-6" along foundation rock
2	Between 30' and 120' above the highest point of the foundation	5'-0"	4'-0"
3	Greater than 120' above the highest point of the foundation and where the 7'-6" lift is longer than 120'	6'-0"	5'-0"
4	Greater than 120' above the highest point of the foundation and where the 7'-6" lift is 120' in length or less	6'-0"	6'-0"



TYPICAL SECTION THRU DAM



ARRANGEMENT OF INLETS AND OUTLETS AT THE DOWNSTREAM FACE



TYPICAL SECTION THRU GALLERY

NOTES

Actual required foundations may differ widely from assumed excavation lines shown.
Cooling coils shall be placed, on top of each 7'-6" concrete lift.
Place tubing on all rock surfaces to within 24" of the top of the lift being placed.
Cooling tubing to be placed to clear openings in dam a min. of 18" or as directed.
Expansion couplings shall be used at contraction joint crossings.
Where tubing is installed for thermometer wells, the embedded end of the tubing is to be flattened and crimped to seal against grout leakage.
Arrangement of tubing may vary from that shown. The actual arrangement of the tubing in the structure shall be as directed.
Where a block is bounded by the downstream face and requires two or more coils, the contractor may elect to terminate all coils at downstream face in lieu of using 1½" headers. Upstream blocks requiring two or more coils will require headers.
Tubing placed on rock to be spaced at 2'-6"; tubing at top of each 7'-6" lift to be spaced according to zones as shown in table.
Coils placed in Zone 1 shall be approximately 800' in length with no coil longer than 900' in length; all other coils shall be approximately 1200' in length with no coil longer than 1300' in length. Adjacent coils served by the same header shall be as nearly the same length as possible.
Thermometer wells will be used to determine concrete temperatures at locations directed to supplement or replace thermocouple wire.
Cooling tubing placed within 25 to 30 feet of the foundation rock shall, where practicable, be placed as separate coils or with separate headers to facilitate special cooling in this area.
Each block shall have an independent cooling system at each concrete lift.

FIGURE 2.—*Glen Canyon Dam—concrete cooling details.*

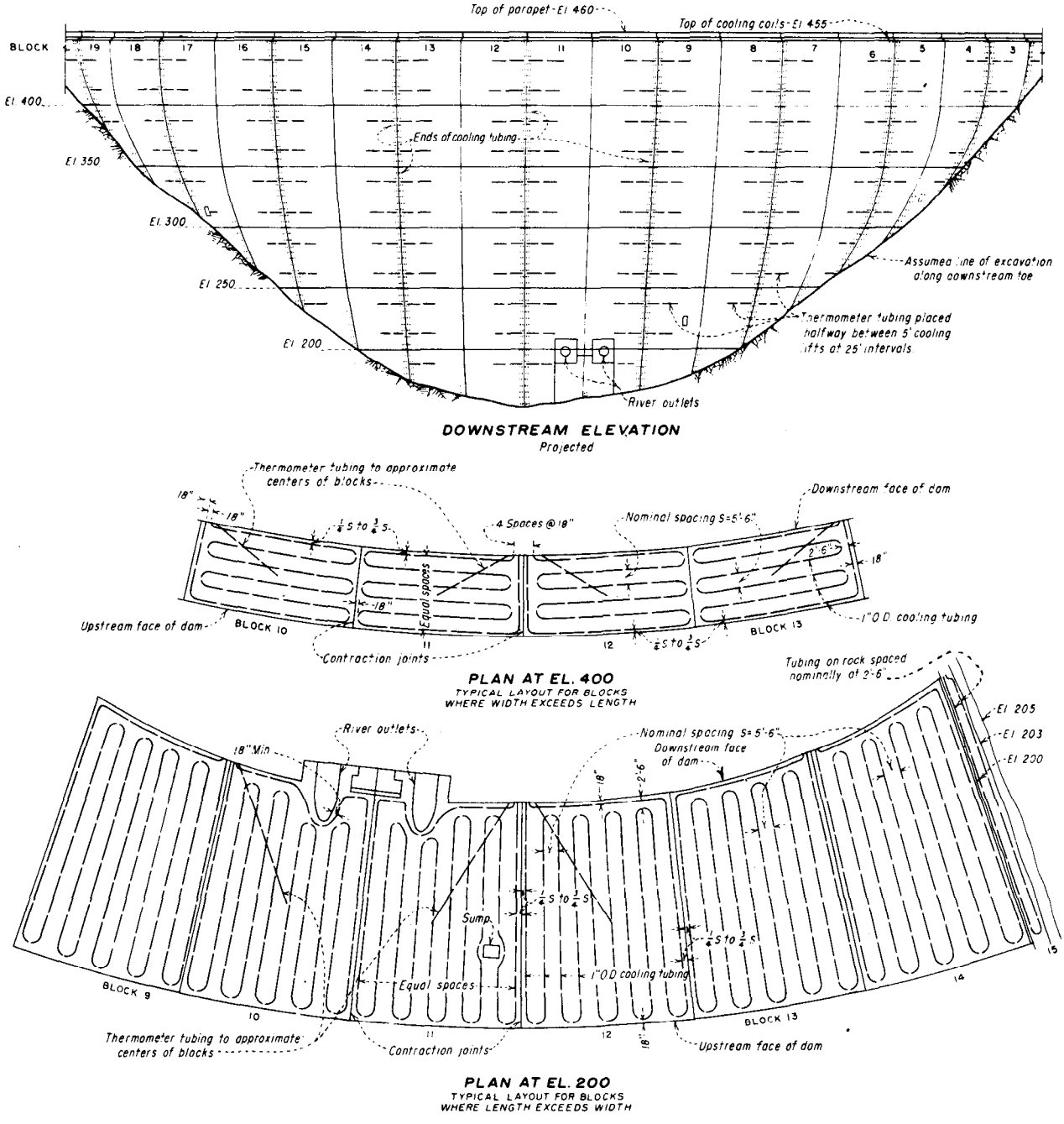


FIGURE 3.—*Monticello Dam—cooling pipe layout.*

summer months to cool the tops of the newly placed lifts.

Complete temperature treatments, over and above the use of embedded pipe cooling and precooling, have been used in some structures. In these instances, reductions were made in the amount of cement used, low-heat cements were specified, or effective use was made of a pozzolan to replace part of the cement to lower the temperature rise. Special cements, low-cement contents, precooling measures, and embedded pipe cooling systems were used in the base blocks of Detroit Dam, constructed by the Corps of Engineers. Glen Canyon Dam, because of the size of the blocks in the dam and the relatively low grouting temperature, was constructed with a 50° F. maximum placing temperature, embedded cooling coils, a Type II cement, and mix containing two sacks of cement and one sack of pozzolan.

Costs

Unit costs per cubic yard of concrete vary considerably depending on the size of the structure and the method or degree of control. Each of the pipe-cooling and precooling methods is costly, but the precooling will ordinarily cost slightly less than artificially cooling the concrete by use of embedded pipes.

The cost of all labor and materials chargeable to cooling at Hoover Dam (1931) was about $0.25 per cubic yard of concrete for an embedded pipe cooling system with refrigeration plant. At Shasta Dam (1937), the cost of furnishing and installing cooling systems and circulating river and refrigerated water through the coils was $0.21 per cubic yard. At Hungry Horse Dam (1948), the contractor was paid $0.49 per cubic yard for installing the embedded pipe systems and circulating river water through the embedded coils, and the cooling materials cost about $0.22 per cubic yard, for a total cooling cost to the Government of $0.71 per cubic yard of concrete. Costs at Flaming Gorge Dam (1963) totaled $0.85 per cubic yard for furnishing, installing, operating, and grouting the cooling systems. An additional $0.11 per cubic yard was paid to the contractor for sprinkling the aggregates for about 200,000 cubic yards of concrete placed during the 1961 summer period. Aside from cooling, the cost of furnishing and installing the grouting systems and grouting the contraction joints at Flaming Gorge Dam was almost $0.19 per cubic yard.

Because the cost of precooling is usually included in the cost per cubic yard of concrete, no summary can be made as to the actual cost per cubic yard for this method of temperature control. The degree of precooling necessary, of course, would cause the cost to vary considerably. At Detroit Dam, completed in 1953, the cost of precooling measures to obtain a placing temperature of 50° F. was $0.56 per cubic yard. Additional cooling through an embedded pipe system at the base of the dam amounted to $0.74 per cubic yard for the concrete cooled in that region. The total cost of temperature control, distributed over the total volume in Detroit Dam, was $0.63 per cubic yard of concrete.

Temperature Control Studies

General

THE measures required to obtain a monolithic structure and the measures necessary to reduce cracking tendencies to a minimum are determined by temperature control studies. Topographic conditions, accessibility of the site, length of construction period, and the size and type of dam will influence the size of blocks, rate of placement, and the temperature control and related crack prevention measures. Temperature control studies begin early in the design stage when programs are laid out to collect essential air and water temperature data. Such programs should obtain maximum and minimum daily air temperatures, maximum and minimum daily river water temperatures, and representative wet- and dry-bulb temperatures during the year.

In addition to the climatic conditions at the site, the period of flood runoff and reservoir operation requirements are studied to see what methods of temperature control would be adaptable to the conditions during the construction period. The ever-growing collection of data on the site, dam, and construction problems are gradually assembled and decisions made on the nature and extent of the concrete temperature control, closure or grouting temperature, reduction of temperature gradients near exposed faces, insulation, and other crack prevention measures.

Range of Mean Concrete Temperatures

The temperature and temperature distributions which exist at any given time are of interest in several design problems, but are not normally used as such in the trial-load analysis to determine temperature stresses in arch dams. Of primary interest in the trial-load analysis is the range or amplitude of the mean concrete temperature for each of the arches or voussoirs used in the analysis. This mean concrete temperature will be sufficient to determine temperature stresses for most arch dams. Temperature gradients which exist between the upstream and downstream faces may also be taken into consideration in some instances.

The average arch thickness from abutment to abutment is used in computing the range of mean

concrete temperature where the arch does not appreciably change thickness. For variable thickness arches, temperature ranges at the quarter-points should be computed. The number of arches for which these ranges of temperature are determined should be the same as the arches used in the trial-load analysis. The range of mean concrete temperature with reservoir full is the normal condition. When stage construction is taken into consideration, when the reservoir is to be filled or partially filled before concrete temperatures have reached their final state of temperature equilibrium, or when the operation of the reservoir is such that maximum and minimum reservoir levels occur at times other than extremes of temperature, further studies are prepared to determine mean concrete temperatures existing for the particular condition. Each study is numbered according to the trial-load study number to differentiate between later studies.

The range of mean concrete temperature for each theoretical arch or voussoir is determined from the air and water temperatures which are assumed to exist at the site, as modified by the effects of solar radiation. Thermal properties of the concrete establish the ability of the concrete to undergo temperature change.

Figures 4 to 7, inclusive, show the general features, climatic conditions, and actual ranges of reservoir water temperatures and mean concrete temperatures at Hungry Horse Dam in Montana and Monticello Dam in California.

Figures 8 and 9 show the computations for the range of mean concrete temperature at Hungry Horse Dam using assumed reservoir water and air temperatures. Explanations for the computations are given in the following discussions.

Diffusivity of Concrete.—The diffusivity of concrete, h^2, is an index of the facility with which concrete will undergo temperature change. Although desirable from the heat standpoint, it is not practicable to select aggregate, sand, and cement for a concrete on the basis of heat characteristics. The thermal properties of the concrete must therefore be accepted for what they are.

The value of the diffusivity of concrete is usually expressed in ft ^{2}/hr., and can be determined from the relationship $h^2=\frac{K}{C\rho}$ where K is the conductivity in B.t.u./ft /hr /° F., C is the specific heat in B.t.u./lb /° F., and ρ is the density in lb /cu ft. Values of the diffusivity for a given concrete are determined from laboratory tests, although they will normally be estimated for the earlier studies. As the thermal characteristics of the coarse aggregate govern to a large extent the thermal characteristics of the concrete, the earliest of these estimates can be based upon the probable type of coarse aggregate for preliminary studies. Table I gives thermal properties of concrete for various dams and for several coarse aggregates. Empirical factors of the contribution of each percent by weight of the various concrete materials on the thermal properties of concrete can also be used to estimate the thermal properties. These factors are presented in "Thermal Properties of Concrete," Bulletin No. 1, Part VII, of the Boulder Canyon Project Final Reports.

Amplitudes of Air Temperatures.—Any estimation of air temperatures which will occur in the future at a given site must be based on air temperatures which have occurred in the past, either at that location or in the near vicinity. The Weather Bureau has collected weather data at a great number of locations, and records from one or more of these locations may be selected and adjusted to the site. For this adjustment, an increase of 250 feet in elevation is assumed to decrease the air temperature 1° F. Similarly, an increase of 1.4° in latitude is assumed to decrease the temperature 1° F. A program for obtaining actual air temperatures at the site should be instituted as soon as possible to verify the above data. Figure 10 shows the data assembled for the start of the temperature studies on Glen Canyon Dam, along with actual data obtained at the site during the 3-year period immediately prior to construction.

Mean daily and mean annual air temperatures are used, as it would be physically impossible to apply theoretical day-to-day temperatures to the concrete. The theory developed applies these daily and annual air temperature cycles as sinusoidal variations of temperature, although the cycles are not true sine waves. The amplitudes of these two sine waves, with periods of 1 day (24 hours) and 365 days (8,760 hours), are obtained from the mean annual, the mean monthly, and the mean monthly maximum and mean minimum air temperatures. In the computation, the annual and

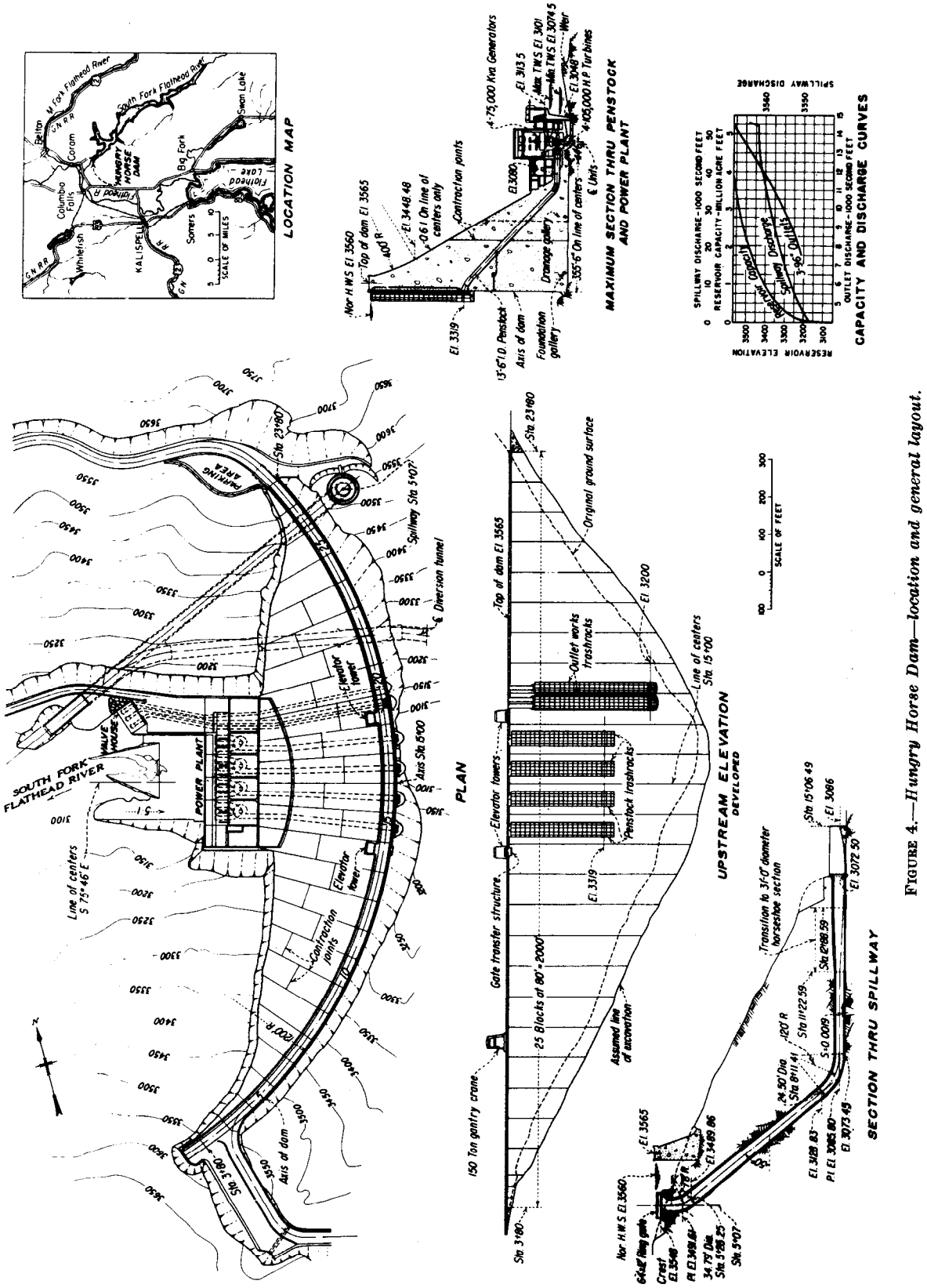


FIGURE 4.—*Hungry Horse Dam—location and general layout.*

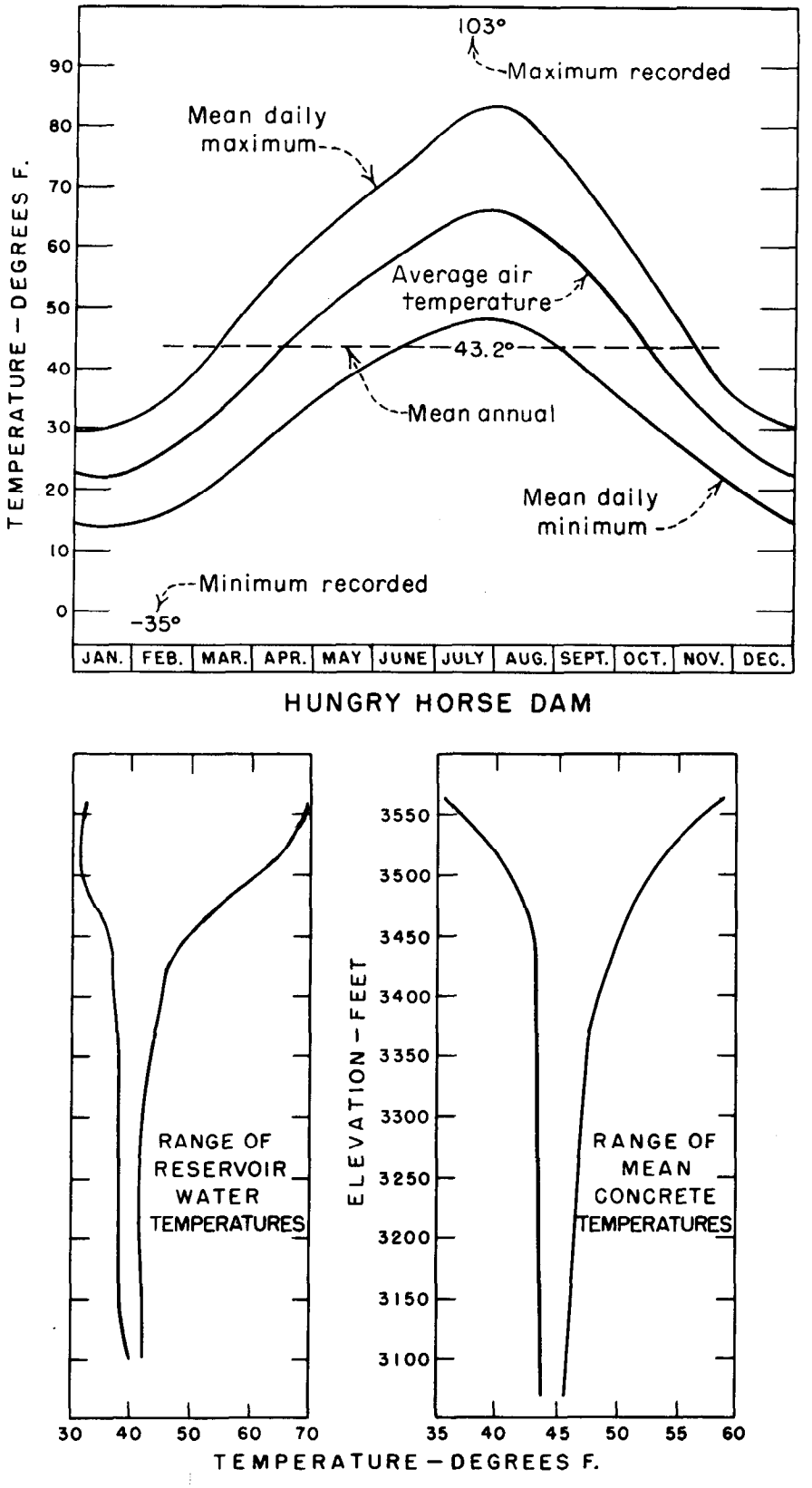


FIGURE 5.—*Hungry Horse Dam—climatic and mean concrete temperatures.*

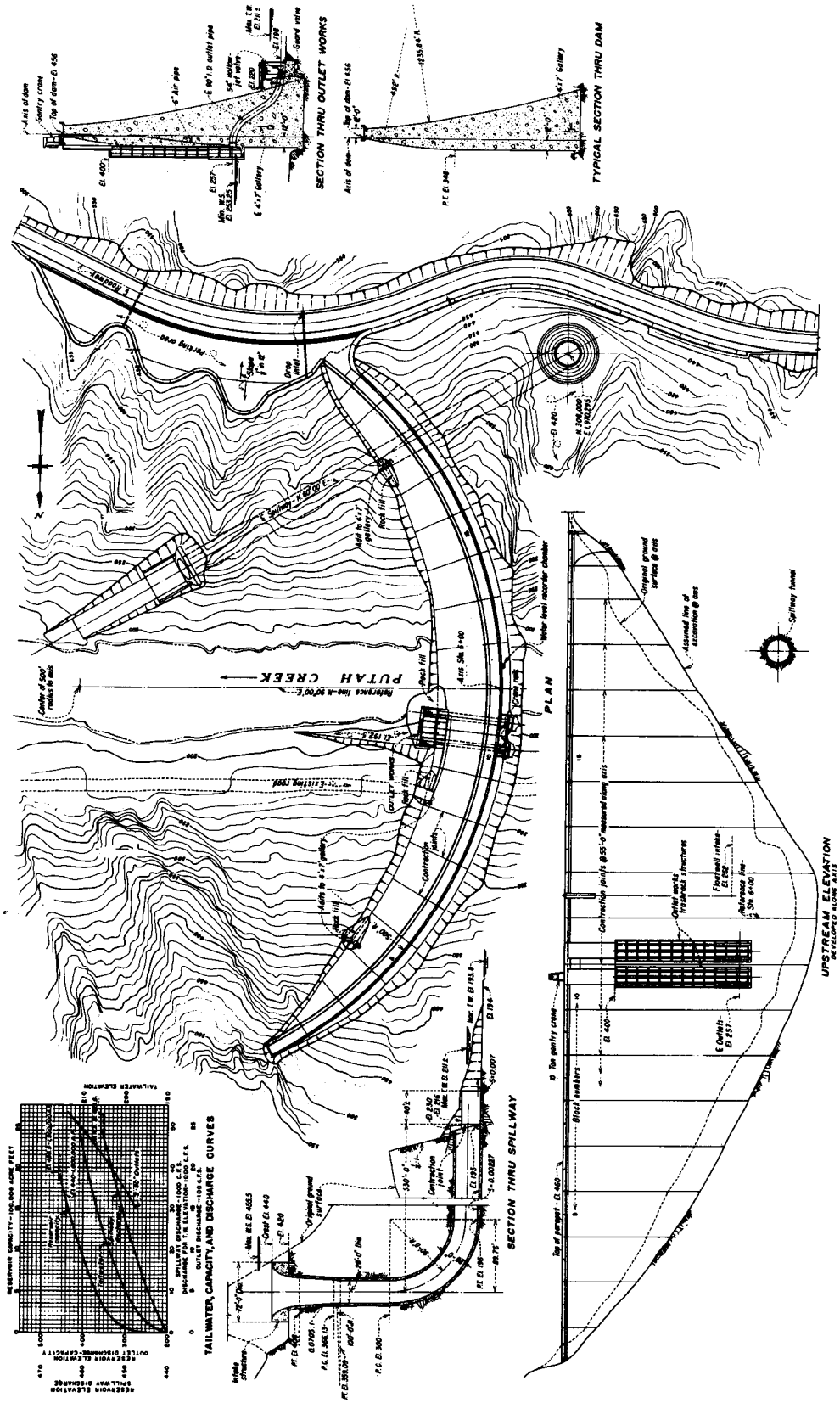


FIGURE 6.—*Monticello Dam—general layout.*

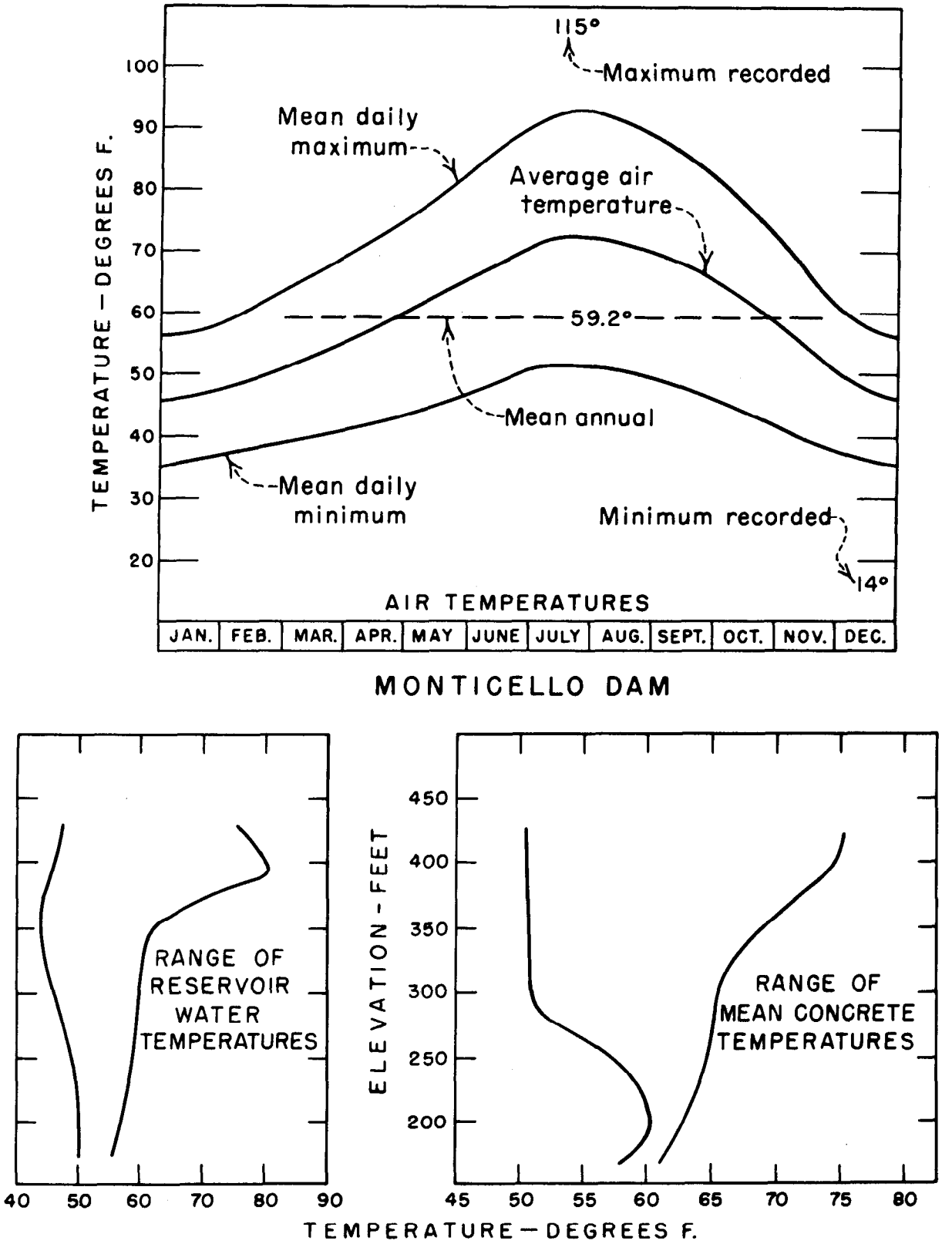


FIGURE 7.—*Monticello Dam—climatic and mean concrete temperatures.*

TEMPERATURE RANGE OF Hungry Horse DAM

(Effect of solar radiation not included)

For yearly change $\ell_1 = \frac{\ell_2}{\sqrt{h_4^2 \gamma_2}} = \frac{\ell_2}{\sqrt{0.053 \times 8760}} = 0.0464\,\ell_2$

For 365-hr change $\ell_1 = \frac{\ell_2}{\sqrt{0.053 \times 365}} = 0.227\,\ell_2$

For daily change $\ell_1 = \frac{\ell_2}{\sqrt{0.053 \times 24}} = 0.887\,\ell_2$

Remarks: Air temperatures taken from 34-year record at Columbia Falls, Montana. $h^2 = 0.053$ from laboratory data

Avg. Annual Air Temperature 43.2°F.	Extreme Weather Conditions		Usual Weather Conditions	
	Above Mean	Below Mean	Above Mean	Below Mean
Yearly	22.0	21.4	22.0	21.4
365-hr.	30.6	49.6	20.4	25.2
Daily	7.2	7.2	7.2	7.2

Elevation Ft.	Thickness of Dam Ft.	Due to Yearly Range ℓ_1	Ratio from Curve	Due to 365-hr. Range ℓ_1	Ratio from Curve	Due to Daily Range ℓ_1	Ratio from Curve	Exposed to air on both surfaces: Ext. Wea. Conditions Amplitude Above Mean	Ext. Wea. Conditions Amplitude Below Mean	Ext. Wea. Conditions Temperature Max.	Ext. Wea. Conditions Temperature Min.	Usual Wea. Conditions Amplitude Above Mean	Usual Wea. Conditions Amplitude Below Mean	Usual Wea. Conditions Temperature Max.	Usual Wea. Conditions Temperature Min.	Water Temperature Max.	Water Temperature Min.	Water Temperature Avg.	Water Temperature Amp.	Mean Conc. Temp. Exposed to Water on Both Faces Amp.	Mean Conc. Temp. Exposed to Water on Both Faces Max.	Mean Conc. Temp. Exposed to Water on Both Faces Min.	Air on D.S. Face Water on U.S. Face: Ext. Wea. Conditions Max.	Ext. Wea. Conditions Min.	Usual Wea. Conditions Max.	Usual Wea. Conditions Min.
3565	40	1.86	0.460	9.08	0.087	35.48	0.022	12.9	14.3	56.1	28.9	12.1	12.2	55.3	31.0	69.0	32.0	50.5	18.5	8.5	59.0	42.0	57.5	35.5	57.2	36.5
3550	39	1.81	.430	8.85	.089	34.59	.023	13.4	14.9	56.6	28.3	12.5	12.7	55.7	30.5	69.0	33.0	51.0	18.0	8.6	59.6	42.4	58.1	35.4	57.6	36.4
3500	56	2.60	.310	12.71	.063	49.67	.016	8.9	9.9	52.1	33.3	8.2	8.3	51.4	34.9	54.5	36.8	45.6	8.8	2.7	48.3	42.9	50.2	38.1	49.9	38.9
3450	81	3.76	.215	18.39	.043	71.85	.011	6.1	6.8	49.3	36.4	5.7	5.8	48.9	37.4	47.5	38.5	43.0	4.5	1.2	44.2	41.8	46.7	39.1	46.6	39.6
3400	111	5.15	.156	25.20	.032	-	-	4.4	4.9	47.6	38.3	4.1	4.1	47.3	39.1	45.0	39.0	42.0	3.0	0.5	42.5	41.5	45.0	39.7	44.9	40.3
3350	141	6.54	.122	32.01	.025			3.4	3.9	46.6	39.3	3.2	3.2	46.4	40.0	43.0	39.0	41.0	2.0	0.2	41.2	40.8	43.9	40.0	43.3	40.4
3300	171	7.93	.100	38.82	.021			2.8	3.2	46.0	40.0	2.6	2.7	45.8	40.5	43.0	39.0	41.0	2.0	0.2	41.2	40.8	43.6	40.4	43.5	40.6
3250	201	9.33	.085	45.63	.018			2.4	2.7	45.6	40.5	2.2	2.3	45.4	40.9	43.0	39.0	41.0	2.0	0.2	41.2	40.8	43.4	40.6	43.3	40.8
3200	231	10.72	.075	52.44	.015			2.1	2.3	45.3	40.9	2.0	2.0	45.2	41.2	43.0	39.0	41.0	2.0	0.2	41.2	40.8	43.2	40.9	43.2	41.0
3150	274	12.71	.063	62.20	.013			1.8	2.0	45.0	41.2	1.7	1.7	44.9	41.5	43.0	39.0	41.0	2.0	0.1	41.1	40.9	43.0	41.0	43.0	41.2
3100	292	13.55	.060	66.28	.012			1.7	1.9	44.9	41.3	1.6	1.6	44.8	41.6	43.0	39.0	41.0	2.0	0.1	41.1	40.9	43.0	41.1	42.9	41.3
3067	366	16.98	.047	83.08	-			1.0	1.0	44.2	42.2	1.0	1.0	44.2	42.2	43.0	39.0	41.0	2.0	0.1	41.1	40.9	42.7	41.5	42.7	41.5

EQUIVALENT THICKNESS FACTOR			
h_4^2	YEARLY	15-DAY	DAILY
0.036	0.0563	0.2759	1.0760
0.040	0.0534	0.2617	1.0206
0.044	0.0509	0.2495	0.9731
0.048	0.0488	0.2389	0.9317
0.052	0.0468	0.2295	0.8951
0.056	0.0452	0.2212	0.8626

NOTES

ℓ_2 = Thickness of dam, ft.

Curve referred to is "Temperature Variations of Flat Slabs Exposed to Sinusoidal Temperature Variations on Both Faces."

FIGURE 8.—*Computation form (sheet 1 of 2)—range of mean concrete temperatures.*

daily amplitudes are assumed to be the same for extreme and usual weather conditions.

To account for the maximum and minimum recorded air temperatures, a third and somewhat fictitious temperature cycle must be assumed. This third temperature variation is associated with the movements of barometric pressures across the country. Plots of mean daily air temperatures for stations in the vicinity of Hoover Dam indicated that about two cycles per month could be expected. Similar plots throughout the western part of the United States show from one to three cycles per month. Arbitrarily, therefore, the third temperature variation is set up as a sine wave with a 15-day (365-hour) period. By adding the three sine waves together, any and all recorded temperatures can be accounted for.

For extreme weather conditions, the amplitudes of the 15-day cycle are assigned numerical values which, when added to the amplitudes of the daily and annual cycles, will account for the actual maximum and minimum recorded air temperatures at the site. For usual weather conditions, these amplitudes are assigned values which account for temperatures halfway between the mean monthly maximum (minimum) and the maximum (minimum) recorded. Table II summarizes the above discussion and shows how the amplitudes of the above three temperature cycles are found.

Amplitudes of Concrete Temperatures.—The range or amplitude of concrete temperatures which result from air and water exposures are determined by applying assumed external sinusoidal temperature variations to the edges of a theoretical flat slab. The problem is idealized by assuming that the width of the slab is the same as the thickness of the dam at the elevation under consideration, and that no heat flows in a direction nor-

TEMPERATURE RANGE

OF Hungry Horse DAM

Effect of Solar Radiation included

Latitude 48° N

Remarks:

Elevation Ft.	Thickness of Dam Ft.	Effect of Solar Radiation US.	DS.	Avg.	½DS.	MEAN CONCRETE TEMPERATURES: Exposed to air on both faces, Ext. Wea. Conditions Max.	Min.	Usual Wea. Conditions Max.	Min.	Air on D.S. Face, Water on U.S. Face, Ext. Wea. Conditions Max.	Min.	Usual Wea. Conditions Max.	Min.
3565	40	6.1	4.3	5.2	2.2	61.3	34.1	60.5	36.2	59.7	37.6	59.4	38.7
3550	39	6.0	5.0	5.5	2.5	62.1	33.8	61.2	36.0	60.6	37.9	60.1	38.9
3500	56	5.8	5.6	5.7	2.8	57.8	39.0	57.1	40.6	53.0	40.9	52.7	41.7
3450	81	5.6	5.6	5.6	2.8	54.9	42.0	54.5	43.0	49.5	41.9	49.4	42.4
3400	111	5.4	5.4	5.4	2.7	53.0	43.7	52.7	44.5	47.7	42.4	47.6	43.0
3350	141	5.2	5.5	5.3	2.7	51.9	44.6	51.7	45.3	46.6	42.7	46.5	43.1
3300	171	5.0	5.4	5.2	2.7	51.2	45.2	51.0	45.7	46.3	43.1	46.1	43.3
3250	201	5.0	6.0	5.5	3.0	51.1	46.0	50.9	46.4	46.4	43.6	46.3	43.8
3200	231	4.8	4.7	4.8	2.3	50.1	45.7	50.0	46.0	45.5	43.1	45.5	43.3
3150	274	4.6	4.7	4.6	2.3	49.6	45.8	49.5	46.1	45.3	43.3	45.3	43.5
3100	292	4.4	4.7	4.5	2.3	49.4	45.8	49.3	46.1	45.3	43.4	45.2	43.6

SOLAR RADIATION VALUES

Elevation Ft.	Terrain Factor %	131° Point 1 49° Upstream Normal angle Slope	Temp. Rise 100%	Actual	Downstream Normal angle Slope	Temp. Rise 100%	Actual	106° Point 2 74° Upstream Normal angle Slope	Temp. Rise 100%	Actual	Downstream Normal angle Slope	Temp. Rise 100%	Actual	81° Point 3 99° Upstream Normal angle Slope	Temp. Rise 100%	Actual	Downstream Normal angle Slope	Temp. Rise 100%	Actual	Average Temp. Rise U.S.	D.S.
3565	100	0	7.3	7.3	0	2.7	2.7	0	6.3	6.3	0	4.3	4.3	0	4.8	4.8	0	6.0	6.0	6.1	4.3
3550	97	0	7.3	7.1	0.25	3.4	3.3	0	6.3	6.1	0.25	5.1	4.9	0	4.8	4.7	0.25	7.1	6.9	6.0	5.0
3500	94	0	7.3	6.9	.5	4.0	3.8	0	6.3	5.9	.5	6.0	5.6	0	4.8	4.5	.5	7.9	7.4	5.8	5.6
3450	91	0	7.3	6.6	.6	4.2	3.8	0	6.3	5.7	.6	6.2	5.6	0	4.8	4.4	.6	8.1	7.4	5.6	5.6
3400	88	0	7.3	6.4	.6	4.2	3.7	0	6.3	5.6	.6	6.2	5.5	0	4.8	4.2	.6	8.1	7.1	5.4	5.4
3350	85	0	7.3	6.2	.87	4.9	4.2	0	6.3	5.4	.6	6.2	5.3	0	4.8	4.1	.6	8.1	6.9	5.2	5.5
3300	82	0	7.3	6.0	.87	4.9	4.0	0	6.3	5.2	.6	6.2	5.1	0	4.8	3.9	.82	8.6	7.1	5.0	5.4
3250	79							0	6.3	5.0	.6	6.2	4.9	0	4.8	3.8	.97	8.8	7.0	5.0	6.0
3200	76							0	6.3	4.8	.6	6.2	4.7							4.8	4.7
3150	73							0	6.3	4.6	.7	6.5	4.7							4.6	4.7
3100	70							0	6.3	4.4	.8	6.7	4.7							4.4	4.7

FIGURE 9.—*Computation form (sheet 2 of 2)—range of mean concrete temperatures.*

mal to the slab. The law of superposition is used in that the final amplitude in the concrete slab is the sum of the amplitudes obtained from the external temperature variations. For a slab exposed to air on both faces, three amplitudes representing daily, 15-day, and annual cycles are applied to the slab. Although not strictly true at the reservoir surface, the amplitude of the reservoir water temperature is assumed to have an annual cycle.

To apply the results of theoretical heat-flow studies in a practical and usable form, unit values were assumed for the several variables and a curve was obtained showing the ratio of the variation of the mean temperature of the slab to the variation of the external temperature. Figure 11 shows the relationships thus derived for temperature variations in flat slabs exposed to sinusoidal temperature variations for $h^2=1.00$ ft ^{2}/day, a period of 1 day, and a thickness of slab of l_1. To use this figure, a correlation equation is given by which l_1 can be computed for any other combination of actual thickness of dam, diffusivity constant, and period.

The computation shown in Figure 8 follows the above theory. From the actual thickness of dam, l_2, a value of l_1 is obtained from the correlation equation for each of the air temperature cycles. For each value of l_1, a ratio of the variation of mean concrete temperature to the variation of ex-

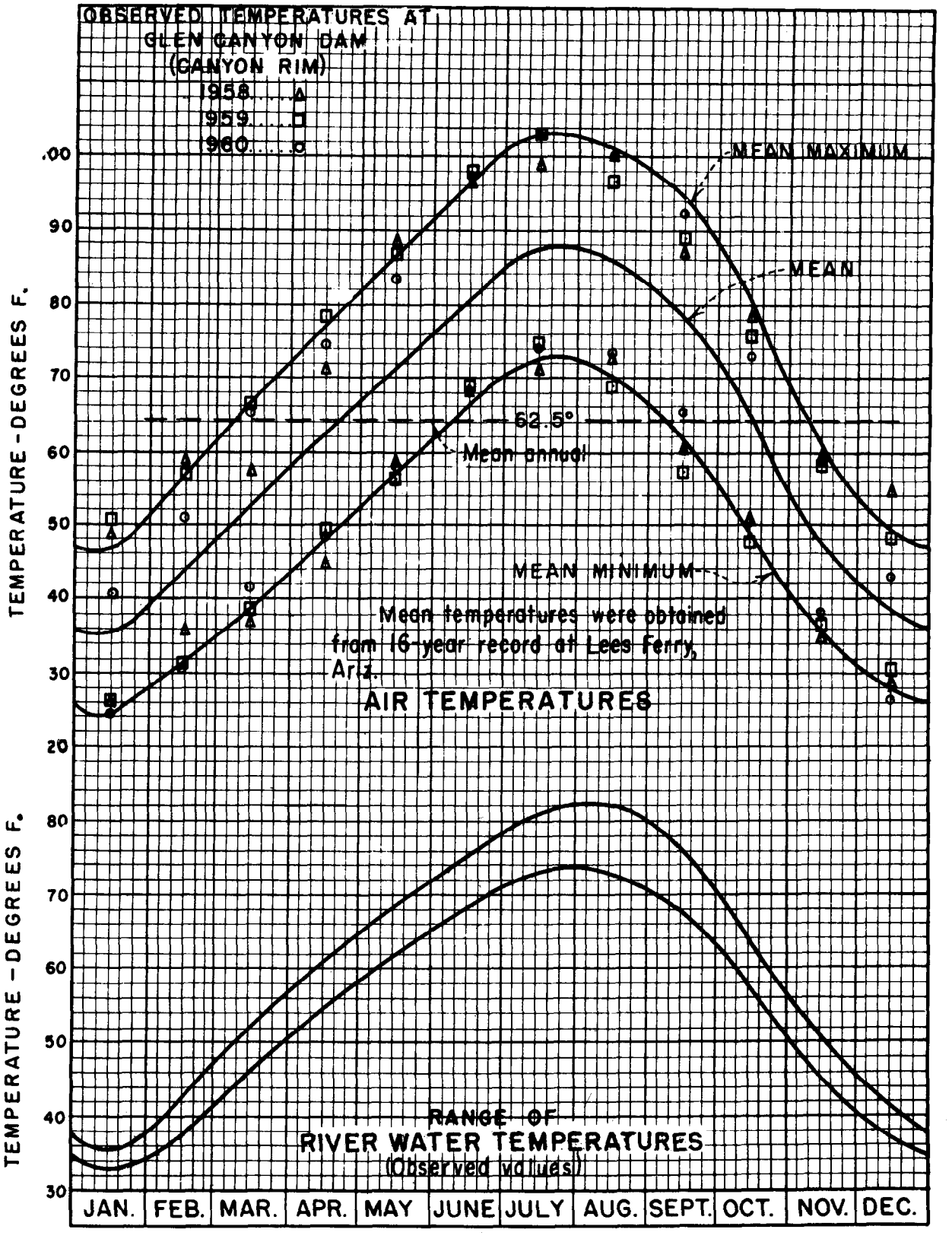


FIGURE 10.—*Glen Canyon Dam—air and water temperatures.*

TABLE I.—*Thermal properties of concrete for various dams*

Dam	Density (saturated) lb /cu ft	Conductivity K B.t.u./ft /hr /°F.			Specific heat C B.t.u./lb /°F.			Diffusivity h^2 ft ^{2}/hr		
		50°	70°	90°	50°	70°	90°	50°	70°	90°
Norris	160.6	2.120	2.105	2.087	0.234	0.239	0.247	0.056	0.055	0.053
Glen Canyon	148.4	2.02	2.01	2.01	.211	.216	.222	.065	.063	.061
Seminoe	155.3	1.994	1.972	1.951	.204	.213	.222	.063	.060	.057
Wheeler	145.5	1.815	1.800	1.785	.223	.229	.236	.056	.054	.052
Flaming Gorge	150.4	1.78	1.77	1.76	.221	.226	.232	.054	.052	.050
Kortes mixes:										
1 bbl cement/cu yd and 0.0-percent air	157.6	1.736	1.724	1.711	.210	.215	.221	.052	.051	.049
0.85 bbl cement cu yd and 0.0-percent air	158.1	1.715	1.710	1.705	.209	.215	.220	.052	.050	.049
Hungry Horse	150.1	1.72	1.72	1.71	.217	.223	.229	.053	.051	.050
Hoover	156.0	1.699	1.688	1.677	.212	.216	.221	.051	.050	.049
Gibson	155.2	1.676	1.667	1.657	.218	.222	.229	.050	.048	.047
Canyon Ferry	151.3	1.63	1.62	1.61	.214	.218	.222	.050	.049	.048
Altus	149.7	1.578	1.579	1.580	.225	.229	.234	.047	.046	.045
Monticello	153.1	1.57	1.56	1.55	.225	.230	.235	.046	.044	.043
Yellowtail	152.8	1.57	1.56	1.55	.219	.223	.227	.047	.046	.045
Angostura mixes:										
0.9 bbl cement/cu yd and 3.0-percent air	151.2	1.491	1.484	1.478	.221	.228	.234	.045	.043	.042
1.04 bbl cement/cu yd and 0.0-percent air	152.6	1.571	1.554	1.537	.227	.234	.240	.045	.044	.042
Hiwassee	155.7	1.505	1.491	1.478	.218	.225	.233	.044	.042	.041
Parker	155.1	1.409	1.402	1.395	.213	.216	.221	.043	.042	.041
Owyhée	152.1	1.376	1.373	1.369	.208	.214	.222	.044	.042	.041
O'Shaughnessy	152.8	1.316	1.338	1.354	.217	.218	.223	.040	.040	.040
Friant mixes:										
Portland cement	153.6	1.312	1.312	1.312	.214	.214	.217	.040	.040	.039
20-percent pumicite	153.8	1.229	1.232	1.234	.216	.221	.227	.037	.036	.035
Shasta	157.0	1.299	1.309	1.319	.222	.229	.235	.037	.037	.036
Bartlett	156.3	1.293	1.291	1.289	.216	.222	.230	.038	.037	.036
Morris	156.9	1.290	1.291	1.293	.214	.216	.222	.039	.038	.037
Chickamauga	156.5	1.287	1.277	1.266	.225	.229	.233	.037	.036	.035
Grand Coulee	158.1	1.075	1.077	1.079	.219	.222	.227	.031	.031	.030
Ariel	146.2	0.842	0.884	0.915	.228	.235	.244	.025	.026	.026
Bull Run	159.1	0.835	0.847	0.860	.215	.225	.234	.024	.024	.023
Thermal Properties of Coarse Aggregate										
Quartzite	151.7	2.052	2.040	2.028	.209	.217	.226	.065	.062	.059
Dolomite	156.2	1.948	1.925	1.903	.225	.231	.238	.055	.053	.051
Limestone	152.8	1.871	1.842	1.815	.221	.224	.230	.055	.054	.052
Granite	150.9	1.515	1.511	1.588	.220	.220	.224	.046	.045	.045
Basalt	157.5	1.213	1.212	1.211	.226	.226	.230	.034	.034	.033
Rhyolite	146.3	1.197	1.203	1.207	.220	.226	.232	.037	.036	.036

TABLE II.—*Amplitudes of air temperatures*

Period	Extreme weather conditions		Usual weather conditions	
	Above mean	Below mean	Above mean	Below mean
Annual	(1)	(2)	(1)	(2)
15-day	(4)	(5)	(6)	(7)
Daily	(3)	(3)	(3)	(3)

[1] The difference between the *highest mean monthly* and the *mean annual*.
[2] The difference between the *lowest mean monthly* and the *mean annual*.
[3] One-half the minimum difference between any *mean monthly maximum* and the corresponding *mean monthly minimum*.
[4] The difference between [1+3] and [the *highest maximum recorded* minus the *mean annual*].
[5] The difference between [2+3] and [the *lowest minimum recorded* difference from *mean annual*].
[6] The difference between [1+3] and [the difference between the *mean annual* and the average of the *highest maximum recorded* and the *highest mean monthly maximum*].
[7] The difference between [2+3] and [the difference between the *mean annual* and the average of the *minimum recorded* and the *lowest mean monthly minimum*].

ternal temperature is obtained. The products of these ratios and their respective amplitudes are algebraically added to and subtracted from the mean annual air temperature to obtain mean concrete temperatures for the condition of air on both faces. Mean concrete temperatures which would result from a fictitious condition of water on both faces are then obtained, and the two conditions are simply averaged together to obtain the condition of air on one face and water on the other.

Reservoir Water Temperatures.—The stabilized temperature distributions in a concrete dam are dependent, to a large extent, upon the reservoir water temperatures. These reservoir water temperatures vary with depth, and for all practical purposes can be considered to have only an annual cycle. There will also be a time lag between the air and water temperatures, the greatest lag occurring in the lower part of the reservoir. Normally this time lag need not be taken into consideration. There are circumstances, however, as with extremely thin arch dams, when the effects of this time lag should be investigated. The reservoir water temperatures normally used in determining the range of mean concrete temperatures for a proposed dam are those temperatures which will occur after the reservoir is in operation.

The best estimate of the expected reservoir water temperatures would be one based on water temperatures recorded at nearby lakes and reservoirs of similar depth and with similar inflow and outflow conditions. The Bureau of Reclamation has taken reservoir water temperatures over a period of several years in a number of reservoirs. From these data, maximum ranges of temperature for the operating conditions encountered during the data period were obtained as shown in Figures 12 and 13. Reservoir water temperatures were also determined for periods of about 1½ years at Mormon Flat, Horse Mesa, Roosevelt, and Stewart Mountain Dams. These data are shown in Figure 14. Figures 15 through 19 show temperature variations with depth throughout the year at various dams in the western United States. Figures 20 and 21 show similar data obtained by the Tennessee Valley Authority for 5- and 9-year periods at Fontana and Hiwassee Dams. Figures 22 and 23 not only provide data at Elephant Butte Dam and at Grand Lake (Colorado), but also illustrate two methods of plotting data on reservoir temperatures.

When no data are available at nearby reservoirs, the next best estimate of the reservoir temperatures would be obtained by the principle of heat continuity. This method takes into consideration the amount and temperature of the water entering and leaving the reservoir, and the heat transfer across the reservoir surface. These heat budget computations, though accurate in themselves, are based on estimates of evaporation, conduction, absorption and reflection of solar radiation, and reradiation, which are in turn based on cloud cover, air temperatures, wind, and relative humidity. Variable as these factors are, any forecast of temperature conditions in a reservoir based on the principle of heat continuity must be considered only as an estimate.

In the absence of data from nearby reservoirs and data on the flow and temperature of the river, a rough estimate of the reservoir temperatures can be prepared based on the monthly mean air temperatures at the site, the operation and capacity of the reservoir, and the general characteristics of the river flow. The amplitude of the water temperature variation at the surface will be nearly equal to that of the mean monthly air temperature, except that it will not go below 32° F. Below the surface, the operation of the reservoir contributes to a wide range of temperatures experienced at the different elevations on the upstream face of the dam. Flood control reservoirs vary between wide limits depending upon the operating criteria of

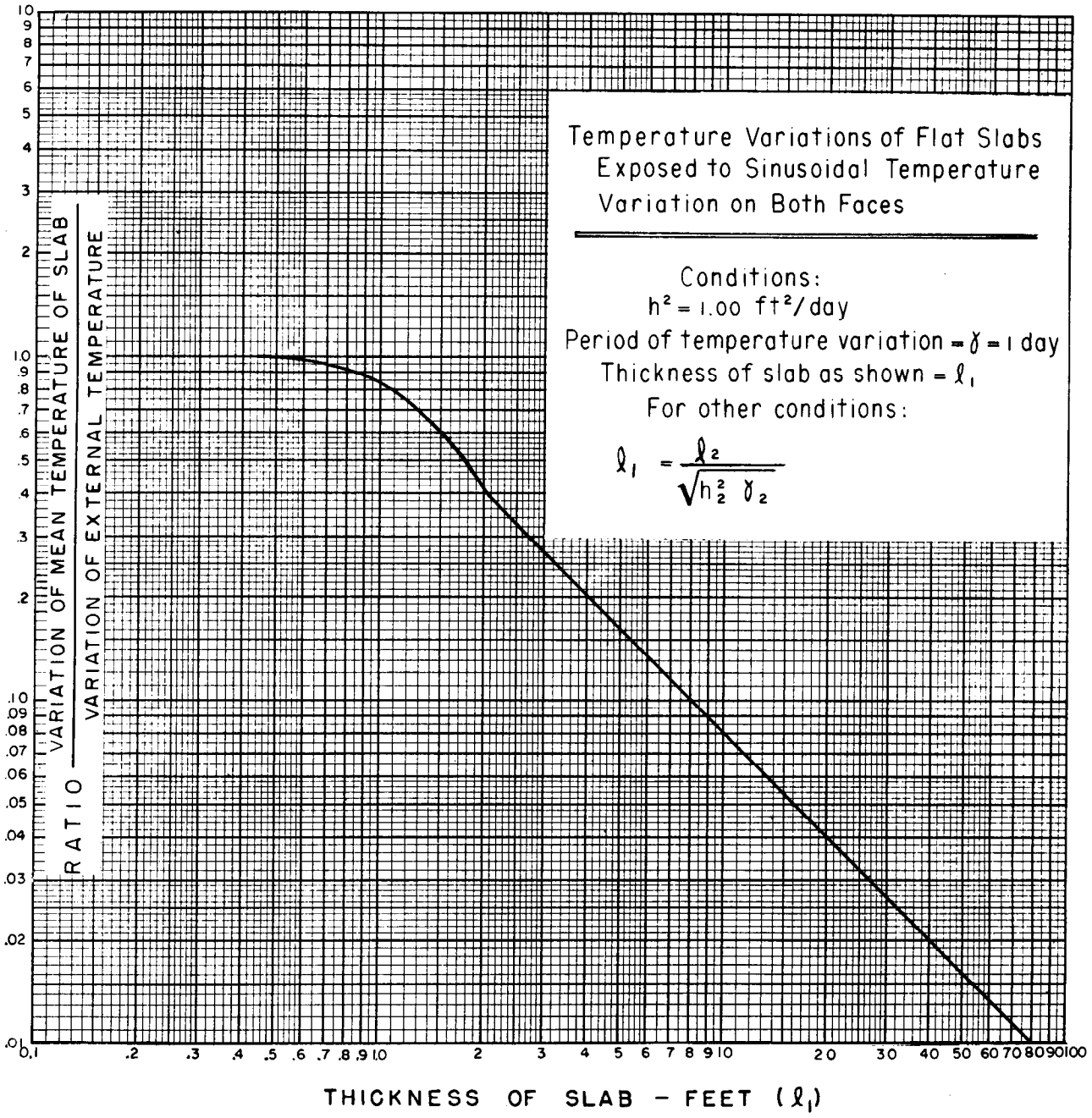


FIGURE 11.—*Temperature variations of flat slabs exposed to sinusoidal temperature variation on both faces.*

the reservoir. The water surface of a storage reservoir will fluctuate through a wide range as it is filled and then drawn upon for irrigation water; the reservoir water surface for a power dam will normally remain at a relatively uniform level.

The flow of the river determines, to a large extent, the reservoir water temperatures. Snowmelt and proximity to the source may mean that the temperature of the incoming water will be very low in the winter and early spring months. The distance from the source will affect the stream's temperature considerably by the time it enters the reservoir. A large part of the water for Lake Mead starts as snowmelt, but by the time it reaches the reservoir, hundreds of miles downstream, it has warmed up considerably. Colorado River

LOCATION	RANGE OF MONTHLY MEAN AIR TEMP.	MEAN ANNUAL AIR TEMP.	RANGE OF MONTHLY MEAN RIVER WATER TEMP.	MEAN ANNUAL RIVER DISCHARGE RESERVOIR CAPACITY	PERIOD OF RECORD	REMARKS
Glen Canyon Dam	36°-88°	62°	34°-80°	1:2.5	1964	Res. storage started in March 1963
Hoover Dam	51°-96°	73°	40°-82°	1:2.4	1938-1948	Reservoir filling thru 1937
Hungry Horse Dam	22°-65°	43°	34°-55°	1:1.6	1954-1959	Reservoir filling 1951-1953
Flaming Gorge Dam	24°-72°	48°	32°-68°	1:2.6	1964	Res. storage started in Nov. 1962
Shasta Dam	45°-82°	62°	44°-68°	1.2:1	1946-1952	Reservoir filling thru 1945
Grand Coulee Dam	24°-74°	51°	34°-65°	7.9:1	1942-1952	Reservoir filled in June 1942

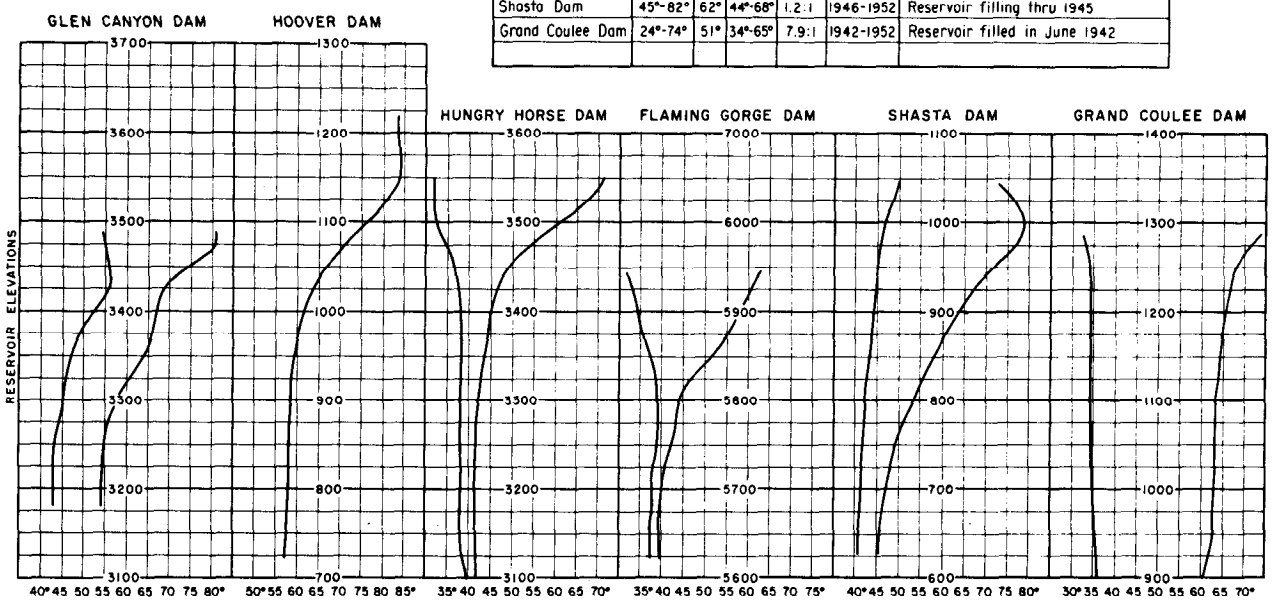
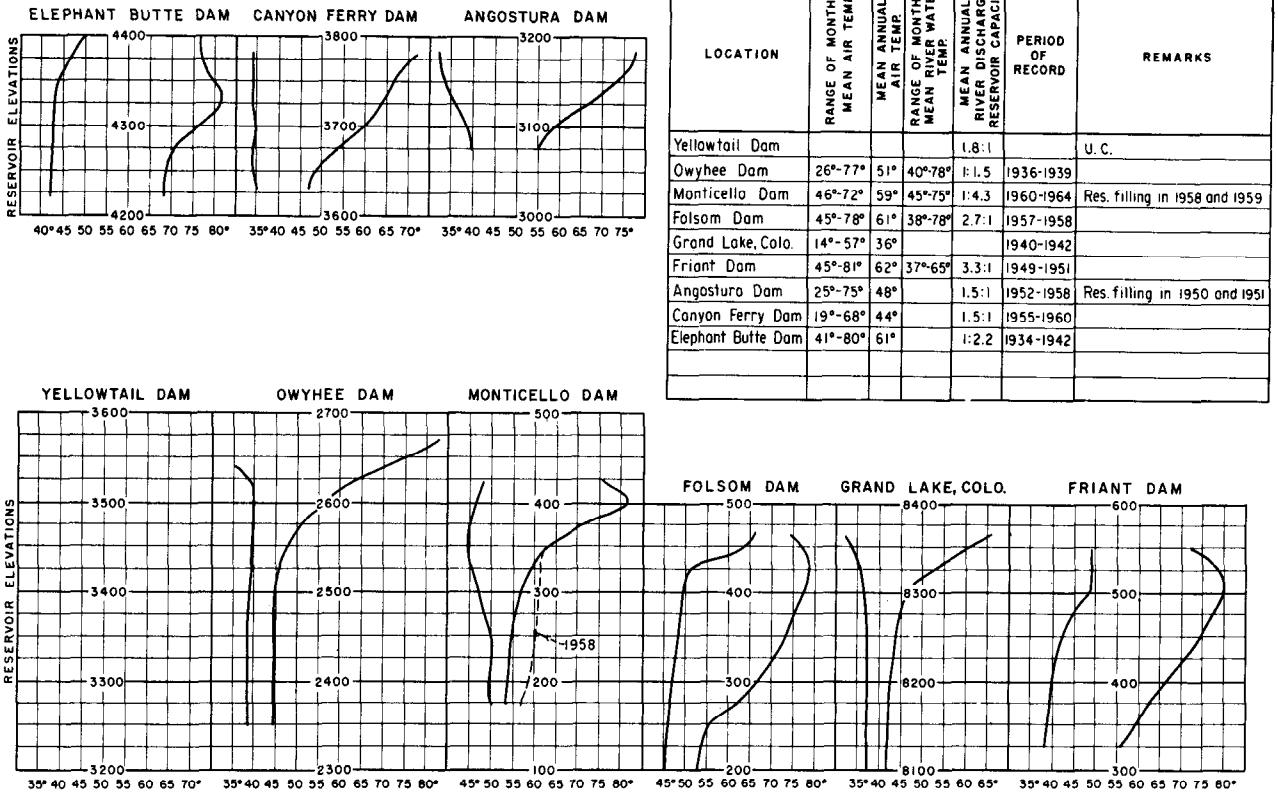


FIGURE 12.—*Range of actual reservoir temperatures under operating conditions (sheet 1 of 2).*

water temperatures vary during the year from 32° to about 60° F. in the river below Grand Lake, Colorado, and from 40° to 82° F. at Hoover Dam. Water temperatures on the Sacramento River in California were obtained at eight locations. Variations of these river water temperatures are shown in Figure 24.

In regard to water temperatures downstream from a reservoir, it has been found that the discharged water changes very slowly in temperature. Convection currents in the flowing water create a rather uniform temperature which must drop to 32° F. before icing conditions begin. Below Hebgen Dam, near Yellowstone, no appreciable quantity of ice begins within 10 miles of the dam, even in extreme temperatures. During the wintertime below Bersimis No. 1 Reservoir on the Bersimis River in Canada, the tailrace temperature was 37° F. This water dropped in temperature to 33° F. by the time it reached another point of measurement 23 miles downstream. "A Mathematical Model of Stream Temperature" by David W. Duttweiler (Thesis, The John Hopkins University, 1963) gives methods for estimating stream temperatures, both in natural flow and downstream from reservoirs. "Prediction of Temperature in Rivers and Reservoirs" by Jerome M. Raphael (*Journal* of ASCE Power Division, Volume 88, July 1962) also gives a mathematical method for predicting water temperatures in streams and shallow reservoirs with no thermoclines.

The ratio of the annual runoff of the river to the reservoir capacity affects the reservoir water temperature, especially in the lower parts of the reservoir. Some reservoirs, such as Franklin D. Roosevelt Lake, behind Grand Coulee Dam, have a large ratio of runoff to capacity. At Roosevelt Lake, the annual flow of the river would theoretically fill the reservoir eight times a year. This means, literally, that the reservoir would be filled and emptied eight times during a year, and as



LOCATION	RANGE OF MONTHLY MEAN AIR TEMP.	MEAN ANNUAL AIR TEMP.	RANGE OF MONTHLY MEAN RIVER WATER TEMP.	MEAN ANNUAL RIVER DISCHARGE RESERVOIR CAPACITY	PERIOD OF RECORD	REMARKS
Yellowtail Dam				1.8:1		U.C.
Owyhee Dam	26°-77°	51°	40°-78°	1:1.5	1936-1939	
Monticello Dam	46°-72°	59°	45°-75°	1:4.3	1960-1964	Res. filling in 1958 and 1959
Folsom Dam	45°-78°	61°	38°-78°	2.7:1	1957-1958	
Grand Lake, Colo.	14°-57°	36°			1940-1942	
Friant Dam	45°-81°	62°	37°-65°	3.3:1	1949-1951	
Angostura Dam	25°-75°	48°		1.5:1	1952-1958	Res. filling in 1950 and 1951
Canyon Ferry Dam	19°-68°	44°		1.5:1	1955-1960	
Elephant Butte Dam	41°-80°	61°		1:2.2	1934-1942	

FIGURE 13.—*Range of actual reservoir temperatures under operating conditions (sheet 2 of 2).*

such, the reservoir temperatures would, for all depths, approach those of the incoming river water temperatures. Other reservoirs may take several years to fill with the full flow of the river. Lake Mead, for example, would require about 2½ years of the mean river discharge to fill the reservoir. The water moves into the reservoir and stays for a relatively long period of time, during which time it is affected by the climatic conditions of the site.

Solar Radiation Effect.—The mean concrete temperatures obtained from air and water temperatures require adjustments due to the effect of solar radiation on the surface of the dam. The downstream face, and the upstream face when not covered by reservoir water, receive an appreciable amount of radiant heat from the sun and this has the effect of warming the concrete surface above the surrounding air temperature. The amount of this temperature rise above the surrounding air temperatures has been recorded at the faces of several dams in the western portion of the United States. These data were then correlated with theoretical studies which take into consideration varying slopes, orientation of the exposed faces, and latitudes.

The results of these studies are presented in an unpublished Bureau of Reclamation memorandum, "The Average Temperature Rise of the Surface of a Concrete Dam Due to Solar Radiation," by W. A. Trimble. Figures 25 through 28 summarize the results obtained by the Trimble memorandum and give values of the temperature increase for various latitudes, slopes, and orientations. It should be noted that the curves give a value for the mean annual increase in temperature and not for any particular hour, day, or month. Examples of how this solar radiation varies throughout the year are given in Figure 29.

If a straight gravity dam is being considered, the orientaton will be the same for all points on the dam, and only one value for each of the upstream and downstream faces will be required. For an arch dam, values at the quarter points

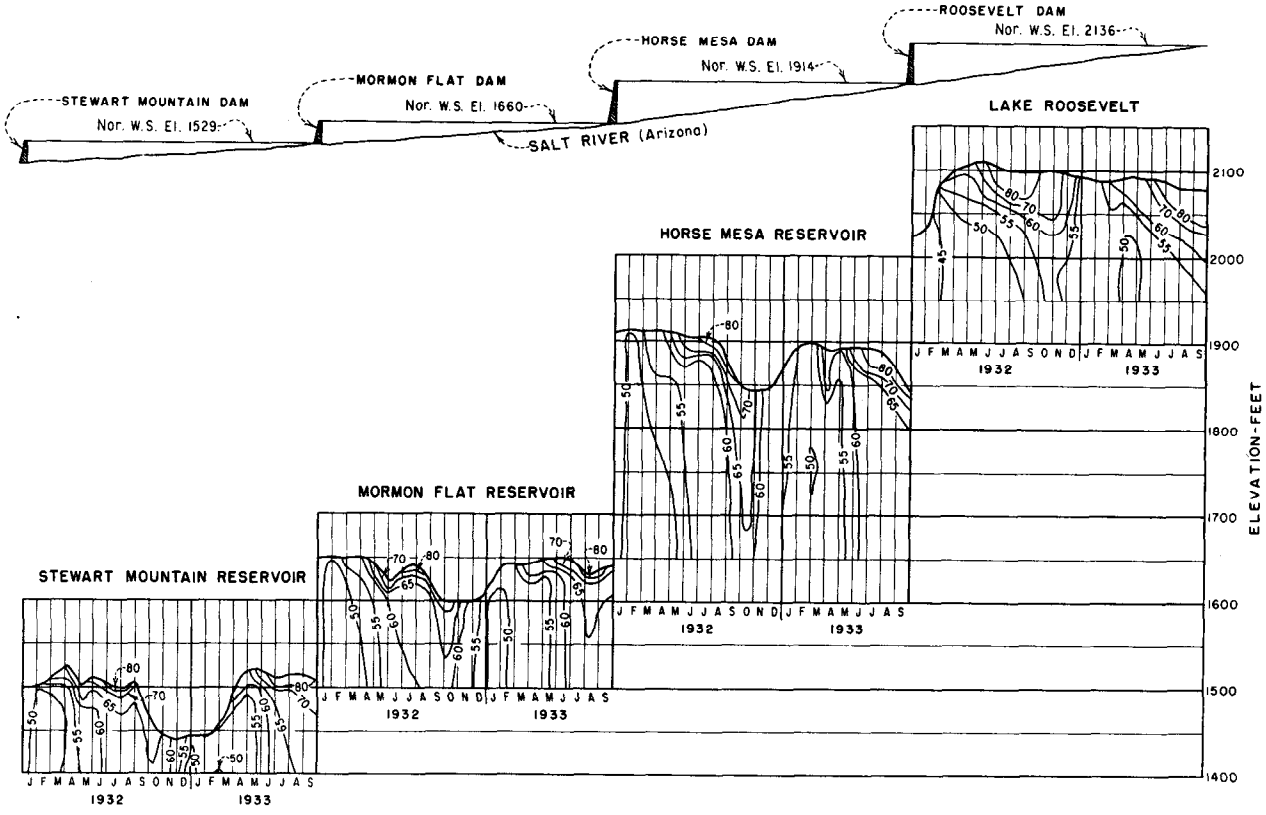
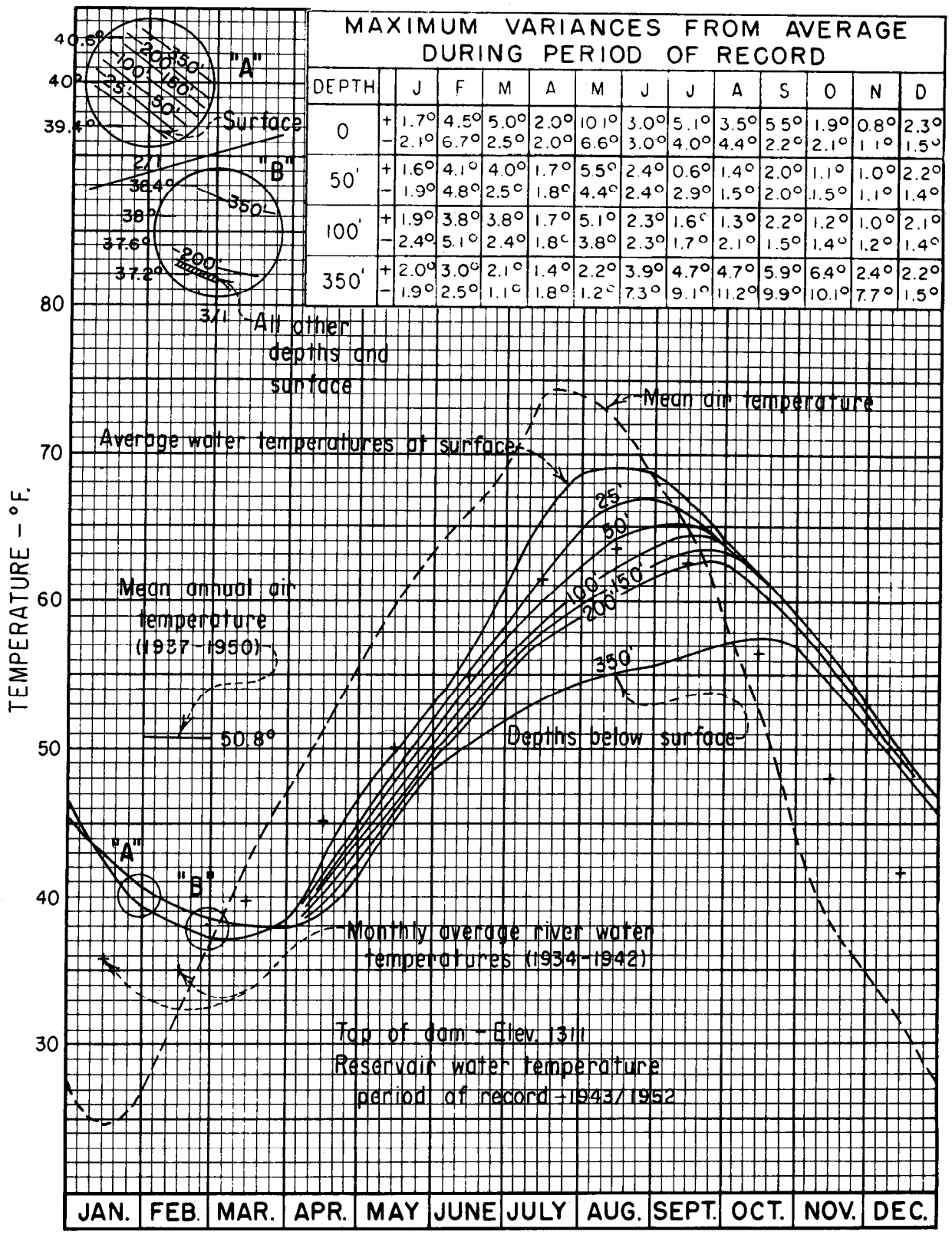


FIGURE 14.—*Range of actual reservoir temperatures under operating conditions—Salt River project dams.*

should be obtained as the sun's rays will strike different parts of the dam at varying angles. The temperature rises shown on the graph should be corrected by a terrain factor which is expressed as the ratio of actual exposure to the sun's rays to the theoretical exposure. This is required because the theoretical computations assumed a horizontal plane at the base of the structure and the effect of the surrounding terrain is to block out certain hours of sunshine. Although this terrain factor will actually vary for different points on the dam, an east-west profile of the area terrain, which passes through the crown cantilever of the dam, will give a single factor which can be used for all points and remain within the limits of accuracy of the method itself. Figure 9 shows the computation of the solar radiation values and how they are applied to the previously obtained mean concrete temperatures.

Closure Temperature

In general, with normal reservoir load and minimum or near-minimum concrete temperatures, tensile stresses in arch dams tend to occur in the lower part of the dam, at the intrados in the central part, and at the extrados near the abutments. With the same loading conditions, vertical tensile stresses tend to occur on the upstream face near the base of the dam and on the downstream face near the top of the dam. With minimum reservoir load and maximum or near-maximum concrete temperatures, tensile stresses tend to occur along the intrados near the abutments, and vertical tensile stresses tend to occur on the downstream face near the base of the dam. The tensile stresses on the upstream face occurring with reservoir load and near-minimum concrete temperatures can be reduced by additional cooling of the concrete prior to grouting the contraction joints. Such additional cooling, however, will increase the tensile stresses for the minimum reservoir—maximum temperature condition. The actual closure temperature for an arch dam is, therefore, a compromise arrived at to minimize the tensile stresses, with additional weight being given to reduce the tensile stresses at the upstream face.



MAXIMUM VARIANCES FROM AVERAGE DURING PERIOD OF RECORD

DEPTH		J	F	M	A	M	J	J	A	S	O	N	D
0	+	1.7°	4.5°	5.0°	2.0°	10.1°	3.0°	5.1°	3.5°	5.5°	1.9°	0.8°	2.3°
	−	2.1°	6.7°	2.5°	2.0°	6.6°	3.0°	4.0°	4.4°	2.2°	2.1°	1.1°	1.5°
50'	+	1.6°	4.1°	4.0°	1.7°	5.5°	2.4°	0.6°	1.4°	2.0°	1.1°	1.0°	2.2°
	−	1.9°	4.8°	2.5°	1.8°	4.4°	2.4°	2.9°	1.5°	2.0°	1.5°	1.1°	1.4°
100'	+	1.9°	3.8°	3.8°	1.7°	5.1°	2.3°	1.6°	1.3°	2.2°	1.2°	1.0°	2.1°
	−	2.4°	5.1°	2.4°	1.8°	3.8°	2.3°	1.7°	2.1°	1.5°	1.4°	1.2°	1.4°
350'	+	2.0°	3.0°	2.1°	1.4°	2.2°	3.9°	4.7°	4.7°	5.9°	6.4°	2.4°	2.2°
	−	1.9°	2.5°	1.1°	1.8°	1.2°	7.3°	9.1°	11.2°	9.9°	10.1°	7.7°	1.5°

FIGURE 15.—*Reservoir temperatures—Grand Coulee Dam.*

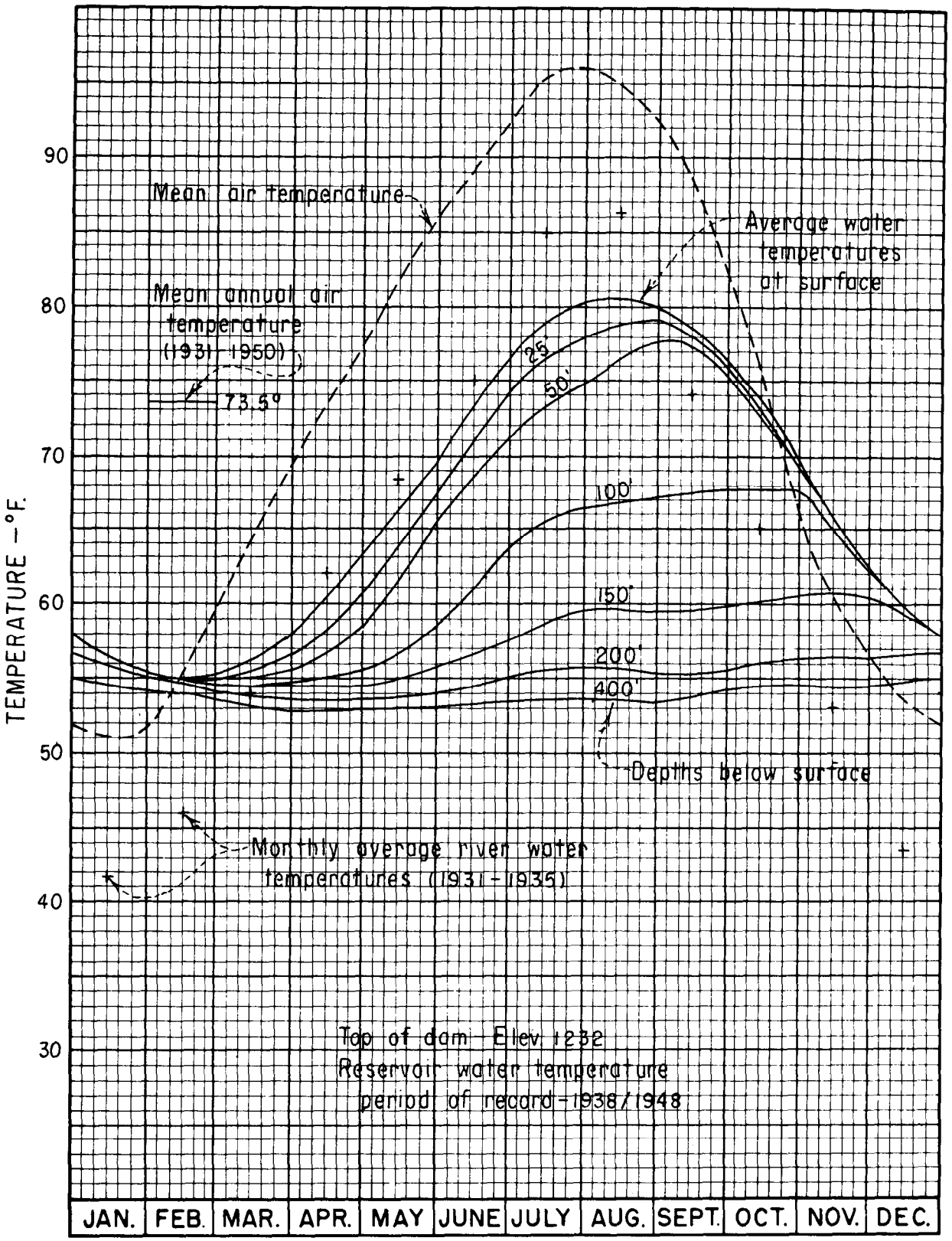


FIGURE 16.—*Reservoir temperatures—Hoover Dam.*

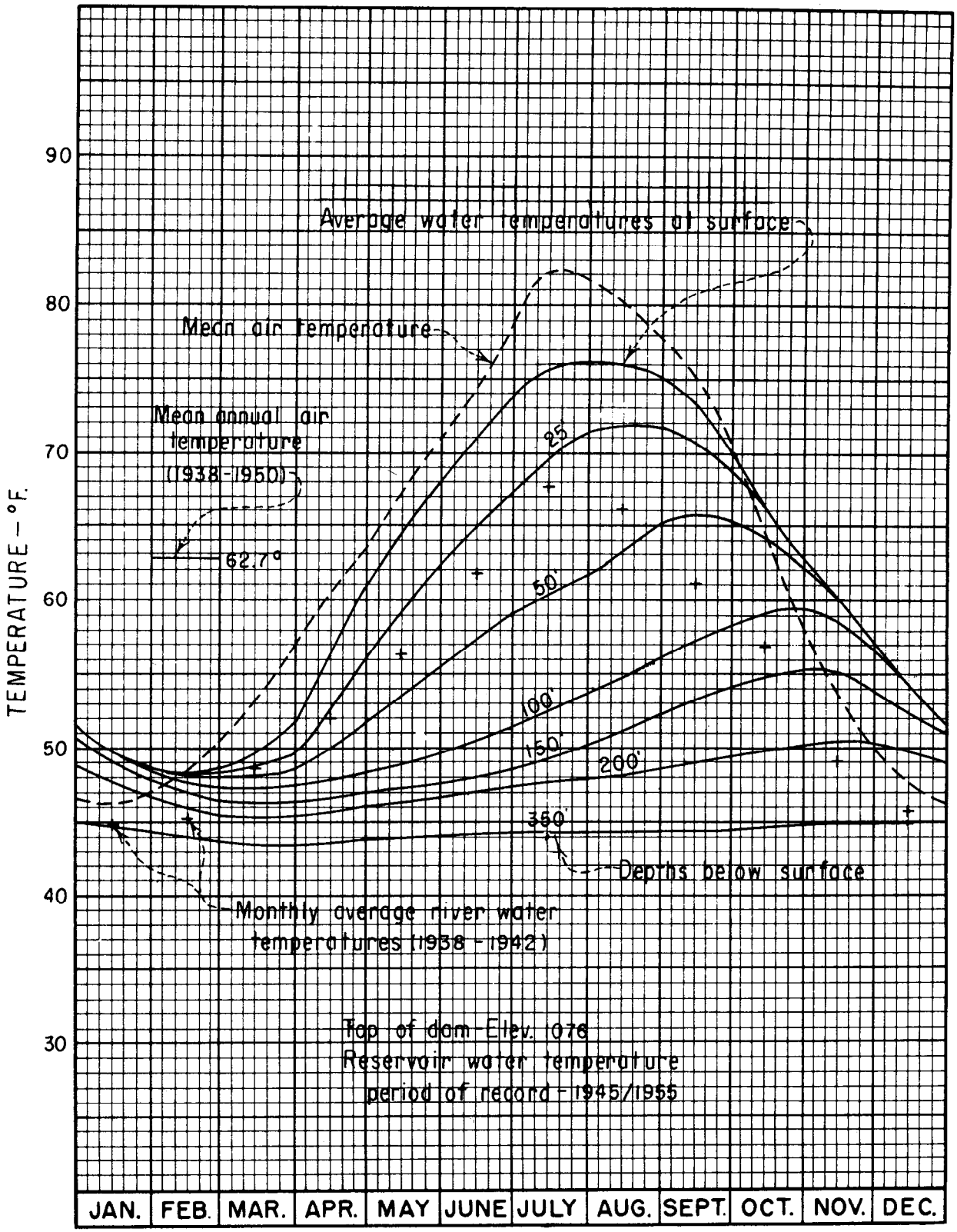


FIGURE 17.—*Reservoir temperatures—Shasta Dam.*

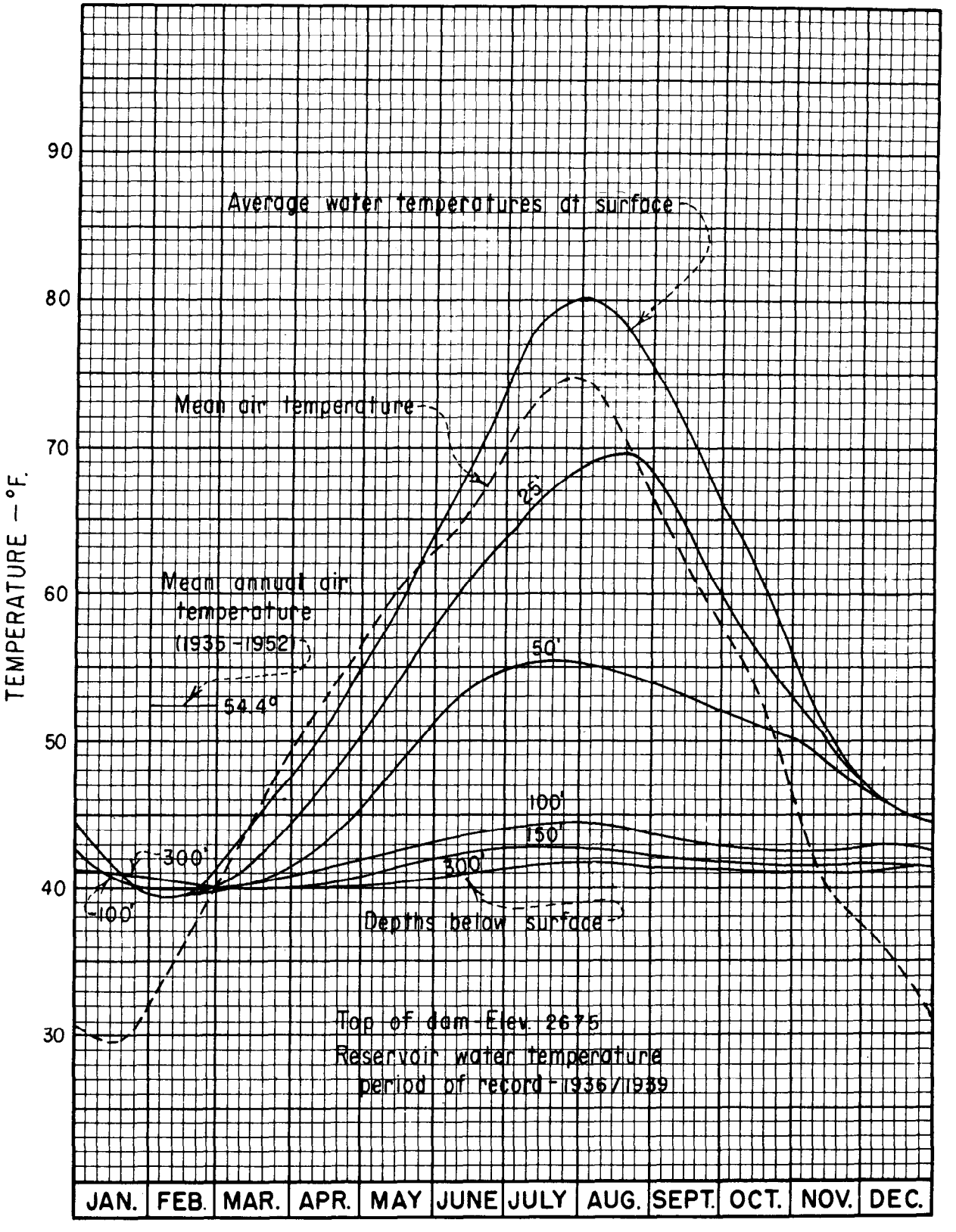


FIGURE 18.—*Reservoir temperatures—Owyhee Dam.*

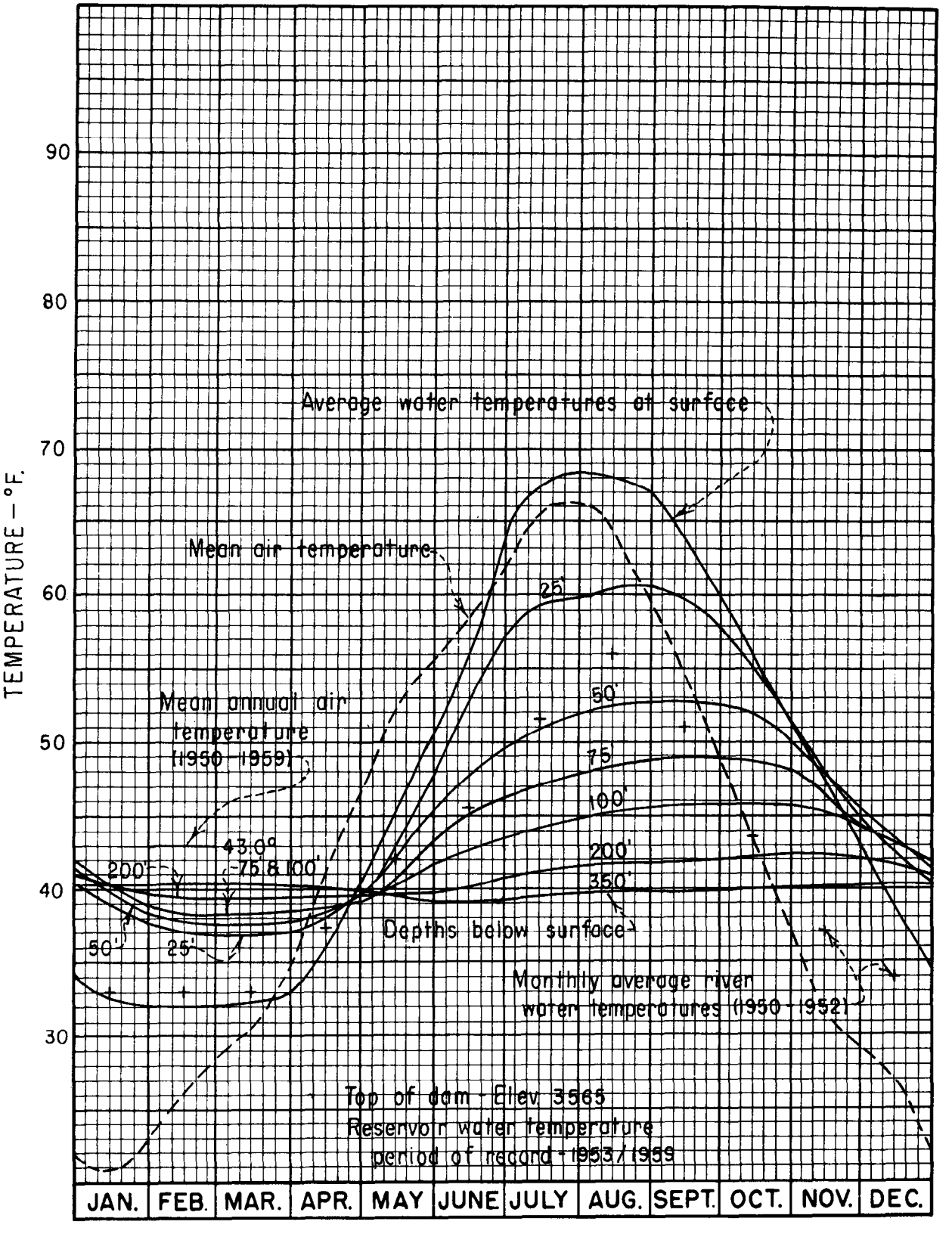


FIGURE 19.—*Reservoir temperatures—Hungry Horse Dam.*

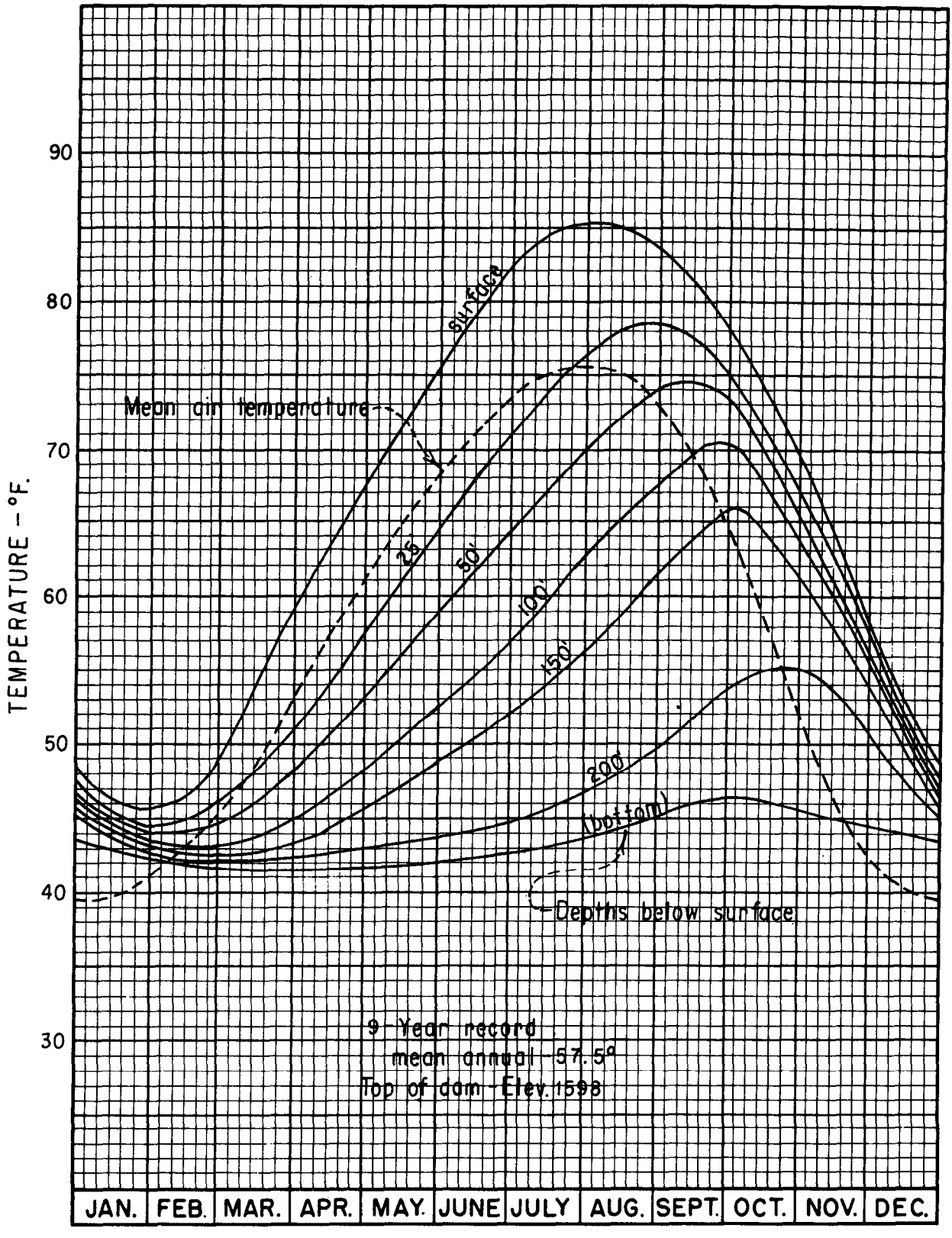


FIGURE 20.—*Reservoir temperatures—Hiwassee Dam.*

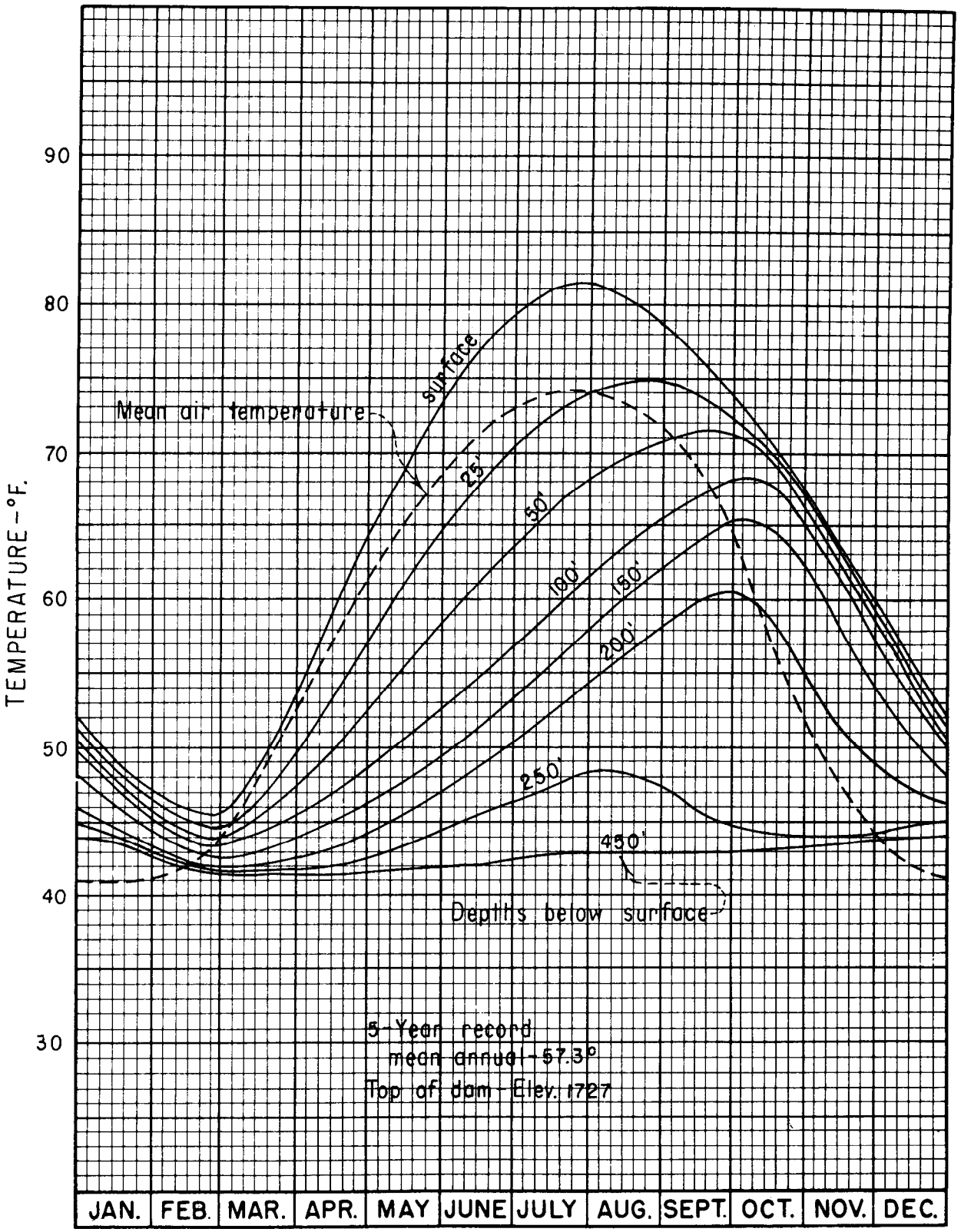
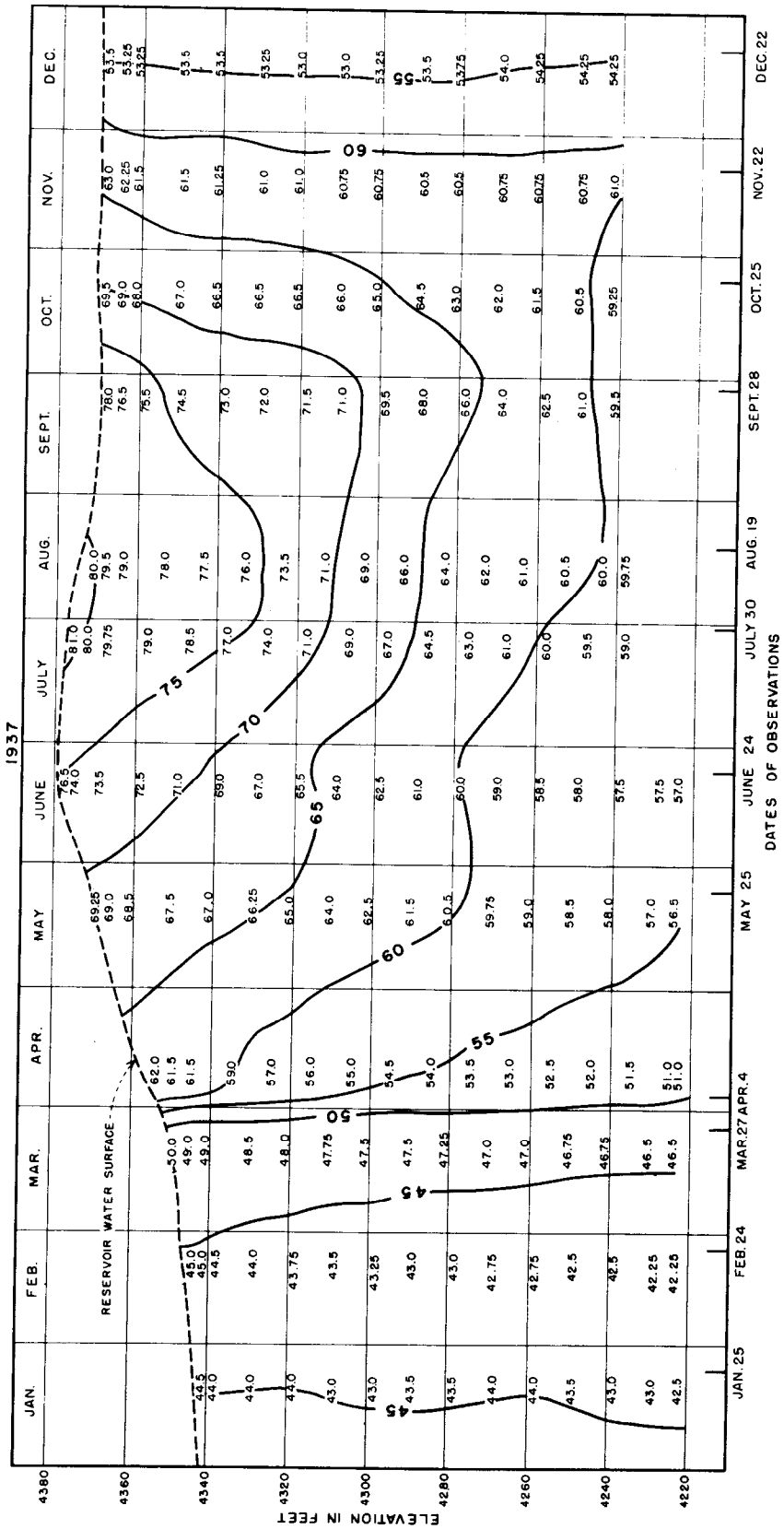


FIGURE 21.—*Reservoir temperatures—Fontana Dam.*



NOTES

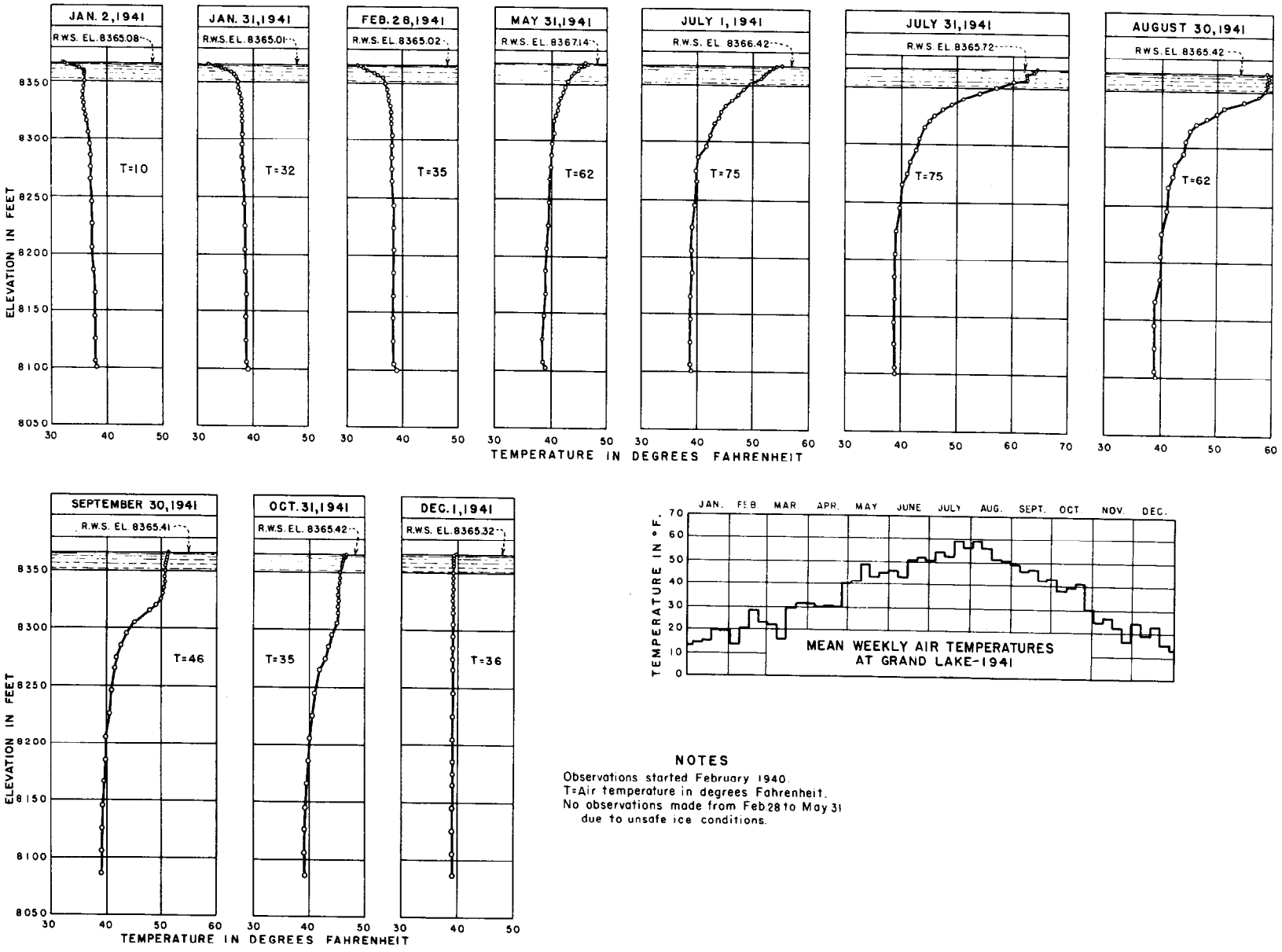
Observations for January and February made at dam; March, April, and May made 100 feet above dam; June, 200 feet above dam; and all others one-half mile above dam.

All temperatures are Fahrenheit.

Elevation of top of dam 4414.

Bottom of reservoir not shown.

FIGURE 22.—*Reservoir temperatures—Elephant Butte Dam.*



NOTES

Observations started February 1940.
T=Air temperature in degrees Fahrenheit.
No observations made from Feb. 28 to May 31 due to unsafe ice conditions.

FIGURE 23.—*Reservoir temperatures—Grand Lake, Colorado.*

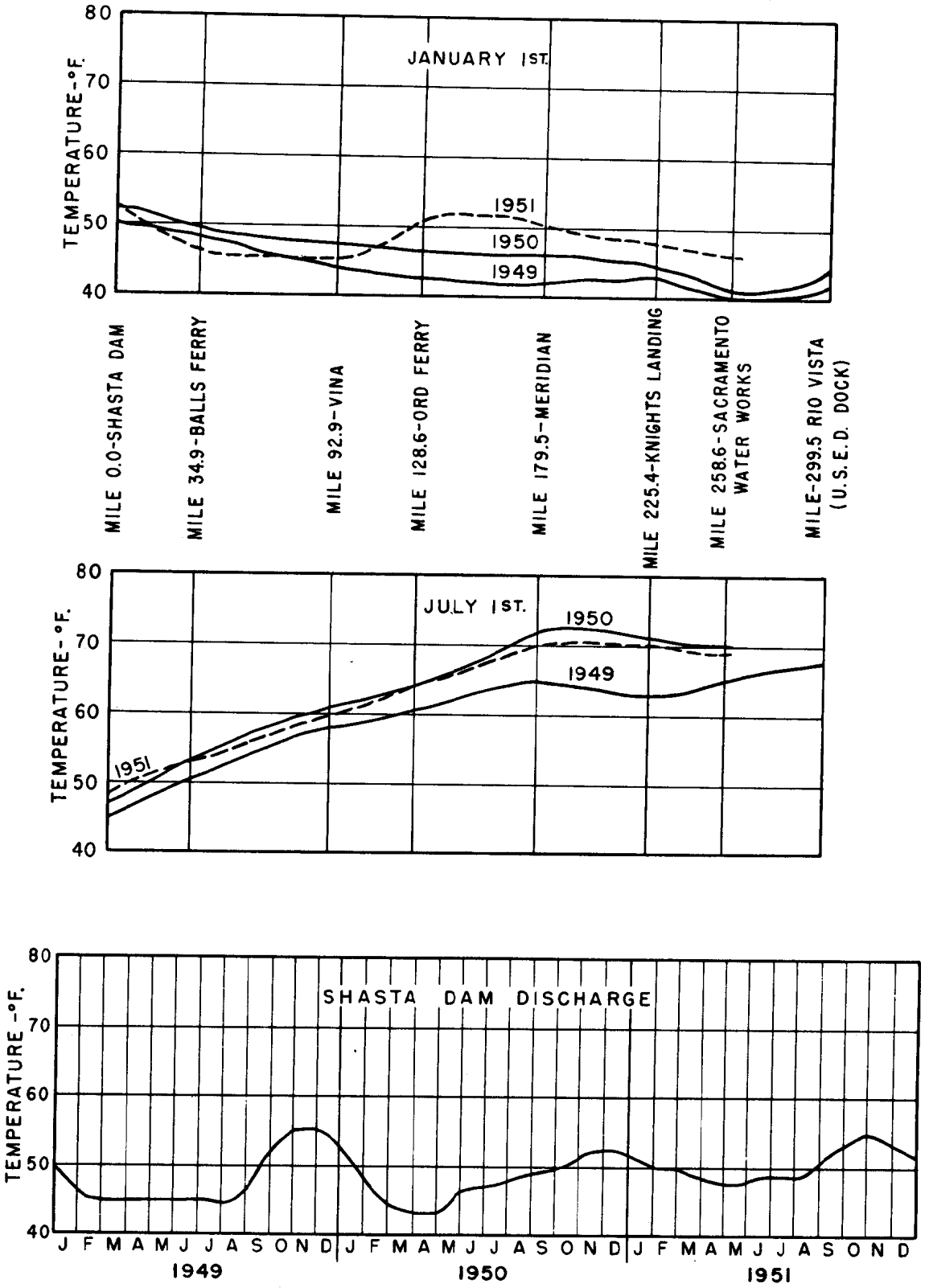
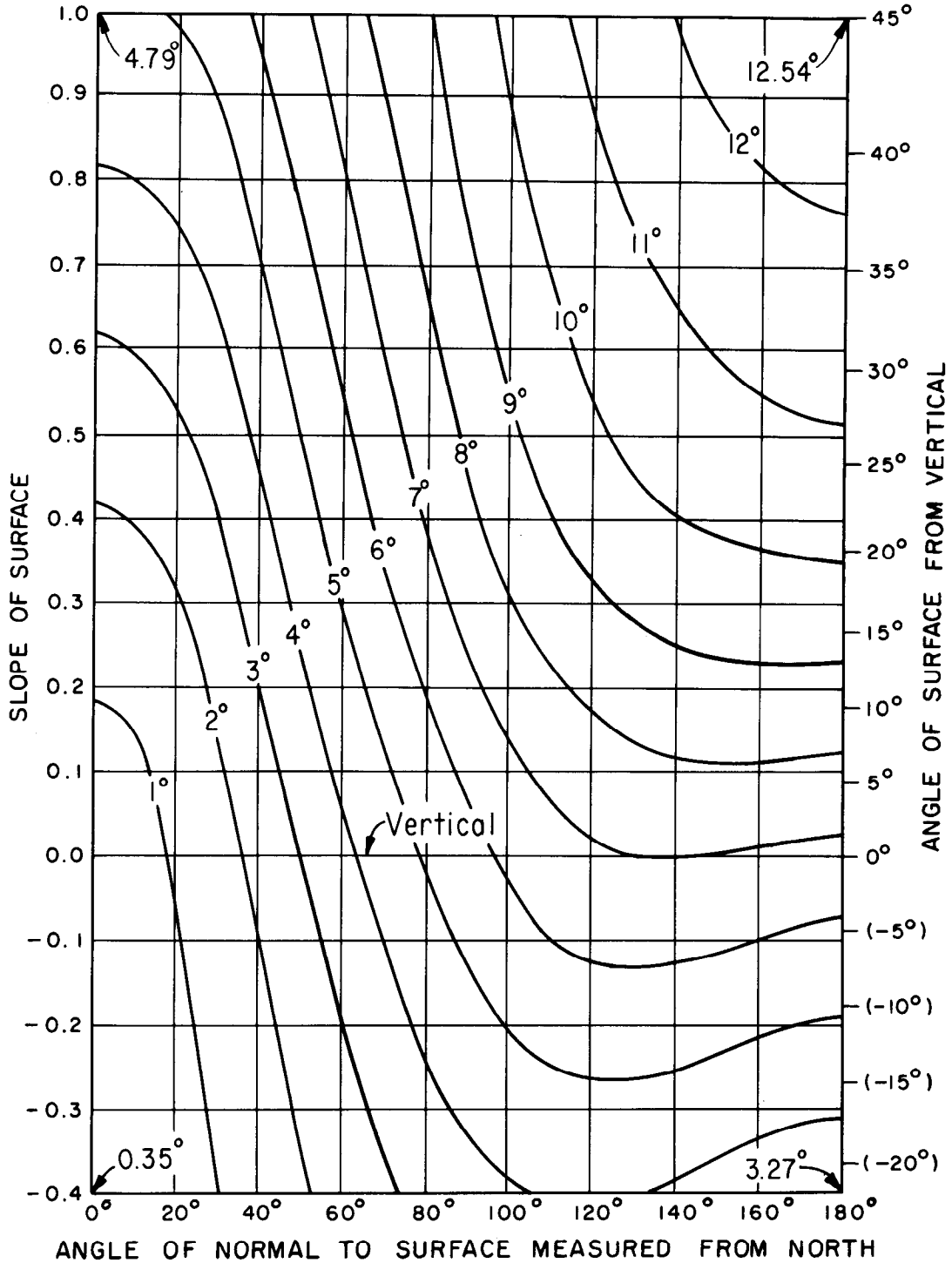
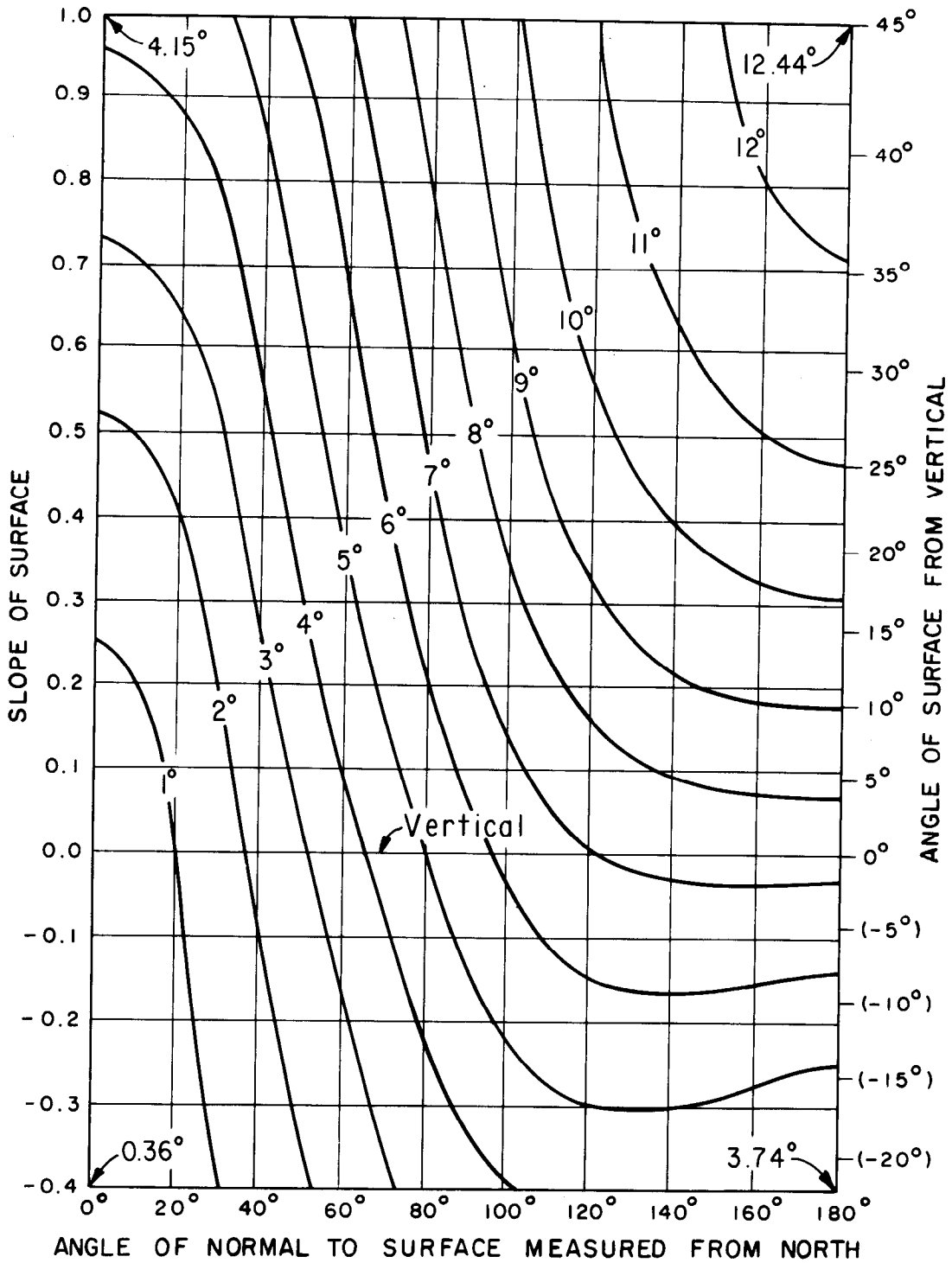


FIGURE 24.—*River water temperatures—Sacramento River.*



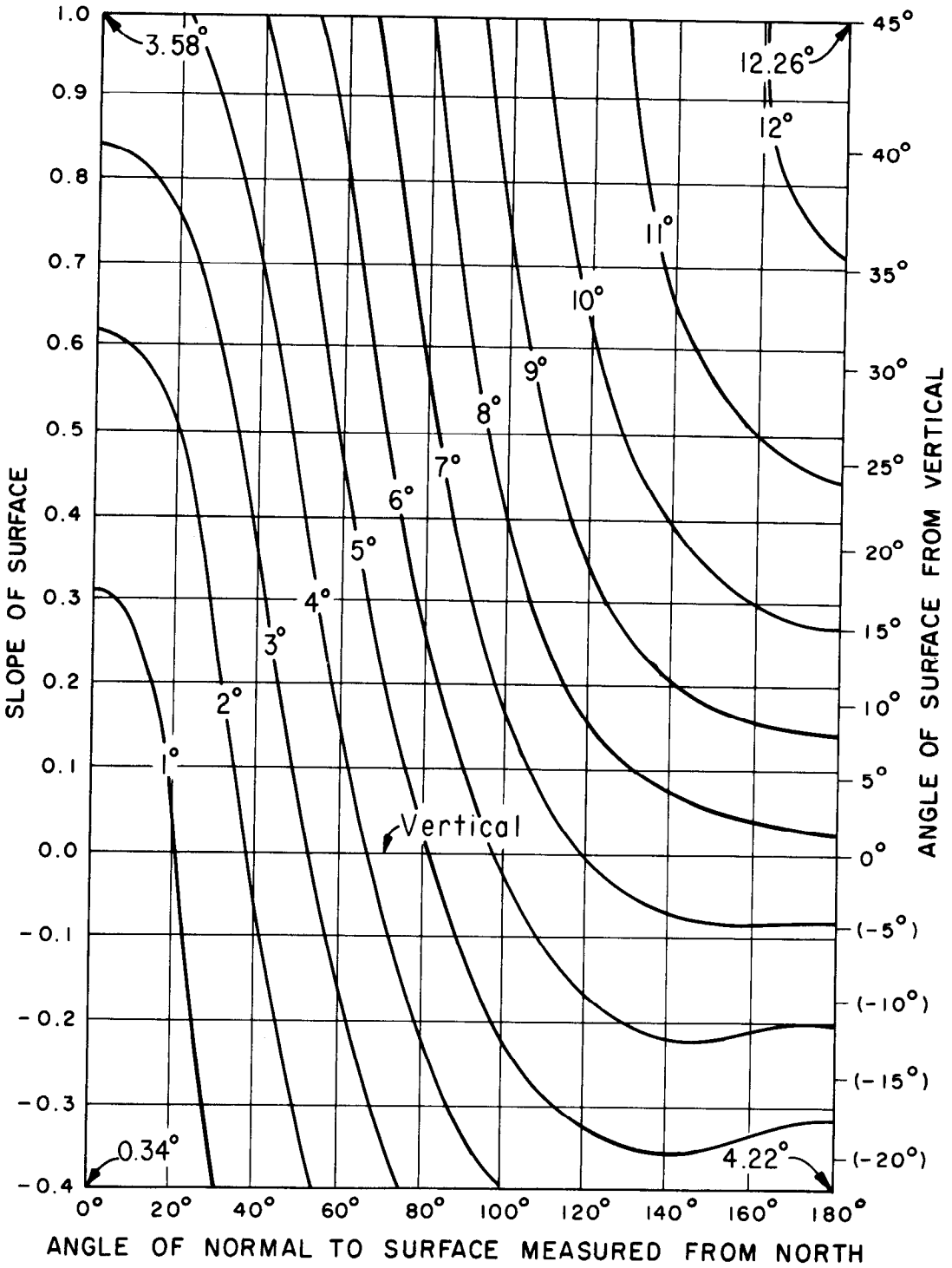
LATITUDES
30°–35°

FIGURE 25.—*Increase in temperature due to solar radiation—Latitudes 30°–35°.*



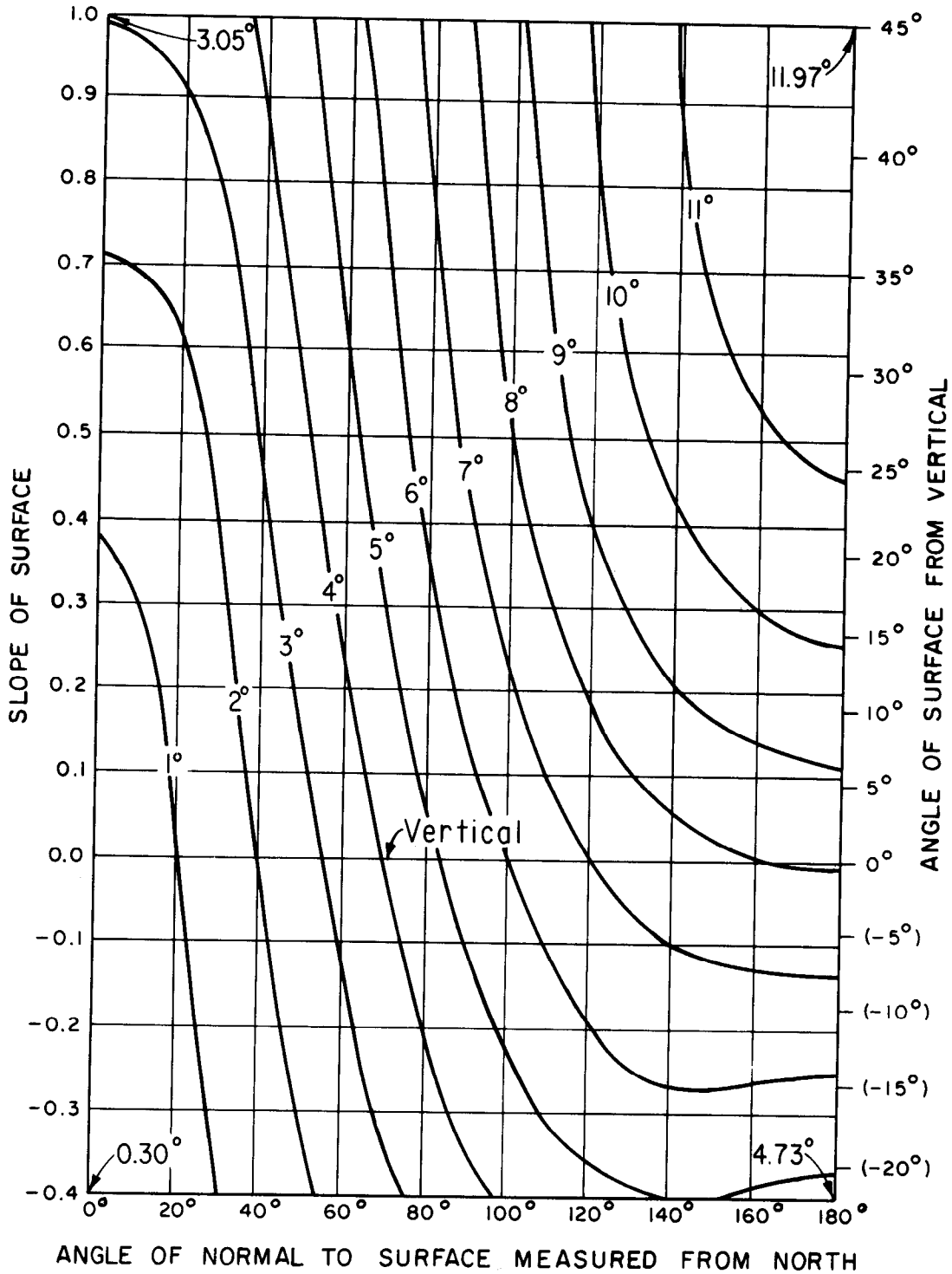
LATITUDES
35°–40°

FIGURE 26.—*Increase in temperature due to solar radiation—Latitudes 35°–40°.*



LATITUDES
40°-45°

Figure 27.—*Increase in temperature due to solar radiation—Latitudes 40°-45°.*



LATITUDES
45°–50°

FIGURE 28.—*Increase in temperature due to solar radiation—Latitudes 45°–50°.*

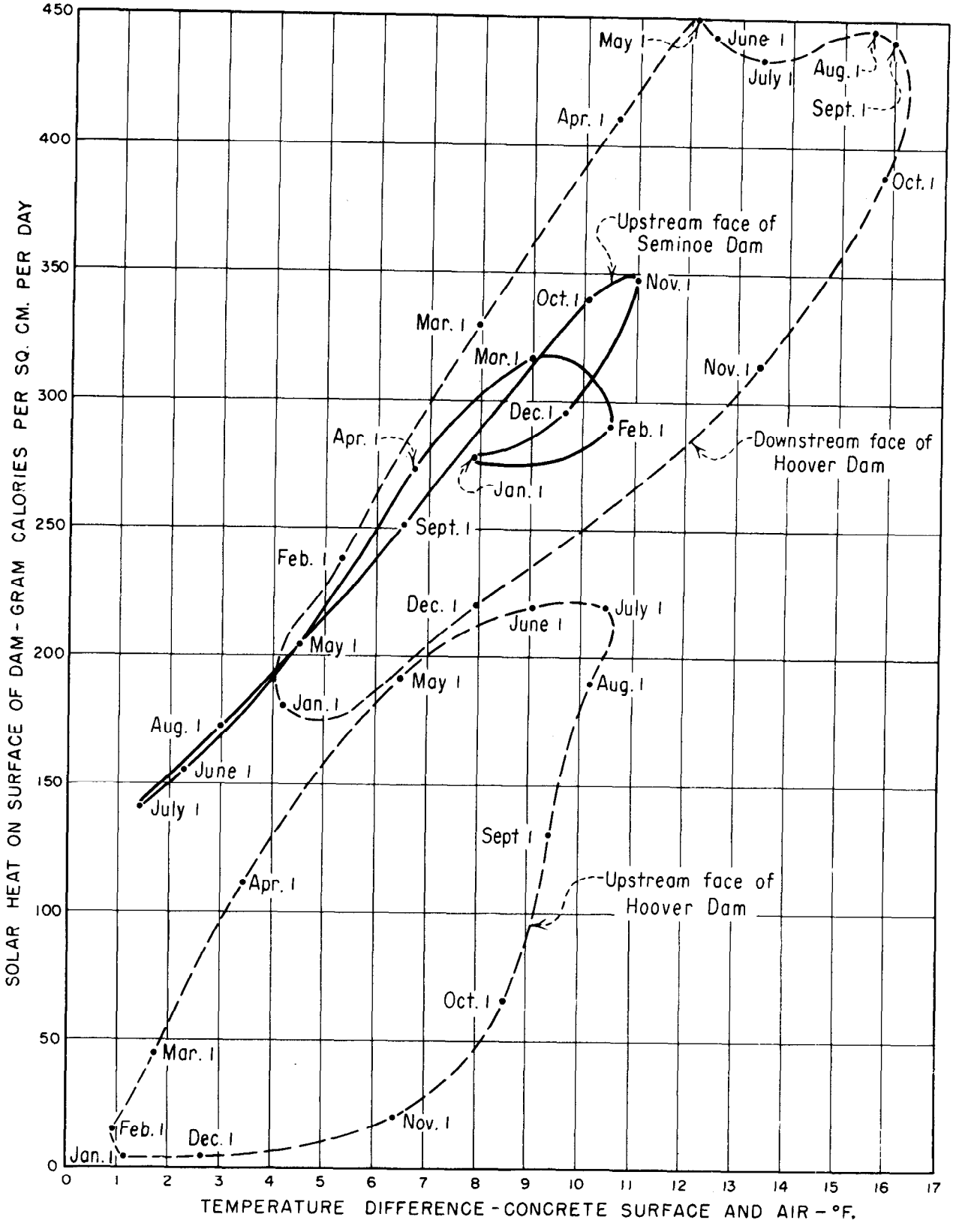


FIGURE 29.—*Variation of solar radiation during year.*

The degree of subcooling, that is, the actual amount of cooling below the final stable temperature, is normally based upon what the trial-load analyses show to be desirable from a stress standpoint; but the degree of subcooling may be influenced by practical or economic considerations. The designer often has to make a design decision whether to lose 2° to 5° of temperature benefit in the arches by using the river water available to cool artificially the concrete in the dam, or whether to obtain the desired temperature benefit by requiring mechanically refrigerated water to perform the cooling.

From the practical standpoint, it is possible to cool the concrete by means of an embedded pipe cooling system to within 4° to 5° of the mean temperature of the cooling water available. Concrete temperatures as low as 35° F. have been obtained with a refrigerating plant using brine as the coolant. Where cooling is accomplished with river water, concrete temperatures depend on the mean river water temperature. At Hungry Horse Dam, 32° to 34° F. river water was available during the colder months of the year, and final cooling was accomplished to 38° F. with this river water. A single closure temperature of 38° F. was determined satisfactory for the particular layout and section of Hungry Horse Dam. At Monticello Dam, river water was both limited in quantity and was relatively warm since the stream primarily carries a surface runoff during periods of rainfall. Refrigeration of the cooling water was required in this instance to obtain the desired closure temperatures. Two closure temperatures were used at Monticello Dam, 45° F. in the lower part of the dam and 55° F. in the upper part of the dam. This was done so that more load could be carried by the lower portion of the dam.

In relatively thin arch dams, pipe cooling may be omitted and the concrete permitted to cool naturally over the winter. Depending on the severity of the exposure conditions, it may then be necessary to wait until the concrete temperatures rise to above 32° F. before grouting the contraction joints. Closure temperatures of 35° to 36° F. can be obtained in these structures. Because of the varying thicknesses, concrete temperatures in thin arch dams which are left to cool naturally will reach the grouting temperature at different times in the several grout lifts. This requires close control over the contraction joint grouting program, and may require that lower portions of the dam be artifically warmed to permit an orderly grouting program which can be accomplished before the top of the dam becomes too warm.

Temperatures in Mass Concrete

Before reaching any final stable state of temperature equilibrium, the temperature history of the concrete is associated with many factors. The placing temperature and the rate of placing the concrete; exposure conditions during placement; the amount and types of cement, pozzolan, and admixtures; and whether any additional temperature control measures such as embedded cooling pipes were used are the most important of these factors. A schematic temperature history of concrete placed in an artificially cooled concrete dam is shown in Figure 30.

At any time during the early age of the concrete, concrete temperatures can be obtained by taking into consideration the flow of heat across an exposed face, the flow of heat through an internal boundary such as the rock foundation or a previously placed lift, the heat of hydration of the freshly placed concrete, the diffusivity of the concrete, and the removal of heat by an embedded cooling system. Afterward, the temperatures existing at any specific point in a mass concrete structure are dependent upon a number of other factors. These include the air and water temperatures adjacent to the structure boundaries, ground temperature, wind, precipitation, relative humidity, solar radiation, and canyon wall reradiation. The fluctuation or range of temperatures experienced at a specific point is also related to the distance to the boundaries of the structure.

In regard to the longtime hydration of cement, the rate of heat generation is related to the type of cement. Standard cement generates the majority of its heat in the first 3 or 4 days after placement; a low-heat cement generates perhaps half of its heat during the same period of time. After a period of a year or two, however, all types of cement have either stopped generating heat or are generating heat at such a low rate that the heat is

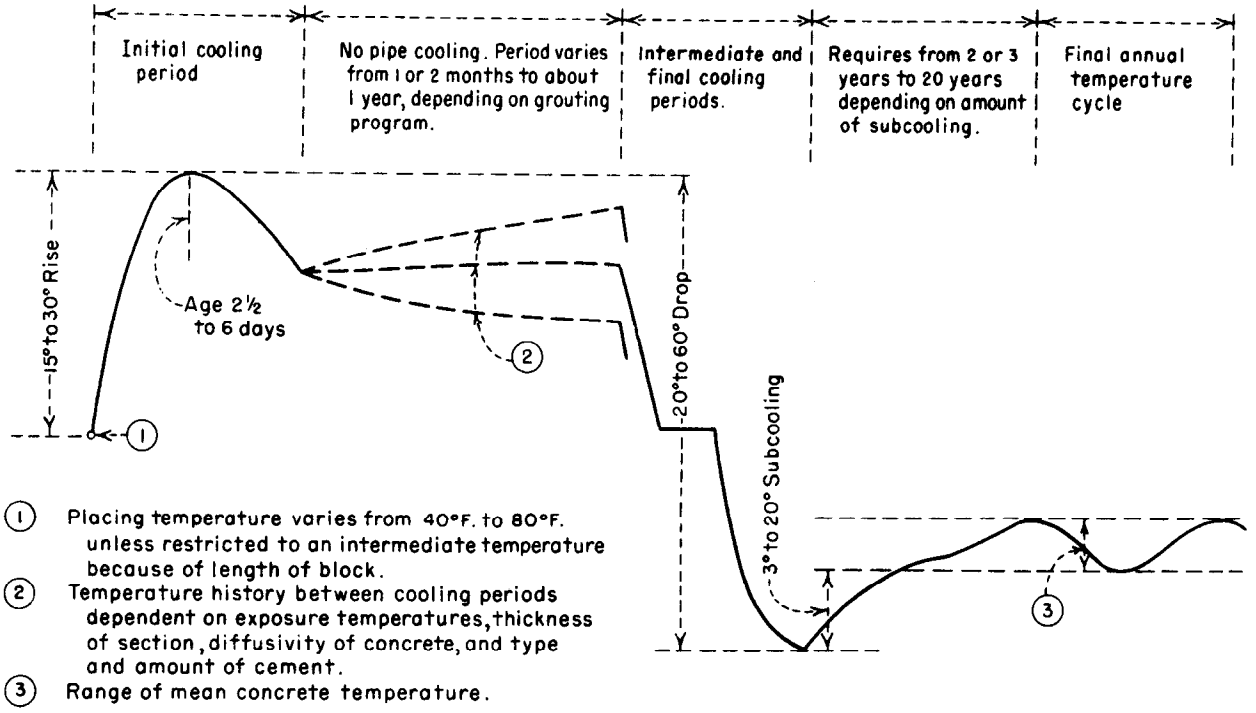


FIGURE 30.—*Temperature history of artificially cooled concrete.*

dissipated to the surfaces of the structure and lost before it can increase the internal concrete temperature. Based on actual temperature observations at Hoover, Shasta, Seminoe, and Hungry Horse Dams, the temperature rises in the concrete subsequent to secondary cooling are almost entirely related to the amount of subcooling accomplished below the final stable temperature and to the exposure conditions existing from the time of such subcooling to the establishment of the final stable temperature.

Average temperatures which occur during operation of a structure, after equilibrium has been established between the concrete temperatures and the external boundary conditions, have been discussed earlier as the range of mean concrete temperature. Often, however, temperature distributions or temperatures at a specific point are desired, and these can be theoretically obtained by several methods, both during the early age of the concrete and in the final state of temperature equilibrium.

The following discussions cover the more common temperature investigations and studies. In most of these studies, certain conditions must be assumed. Since the validity of any heat-flow computation is dependent on the correctness of the assumed exposure conditions and concrete properties, experience and good judgment are essential. To combine the effects of natural and artificial cooling, the solutions of the separate idealized problems are superimposed to obtain final concrete temperatures.

Heat of Hydration.—Newly placed concrete undergoes a rise in temperature due to the exothermic reaction of the cementing materials. The amount of this heat generation is dependent upon the chemical composition, fineness, and amount of cement; the amount and type of pozzolanic material used, if any; the water-reducing agent or retarder used, if any; and the temperature of the concrete during the early period of hydration. Some of these factors may be controlled and used as temperature control measures, others are employed for quality or economy; others must be accepted and used as a given condition.

Portland cement may be considered as being composed of four principal chemical compounds. These are tricalcium silicate, dicalcium silicate, tricalcium aluminate, and tetracalcium alumino-

ferrite, or, as they are more commonly known, C_3S, C_2S, C_3A, and C_4AF. The relative proportions of these chemical compounds determine the different types of cement. Table 5 of the Seventh Edition of the Bureau of Reclamation's *Concrete Manual* gives typical compound composition for the several types of cement. Type I cement is the standard cement and the most commonly used in general construction. Type III cement is a high early-strength cement used primarily in emergency construction and repairs, and in the laboratory where early results are required. Type V cement is a sulfate-resistant cement developed for use where soils or ground water containing relatively high concentrations of sulfates exist.

Because the heat of hydration of cement is largely dependent upon the relative percentages of the chemical compounds in the cement, Types II and IV cement were developed for use in mass concrete construction. Type II cement is commonly referred to as modified cement. It is used where a relatively low-heat generation is desirable and is characterized by low C_3A and high C_4AF contents. Type IV cement is a low-heat cement characterized by its low rate of heat generation during early age. Type IV cement has high percentages of C_2S and C_4AF, and low percentages of C_3S and C_3A. Very often, the run-of-the-mill cement from a plant will meet the requirements of a Type II cement, and the benefits of using this type of cement can be obtained at little or no extra cost. Type IV cement, because of its composition, is obtained at premium prices.

Figure 31 shows typical temperature rise curves for the various types of cement. The temperature rise curves are based on one barrel (four sacks) of cement per cubic yard of concrete and no embedded pipe cooling. These curves should only be used for preliminary studies because there is a wide variation of heat generation within each type of cement. For final temperature control studies, the heat generation [2] should be obtained by laboratory test, using the actual cement, concrete mix proportions, and diffusivity for the concrete under consideration.

Pozzolans are used as a replacement for part of the portland cement to improve workability, to effect economy, to better the quality of the concrete, or to reduce the temperature rise resulting from the hydration of the cementing materials. For early studies, the thermal properties of the different types of pozzolans (fly ash, ground slag, clays, shales, diatomaceous earth, etc.) can be assumed to generate about 50 percent as much heat as the cement it replaced.

Placing concrete at lower temperatures will lower the early rate of heat generation in the concrete. This benefit will be relatively small, as shown in Figure 32. Adiabatic tests in the laboratory normally take this effect into consideration by using the placement temperature and mass cure temperature cycle anticipated for the structure.

Schmidt's Method.—Temperature distributions in a mass where boundary conditions are variable are easily determined by the Schmidt or Schmidt-Binder Method.[3] This method is most frequently used in connection with temperature studies for mass concrete structures in which the temperature distribution is to be estimated. The approximate date for grouting a relatively thin arch dam after a winter's exposure, the depth of freezing, and temperature distributions after placement are typical of the solutions which can be obtained by this step-by-step method. Different exposure temperatures on the two faces of the theoretical slab and heat of hydration can be taken into consideration.

In its simplest form, Schmidt's Method assumes no heat flow normal to the slab and is adapted to a slab of any thickness with any initial temperature distribution. Schmidt's Method states that the temperature, t_2, of an elemental volume at any subsequent time is dependent not only upon its own temperature but also upon the temperatures,

[2] Conversion of calories/gram hydration rise to degree F. temperature rise is made by:

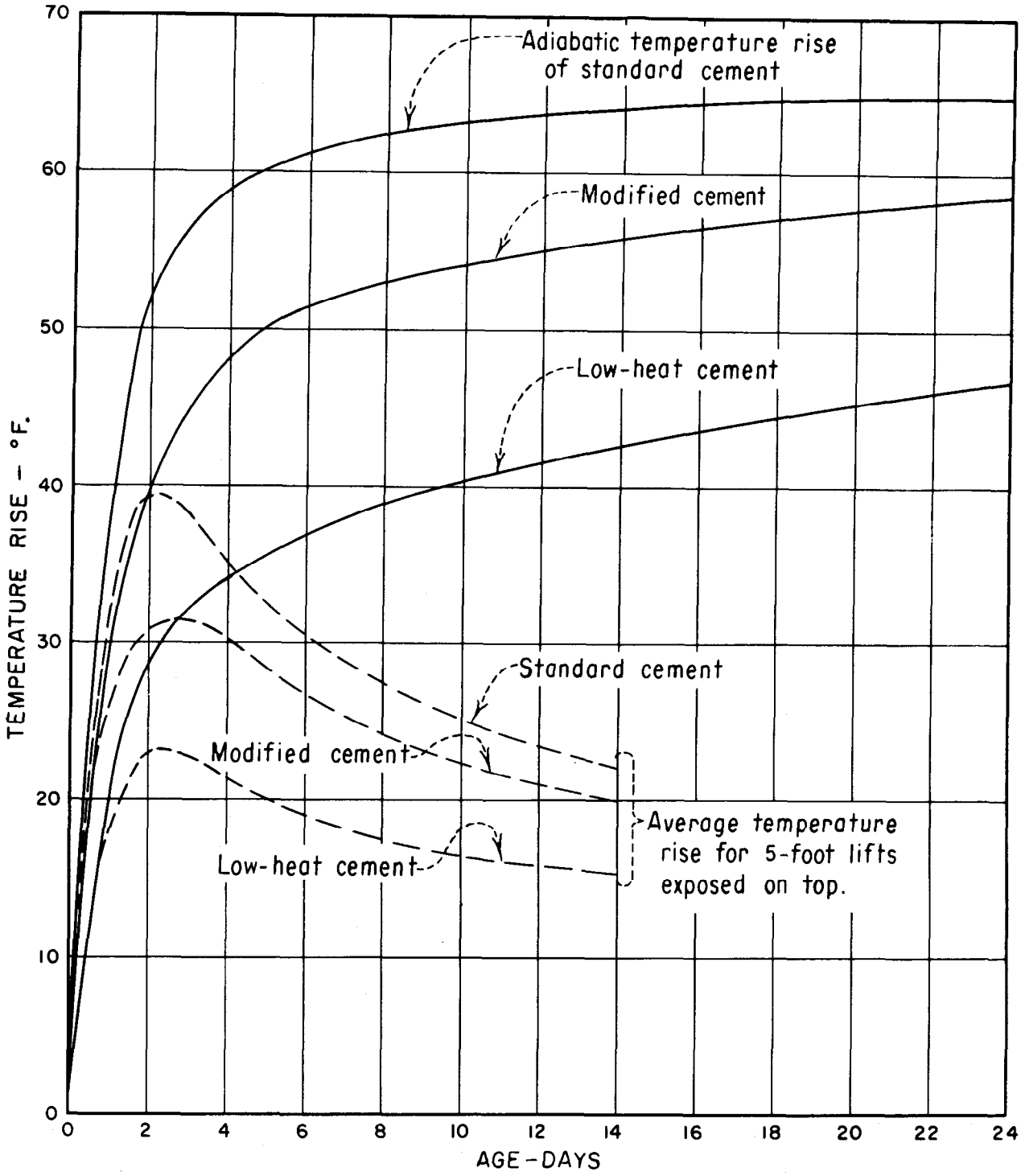
$$\text{Temp. rise } (^\circ\text{F.}) = \frac{1.8(\text{cal/g. hydration})}{C\left(\frac{\text{weight of concrete}}{\text{weight of cement}}\right)}$$

where C=specific heat of concrete and weight is in pounds per cubic yard.

[3] Schack, Alfred, 1933, *Industrial Heat Transfer*, John Wiley and Sons.

Jakob, Max, 1949, *Heat Transfer*, Volume I, pages 373–375, John Wiley and Sons.

Grinter, L. E., 1949, *Numerical Methods of Analysis in Engineering*, page 86, Macmillan Co.



Cement content - 1 bbl per cu yd
Diffusivity - 0.050 ft^2/hr

FIGURE 31.—*Temperature rise in mass concrete—type of cement.*

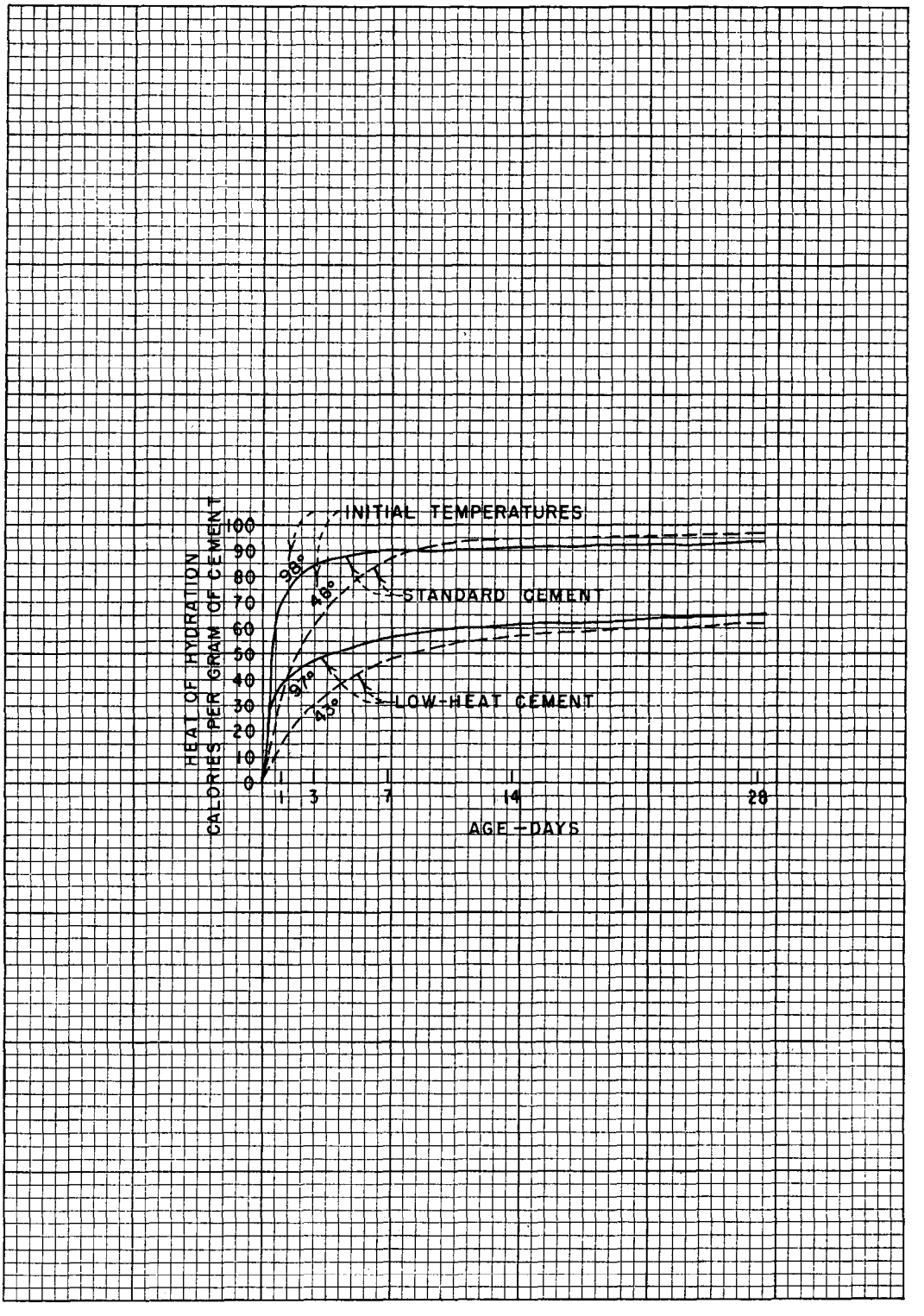


FIGURE 32.—*Effect of initial temperature on heat of hydration.*

TABLE III.—*Related values of Δx and Δt for Schmidt's Method*

h^2 (ft^2/hr)	Δx—feet								$\left(\Delta t=\frac{\Delta x^2}{2h^2}\right)$						
	Δt—hours								Δt—days						
	1	2	3	4	6	8	10	12	1	2	3	4	5	10	15
0.031	0.249	0.352	0.431	0.498	0.610	0.704	0.787	0.863	1.220	1.725	2.113	2.440	2.728	3.857	4.724
.032	.253	.358	.438	.506	.620	.716	.800	.876	1.239	1.753	2.147	2.479	2.771	3.919	4.800
.033	.257	.363	.445	.514	.629	.727	.812	.890	1.259	1.780	2.180	2.517	2.814	3.980	4.874
.034	.261	.369	.451	.521	.639	.738	.825	.903	1.278	1.807	2.213	2.555	2.857	4.040	4.948
.035	.265	.375	.458	.529	.648	.748	.837	.917	1.296	1.833	2.245	2.592	2.898	4.099	5.020
.036	.268	.380	.465	.537	.657	.759	.849	.930	1.315	1.859	2.277	2.629	2.939	4.157	5.091
.037	.272	.385	.471	.544	.666	.769	.860	.942	1.330	1.885	2.308	2.665	2.980	4.214	5.161
.038	.276	.390	.478	.551	.675	.780	.872	.955	1.351	1.910	2.339	2.701	3.020	4.271	5.231
.039	.279	.395	.484	.559	.684	.790	.883	.967	1.368	1.935	2.370	2.736	3.059	4.327	5.299
.040	.283	.400	.490	.566	.693	.800	.894	.980	1.386	1.960	2.400	2.771	3.098	4.382	5.367
.041	.287	.405	.497	.573	.702	.811	.906	.993	1.404	1.986	2.432	2.809	3.140	4.441	5.439
.042	.290	.410	.502	.580	.710	.820	.917	1.004	1.420	2.008	2.459	2.840	3.175	4.490	5.499
.043	.293	.415	.508	.587	.718	.829	.927	1.016	1.437	2.032	2.488	2.873	3.212	4.543	5.564
.044	.297	.420	.514	.593	.727	.839	.938	1.028	1.453	2.055	2.517	2.907	3.250	4.596	5.629
.045	.300	.424	.520	.600	.735	.849	.949	1.039	1.470	2.078	2.546	2.939	3.286	4.648	5.692
.046	.303	.429	.525	.607	.743	.858	.959	1.051	1.486	2.101	2.574	2.972	3.323	4.699	5.755
.047	.307	.434	.531	.613	.751	.867	.970	1.062	1.502	2.124	2.602	3.004	3.359	4.750	5.817
.048	.310	.438	.537	.620	.759	.876	.980	1.073	1.518	2.147	2.629	3.036	3.394	4.800	5.879
.049	.313	.443	.542	.626	.767	.885	.990	1.084	1.534	2.169	2.656	3.067	3.429	4.850	5.940
.050	.316	.447	.548	.632	.774	.894	1.000	1.095	1.549	2.191	2.683	3.098	3.464	4.899	6.000
.051	.319	.452	.553	.639	.782	.903	1.010	1.106	1.565	2.213	2.710	3.129	3.499	4.948	6.060
.052	.322	.456	.559	.645	.790	.912	1.020	1.117	1.580	2.234	2.736	3.160	3.533	4.996	6.119
.053	.326	.460	.564	.651	.797	.921	1.030	1.128	1.595	2.256	2.763	3.190	3.567	5.044	6.177
.054	.329	.465	.569	.657	.805	.930	1.039	1.138	1.610	2.277	2.789	3.220	3.600	5.091	6.235
.055	.332	.469	.574	.663	.812	.938	1.049	1.149	1.625	2.298	2.814	3.250	3.633	5.138	6.293
.056	.335	.473	.580	.669	.820	.947	1.058	1.159	1.640	2.319	2.840	3.279	3.666	5.185	6.350
.057	.338	.477	.585	.675	.827	.955	1.068	1.170	1.654	2.339	2.865	3.308	3.699	5.231	6.406
.058	.341	.482	.590	.681	.834	.963	1.077	1.180	1.669	2.360	2.890	3.337	3.731	5.276	6.462
.059	.344	.486	.595	.687	.841	.972	1.086	1.190	1.683	2.380	2.915	3.366	3.763	5.322	6.518
.060	.346	.490	.600	.693	.849	.980	1.095	1.200	1.697	2.400	2.939	3.394	3.795	5.367	6.573
.061	.349	.494	.605	.699	.856	.988	1.105	1.210	1.711	2.420	2.964	3.422	3.826	5.411	6.627
.062	.352	.498	.610	.704	.863	.996	1.114	1.220	1.725	2.440	2.988	3.450	3.857	5.455	6.681
.063	.355	.502	.615	.710	.869	1.004	1.123	1.230	1.739	2.459	3.012	3.478	3.888	5.499	6.735
.064	.358	.506	.620	.716	.876	1.012	1.131	1.239	1.753	2.479	3.036	3.505	3.919	5.543	6.788
.065	.361	.510	.625	.721	.883	1.020	1.140	1.249	1.766	2.498	3.059	3.533	3.950	5.586	6.841
.066	.363	.514	.629	.727	.890	1.028	1.149	1.259	1.780	2.517	3.083	3.560	3.980	5.628	6.893
.067	.366	.518	.634	.732	.897	1.035	1.158	1.268	1.793	2.536	3.106	3.587	4.010	5.671	6.946
.068	.369	.522	.639	.738	.903	1.043	1.166	1.277	1.807	2.555	3.129	3.613	4.040	5.713	6.997
.069	.371	.525	.643	.743	.910	1.051	1.175	1.287	1.820	2.574	3.152	3.640	4.069	5.755	7.048
.070	.374	.529	.648	.748	.917	1.058	1.183	1.296	1.833	2.592	3.175	3.666	4.099	5.797	7.099

t_1 and t_3, of the adjacent elemental volumes. At time Δt, this can be expressed as:

$$t_{2,\,\Delta t}=\frac{t_1+(M-2)t_2+t_3}{M}$$

where $M=\frac{C\rho(\Delta x)^2}{K\Delta t}=\frac{(\Delta x)^2}{h^2\Delta t}$, since the diffusivity of concrete, h^2, is given as $\frac{K}{C\rho}$. By choosing $M=2$, the temperature, t_2, at a time Δt, becomes $t_{2,\,\Delta t}=\frac{t_1+t_3}{2}$. In this case, by selecting the time and space intervals such that $\Delta t=\frac{(\Delta x)^2}{2h^2}$, the subsequent temperature of an elemental volume is simply the average of the two adjacent elemental temperatures. For $M=2$, Table III gives values of Δt and Δx for various values of h^2.

An example of the use of Schmidt's Method was the determination of concrete temperatures for contraction joint grouting at Ross Dam. At this dam, the temperature distribution at a given elevation was known on August 1, 1945, and it was desired to know the temperature distribution on the following April 1. The diffusivity of Ross Dam concrete was 0.042 ft.^{2}/hr., and a time interval of 10 days was selected as being a reasonable time increment. For these values, a space interval of 4.49 would be required to meet the simple form of Schmidt's Method. Since the diffusivity varies slightly with temperature, the space interval was used as 4.5 feet. For the 126-foot section of dam under consideration, this required 28 space intervals.

The temperature at the center of each space interval was obtained from the initial temperature distribution across the section, as given by embedded strain meter groups and thermometers. An estimate of the exposure conditions on the upstream and downstream faces of the dam between August 1 and April 1 was also required. These data were obtained from the curves of the mean monthly air temperatures and measured reservoir water temperatures. The computations form was then outlined as follows:

Space intervals (Δx) = 4.5 feet

Time intervals (Δt) = 10 days		Exp. temp.	1	2	3	--------	26	27	28	Exp. temp.
	(0)	54	49.0	47.9	47.4	--------	49.9	50.5	52.0	66
	(1)	55	51.0	48.2						66
	(2)	55								65
	(3)	55								63

The computation for the temperature of any space interval at a succeeding time interval then proceeds by simply averaging the two adjacent temperatures in the preceding time interval. For example, the temperature of Space Interval No. 1 at Time Interval No. 1 would be $\frac{54+47.9}{2}=51.0$; the temperature of Space Interval No. 2 at Time Interval No. 1 would be $\frac{49.0+47.4}{2}=48.2$, etc. In this manner, the temperature distribution for Time Interval No. 1 is determined before progressing to Time Interval No. 2, etc. The computations for the Ross Study and the results of these studies are shown in Figures 33 and 34.

The principal objection to the Schmidt Method of temperature computation is the time required to complete the step-by-step computation. This has been overcome by the use of electronic data processing machines which save many man-hours of work. Programs have been prepared which will take into consideration different varying exposures on the two faces of the slab, a varying heat of hydration with time, and the increasing of the slab thickness at regular intervals such as would occur when additional lifts are placed.

Time intervals	Up-stream face	Space intervals													
		1	2	3	4	5	6	7	8	9	10	11	12	13	14
0	54	49. 0	47. 9	47. 4	47. 1	47. 0	47. 4	49. 2	50. 7	51. 4	51. 7	51. 8	51. 8	51. 8	51. 8
1	55	51. 0	48. 2	47. 5	47. 2	47. 2	48. 1	49. 0	50. 3	51. 2	51. 6	51. 8	51. 8	51. 8	51. 8
2	55	51. 6	49. 2	47. 7	47. 4	47. 6	48. 1	49. 2	50. 1	51. 0	51. 5	51. 7	51. 8	51. 8	51. 8
3	55	52. 1	49. 6	48. 3	47. 6	47. 8	48. 4	49. 1	50. 1	50. 8	51. 4	51. 6	51. 8	51. 8	51. 8
4	54	52. 3	50. 2	48. 6	48. 0	48. 0	48. 4	49. 2	50. 0	50. 8	51. 2	51. 6	51. 7	51. 7	51. 8
5	53	52. 1	50. 4	49. 1	48. 3	48. 2	48. 6	49. 2	50. 0	50. 8	51. 2	51. 4	51. 7	51. 7	51. 8
6	52	51. 7	50. 6	49. 4	48. 6	48. 4	48. 7	49. 3	49. 9	50. 6	51. 0	51. 4	51. 6	51. 8	51. 8
7	51	51. 3	50. 6	49. 6	48. 9	48. 6	48. 8	49. 3	50. 0	50. 4	51. 0	51. 3	51. 6	51. 7	51. 8
8	50	50. 8	50. 4	49. 8	49. 1	48. 8	49. 0	49. 4	49. 8	50. 5	50. 8	51. 3	51. 5	51. 7	51. 8
9	48	50. 2	50. 3	49. 8	49. 3	49. 0	49. 1	49. 4	50. 0	50. 3	50. 9	51. 2	51. 5	51. 6	51. 8
10	46	49. 2	50. 0	49. 8	49. 4	49. 2	49. 2	49. 6	49. 8	50. 4	50. 8	51. 2	51. 4	51. 6	51. 8
11	44	48. 0	49. 5	49. 7	49. 5	49. 3	49. 4	49. 5	50. 0	50. 3	50. 8	51. 1	51. 4	51. 6	51. 8
12	42	46. 8	48. 8	49. 5	49. 5	49. 4	49. 4	49. 7	49. 9	50. 4	50. 7	51. 1	51. 4	51. 6	51. 8
13	41	45. 4	48. 2	49. 2	49. 4	49. 4	49. 6	49. 6	50. 0	50. 3	50. 8	51. 0	51. 4	51. 6	51. 8
14	40	44. 6	47. 3	48. 8	49. 3	49. 5	49. 5	49. 8	50. 0	50. 4	50. 6	51. 1	51. 3	51. 6	51. 8
15	40	43. 6	46. 7	48. 3	49. 1	49. 4	49. 6	49. 8	50. 1	50. 3	50. 8	51. 0	51. 4	51. 6	51. 8
16	40	43. 4	46. 0	47. 9	48. 8	49. 4	49. 6	49. 8	50. 2	50. 4	50. 6	51. 1	51. 3	51. 6	51. 8
17	40	43. 0	45. 6	47. 4	48. 6	49. 2	49. 6	49. 9	50. 1	50. 4	50. 8	51. 0	51. 4	51. 6	51. 8
18	40	42. 8	45. 2	47. 1	48. 3	49. 1	49. 6	49. 8	50. 2	50. 4	50. 7	51. 1	51. 3	51. 6	51. 8
19	40	42. 6	45. 0	46. 8	48. 1	49. 0	49. 4	49. 9	50. 1	50. 4	50. 8	51. 0	51. 4	51. 6	51. 8
20	40	42. 5	44. 7	46. 6	47. 9	48. 8	49. 4	49. 8	50. 2	50. 4	50. 7	51. 1	51. 3	51. 6	51. 8
21	40	42. 4	44. 6	46. 3	47. 7	48. 6	49. 3	49. 8	50. 1	50. 4	50. 8	51. 0	51. 4	51. 6	51. 8
22	40	42. 3	44. 4	46. 2	47. 4	48. 5	49. 2	49. 7	50. 1	50. 4	50. 7	51. 1	51. 3	51. 6	51. 8
23	40	42. 2	44. 2	45. 9	47. 4	48. 3	49. 1	49. 6	50. 0	50. 4	50. 8	51. 0	51. 4	51. 6	51. 8
24	40	42. 1	44. 0	45. 8	47. 1	48. 2	49. 0	49. 6	50. 0	50. 4	50. 7	51. 1	51. 3	51. 6	51. 8

Time intervals	Space intervals														Down-stream face
	15	16	17	18	19	20	21	22	23	24	25	26	27	28	
0	51. 8	51. 9	52. 1	52. 3	52. 3	52. 0	51. 7	51. 0	50. 1	49. 7	49. 5	49. 9	50. 5	52. 0	66
1	51. 8	52. 0	52. 1	52. 2	52. 2	52. 0	51. 5	50. 9	50. 4	49. 8	49. 8	50. 0	51. 0	58. 2	66
2	51. 9	52. 0	52. 1	52. 2	52. 1	51. 8	51. 4	51. 0	50. 4	50. 1	49. 9	50. 4	54. 1	58. 5	65
3	51. 9	52. 0	52. 1	52. 1	52. 0	51. 8	51. 4	50. 9	50. 6	50. 2	50. 2	52. 0	54. 4	59. 6	63
4	51. 9	52. 0	52. 0	52. 0	52. 0	51. 7	51. 4	51. 0	50. 6	50. 4	51. 1	52. 3	55. 8	58. 7	61
5	51. 9	52. 0	52. 0	52. 0	51. 8	51. 7	51. 4	51. 0	50. 7	50. 8	51. 4	53. 4	55. 5	58. 4	59
6	51. 9	52. 0	52. 0	51. 9	51. 8	51. 6	51. 4	51. 0	50. 9	51. 0	52. 1	53. 4	55. 9	57. 2	56
7	51. 9	52. 0	52. 0	51. 9	51. 8	51. 6	51. 3	51. 2	51. 0	51. 5	52. 2	54. 0	55. 3	56. 0	53
8	51. 9	52. 0	52. 0	51. 9	51. 8	51. 6	51. 4	51. 2	51. 4	51. 6	52. 8	53. 8	55. 0	54. 2	50
9	51. 9	52. 0	52. 0	51. 9	51. 8	51. 6	51. 4	51. 4	51. 4	52. 1	52. 7	53. 9	54. 0	52. 5	48
10	51. 9	52. 0	52. 0	51. 9	51. 8	51. 6	51. 5	51. 4	51. 8	52. 0	53. 0	53. 4	53. 2	51. 0	43
11	51. 9	52. 0	52. 0	51. 9	51. 8	51. 6	51. 5	51. 6	51. 7	52. 4	52. 7	53. 1	52. 2	48. 1	40
12	51. 9	52. 0	52. 0	51. 9	51. 8	51. 6	51. 6	51. 6	52. 0	52. 2	52. 8	52. 4	50. 6	46. 1	37
13	51. 9	52. 0	52. 0	51. 9	51. 8	51. 7	51. 6	51. 8	51. 9	52. 4	52. 3	51. 7	49. 2	43. 8	35
14	51. 9	52. 0	52. 0	51. 9	51. 8	51. 7	51. 8	51. 8	52. 1	52. 1	52. 0	50. 8	47. 8	42. 1	33
15	51. 9	52. 0	52. 0	51. 9	51. 8	51. 8	51. 8	52. 0	52. 0	52. 0	51. 4	49. 9	46. 4	40. 4	31
16	51. 9	52. 0	52. 0	51. 9	51. 8	51. 8	51. 9	51. 9	52. 0	51. 7	51. 0	48. 9	45. 2	38. 7	30
17	51. 9	52. 0	52. 0	51. 9	51. 8	51. 8	51. 8	52. 0	51. 8	51. 5	50. 3	48. 1	43. 8	37. 6	31
18	51. 9	52. 0	52. 0	51. 9	51. 8	51. 8	51. 9	51. 8	51. 8	51. 0	49. 8	47. 0	42. 8	37. 4	33
19	51. 9	52. 0	52. 0	51. 9	51. 8	51. 8	51. 8	51. 8	51. 4	50. 8	49. 0	46. 3	42. 2	37. 9	34
20	51. 9	52. 0	52. 0	51. 9	51. 8	51. 8	51. 8	51. 6	51. 3	50. 2	48. 6	45. 6	42. 1	38. 1	35
21	51. 9	52. 0	52. 0	51. 9	51. 8	51. 8	51. 7	51. 6	50. 9	50. 0	48. 4	45. 4	41. 8	38. 6	37
22	51. 9	52. 0	52. 0	51. 9	51. 8	51. 8	51. 7	51. 3	50. 8	49. 6	47. 7	45. 1	42. 0	39. 4	39
23	51. 9	52. 0	52. 0	51. 9	51. 8	51. 8	51. 6	51. 2	50. 4	49. 2	47. 4	44. 8	42. 2	40. 5	42
24	51. 9	52. 0	52. 0	51. 9	51. 8	51. 7	51. 5	51. 0	50. 2	48. 9	47. 0	44. 8	42. 6	42. 1	45

FIGURE 33.—*Ross Dam—Schmidt's Method of temperature computation.*

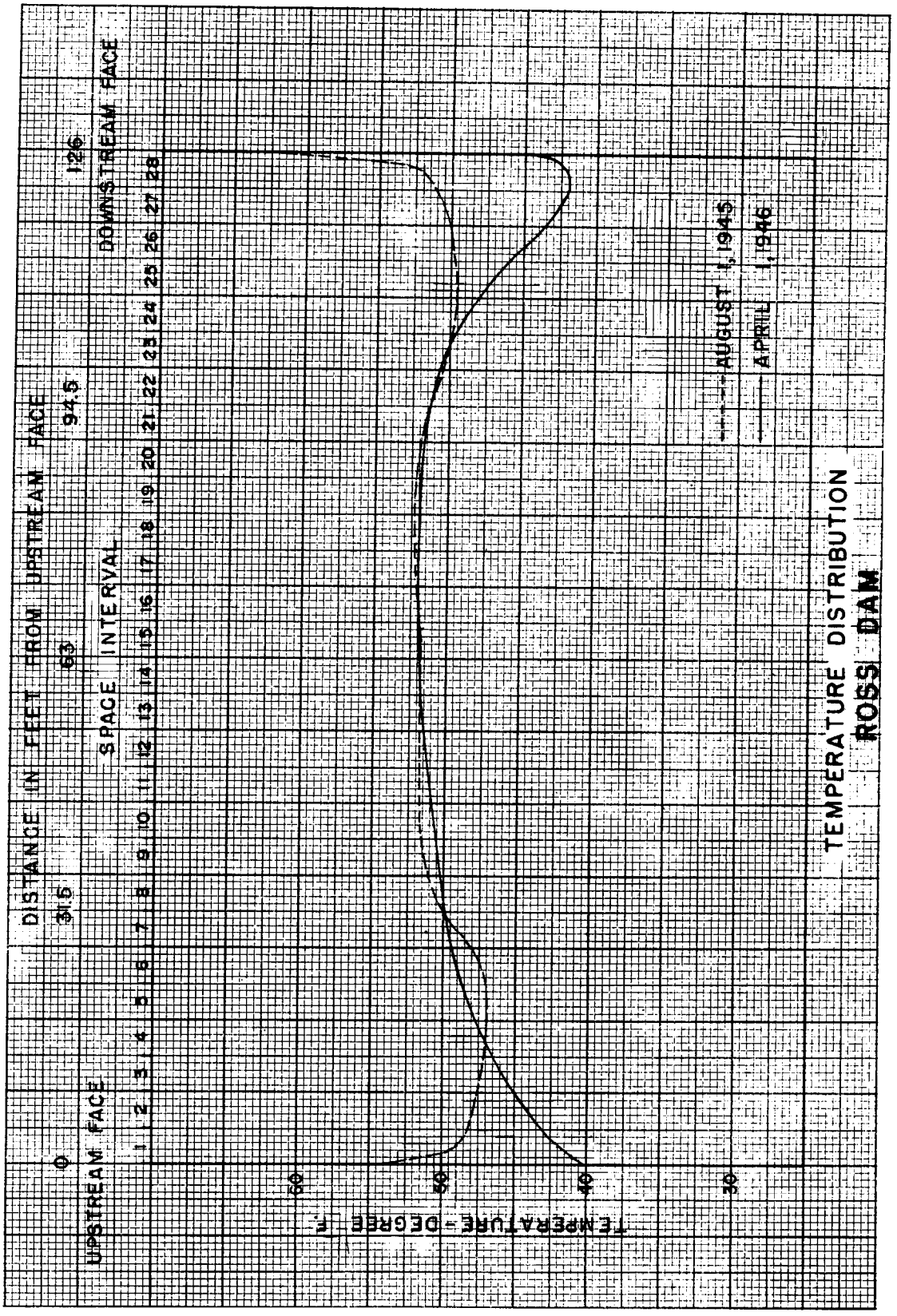


FIGURE 34.—*Ross Dam—temperature distribution by Schmidt's Method.*

Carlson's Method.—A method of temperature computation in mass concrete which is particularly adaptable to temperatures occurring in thick walls and placement lifts near the rock foundation was devised by Roy W. Carlson.[4] This method, like the Schmidt Method, is essentially a step-by-step integration which can be simplified by selection of certain variables. Conditions such as initial temperature distributions, diffusivity, and adiabatic temperature rise must be known or assumed.

The summary of equations used in Carlson's Method is as follows:

All stations except center	*Center station*
$t_n = L_n + Z_n t_{n-1}$	$= L_n + Z_n t_{n-1}$
$L_n = (C_n + L_{n+1}) Z_n$	$= \frac{C_n Z_n}{2}$
$C_n = t'_{n-1} + (2S-2)t'_n + t'_{n+1} + 2S\Delta\theta$	$= 2t'_{n+1} + (2S-2)t'_n + 2S\Delta\theta$
$S = \frac{X^2}{h^2 T}$	$= \frac{X^2}{h^2 T}$
$Z_n = \frac{1}{2S+2-Z_{n+1}}$	$= \frac{1}{S+1}$

where t'_n=initial temperature at station "n"
t_n=final temperature at station "n"
$\Delta\theta$=adiabatic temperature rise in time interval T
T=time interval in days
X=space interval in feet
h^2=diffusivity of concrete in ft.^{2}/day.

To expedite the computations, the following are numerical values of Z_n for various values of S:

Station	$S=4$	$S=2$	$S=1$	$S=\frac{1}{2}$	$S=\frac{1}{4}$	$S=\frac{1}{8}$	$S=\frac{1}{16}$
Distant from center	0.101	0.1715	0.2680	0.3820	0.5000	0.6100	0.7030
Fifth from center	.101	.1715	.2680	.3820	.5006	.6124	.7138
Fourth from center	.101	.1715	.2680	.3821	.5025	.6169	.7241
Third from center	.101	.1716	.2680	.3830	.5058	.6289	.7440
Second from center	.101	.1717	.2692	.3899	.5230	.6601	.7812
Next to center	.102	.1765	.2857	.4286	.5882	.7348	.8446
Center	.200	.3333	.5000	.6667	.8000	.8889	.9410

For a sample computation, assume a 5-foot-thick wall (2.5-foot half-thickness) with noninsulating forms. Select $X=0.5$ feet, $T=\frac{1}{4}$ day, and $h^2=1.00$ ft.^{2}/day. Therefore, $S=1$. Assume $\Delta\Theta$ as 6.2° F. for the first ¼ day and 8.5° F. for the second ¼ day. Assume both the initial and exposure temperature of the concrete as 70° F. from time zero to the end of the first time interval, with the exposure temperature rising to 74° F. 6 hours later. Using a relative temperature of zero rather than an actual temperature of 70° F. simplifies the computation. Columns (1), (2), (3), and (4) are then completed as shown in Table IV.

For each time interval, the first step is to compute the C-values for each station and enter these values in Column (5) of Table IV. These C-values are computed from the top down. Since $S=1$, $C_n = t'_{n-1} + t'_{n+1} + 2\Delta\theta = 0+0+2(6.2) = 12.4$. This holds for all values of n since $t'=0$ at all stations for this first time interval. For the center station, $t'_{n+1} = t'_{n-1}$.

The second step is to compute the L-values for each station and enter these values in Column (6) of Table IV. Here, the computation is begun at the bottom of the column because successive values of L_n are dependent on L_{n+1}. At the center station, $L_n = \frac{C_n Z_n}{2} = \frac{12.4(0.500)}{2} = 3.10$. Successive L values are then computed from the equation $L_n = (C_n + L_{n+1}) Z_n$. For L_4, this becomes $L_4 = (C_4 + L_5) Z_4 = (12.4+3.10)(0.286) = 4.43$.

The final step, for the first time-interval computations, is to compute the final temperatures in Column (7) of Table IV. These are computed in the normal manner from the top of the column using the equation $t_n = L_n + Z_n t_{n-1}$. At Station 1, $t_1 = L_1 + Z_1 t_0 = 4.54 + 0.268(0) = 4.5$; Station 2, $t_2 = 4.54 + (0.268)4.5 = 5.7$; etc.

The temperatures "t" found for the first time interval now become "t'" for the second time interval. This process is repeated, employing the proper $\Delta\theta$ and surface temperature, for succeeding time intervals. This is carried on for as many time intervals as necessary, usually many more than the two steps illustrated. For most cases, the time interval should not exceed ¼ day. At the end of the computations, the relative temperature is converted back to actual temperature.

Carlson's Method of temperature computation can also be modified to take into account the flow of heat between different materials. This would

[4] "A Simple Method for the Computation of Temperatures in Concrete Structures," Volume 34, ACI *Proceedings* (November–December 1937 ACI *Journal*).

TABLE IV.—*Computation of temperatures in 5-foot-thick concrete wall*

(1)	(2)	(3)	(4)	(5)	(6)	(7)			
				1st time interval			2d time interval		
Sta. (n)	Depth below surface (feet)	t' °F.	Z	$\Delta\theta=6.2°$			$\Delta\theta=8.5°$		
				C↓	L↑	t °F.↓	C↓	L↑	t °F.↓
0	0	0				0.0			4.0
1	0.5	0	0.268	12.4	4.54	4.5	22.7	8.83	9.9
2	1.0	0	.268	12.4	4.54	5.7	27.6	10.23	12.9
3	1.5	0	.269	12.4	4.53	6.1	28.9	10.59	14.1
4	2.0	0	.286	12.4	4.43	6.2	29.3	10.48	14.5
5	2.5	0	.500	12.4	3.10	6.2	29.4	7.35	14.6

be the case where insulated or partially insulated forms are used, or where concrete lifts are placed on the rock foundations. In these instances, the given equations cannot be simplified to the extent desired, and the thermal conductivities, specific heats, and space intervals must be taken into consideration. Changing the space intervals in one material so as to maintain a fixed S is not permissible. To the above list of equations must be added additional values of Z_j, C_j, and L_j when passing from one material to another. Examples of these computations are presented in Carlson's paper on pages 96–98.

Temperature Distributions.—Problems involving temperature distributions near exposed faces may be solved in several ways. Schmidt's Method may be used, for example, when it is desired to know the depth of freezing so that piping systems within the mass concrete will not freeze. The same method is used advantageously to determine when a concrete section completely thaws out after it has been exposed over a winter and frozen throughout its thickness. This was the situation at Seminoe Dam where the top portion of the dam was to be grouted as early in the spring as possible.

The variation in temperature at any particular point can also be estimated by the use of Figure 35. This figure gives the ratio of the temperature range in the concrete at any given depth to the temperature range at the surface for the daily, 15-day, and annual cycles of temperature.

Removal of Heat by Cooling Pipe.—The theory for the removal of heat from concrete by embedded cooling pipes was first developed for use in Hoover Dam cooling studies. From these studies, a number of curves and nomographs were prepared for a vertical spacing (height of placement lift) of 5 feet for application at Hoover Dam. Given concrete properties and a single rate of flow of water were also used as constants. Later, the theory was developed using dimensionless parameters. The computations and results of these later studies are presented in the Bureau of Reclamation's Technical Memorandum No. 630, "Cooling of Mass Concrete by Water Flowing in Embedded Pipes," by V. M. Kingston, and in "Cooling of Concrete Dams," Bulletin 3, Part VII—Cement and Concrete Investigations of the Boulder Canyon Project Final Reports. For the more general application, Figures 36, 37, and 38 are master curves for the computation of temperatures resulting from artificial cooling. Figure 36 shows values of "X" for various conditions of concrete properties, flow of cooling water, length of cooling coil, horizontal and vertical spacings, and time. In this case, "X" represents the final mean temperature difference in degrees per degree initial temperature difference, or

$$X=\frac{\text{Mean temp. of entire length of cylinder—initial temp. of water.}}{\text{Initial temp. of concrete—initial temp. of water.}}$$

For the same variables, curves of "Y" and "Z" are given in Figures 37 and 38. The "Y" curve is used to compute the temperature rise in the cooling water and the "Z" curve is used to compute the mean temperature of concrete at a given length

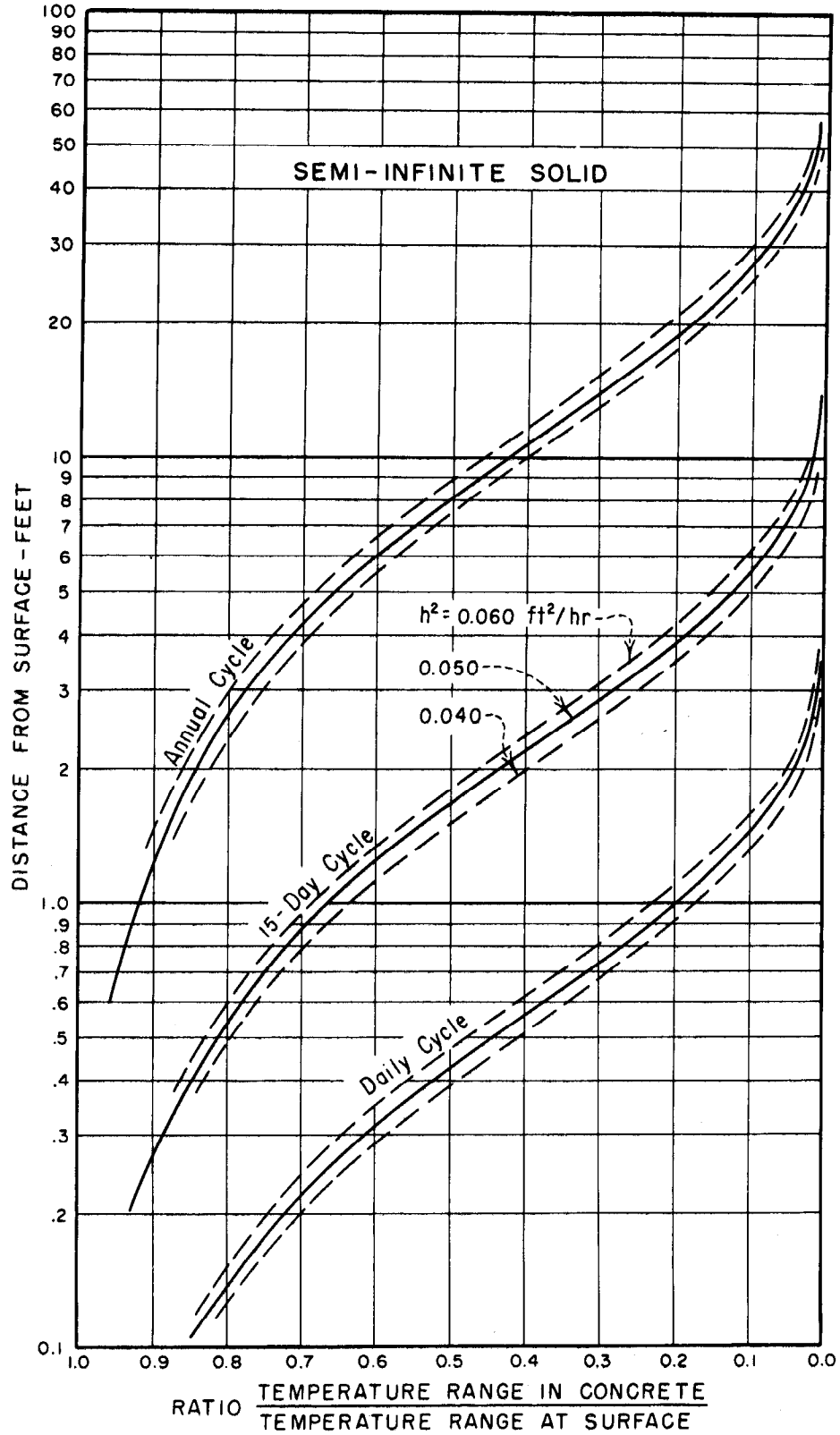


FIGURE 35.—*Temperature variations with depth in semi-infinite solid.*

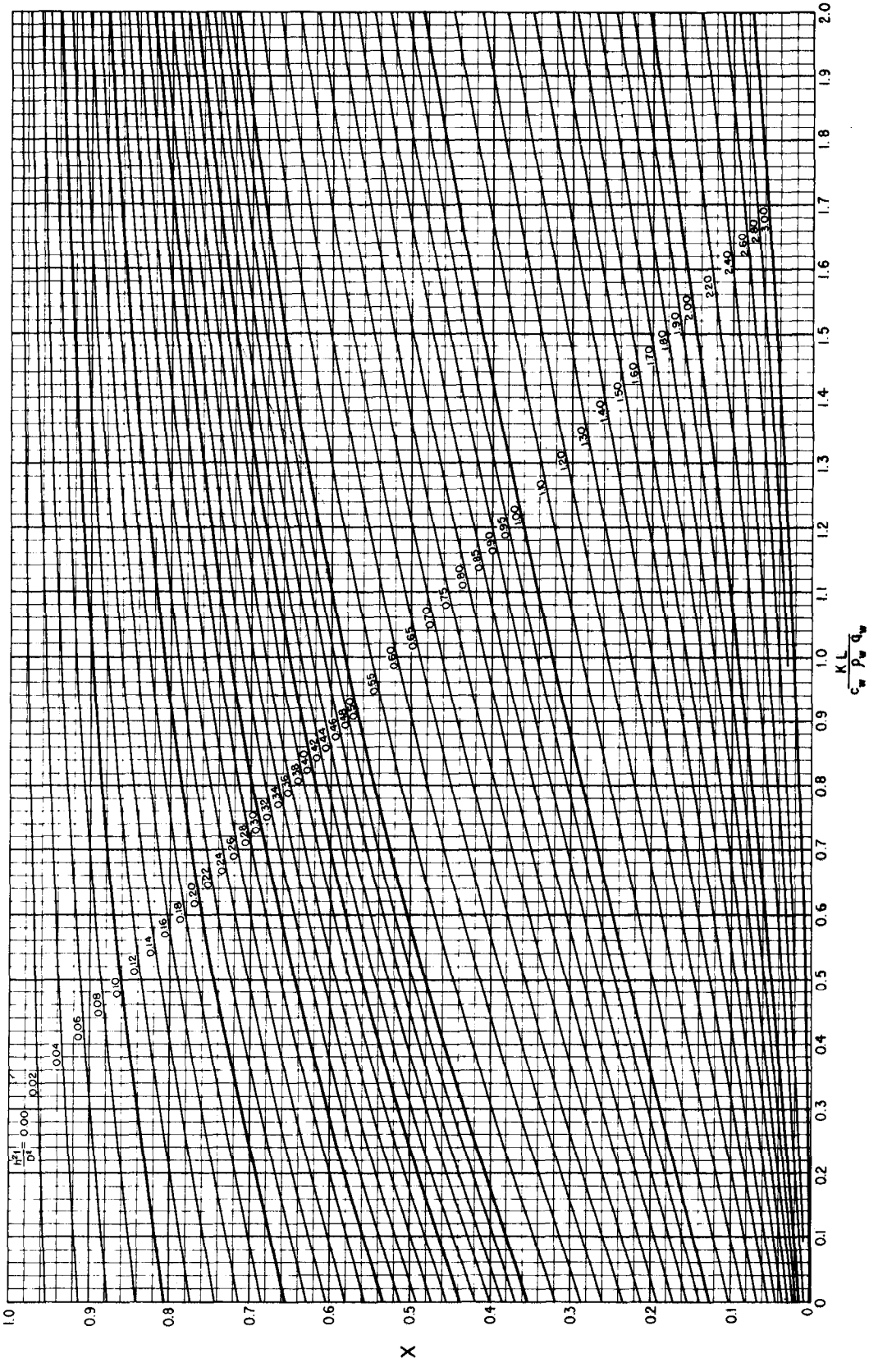


FIGURE 36.—*Pipe cooling of concrete—values of "X".*

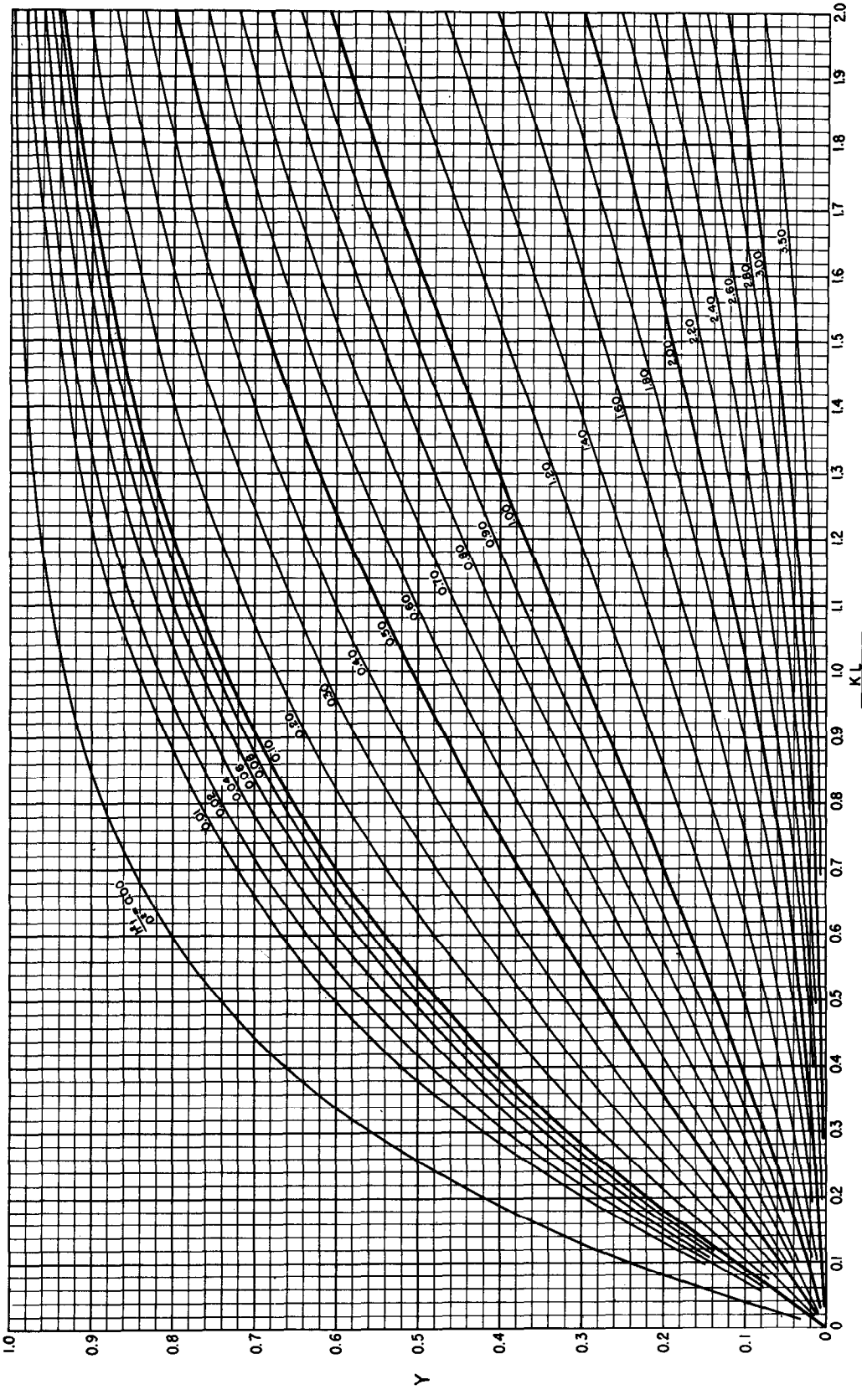


FIGURE 37.—*Pipe cooling of concrete—values of "Y".*

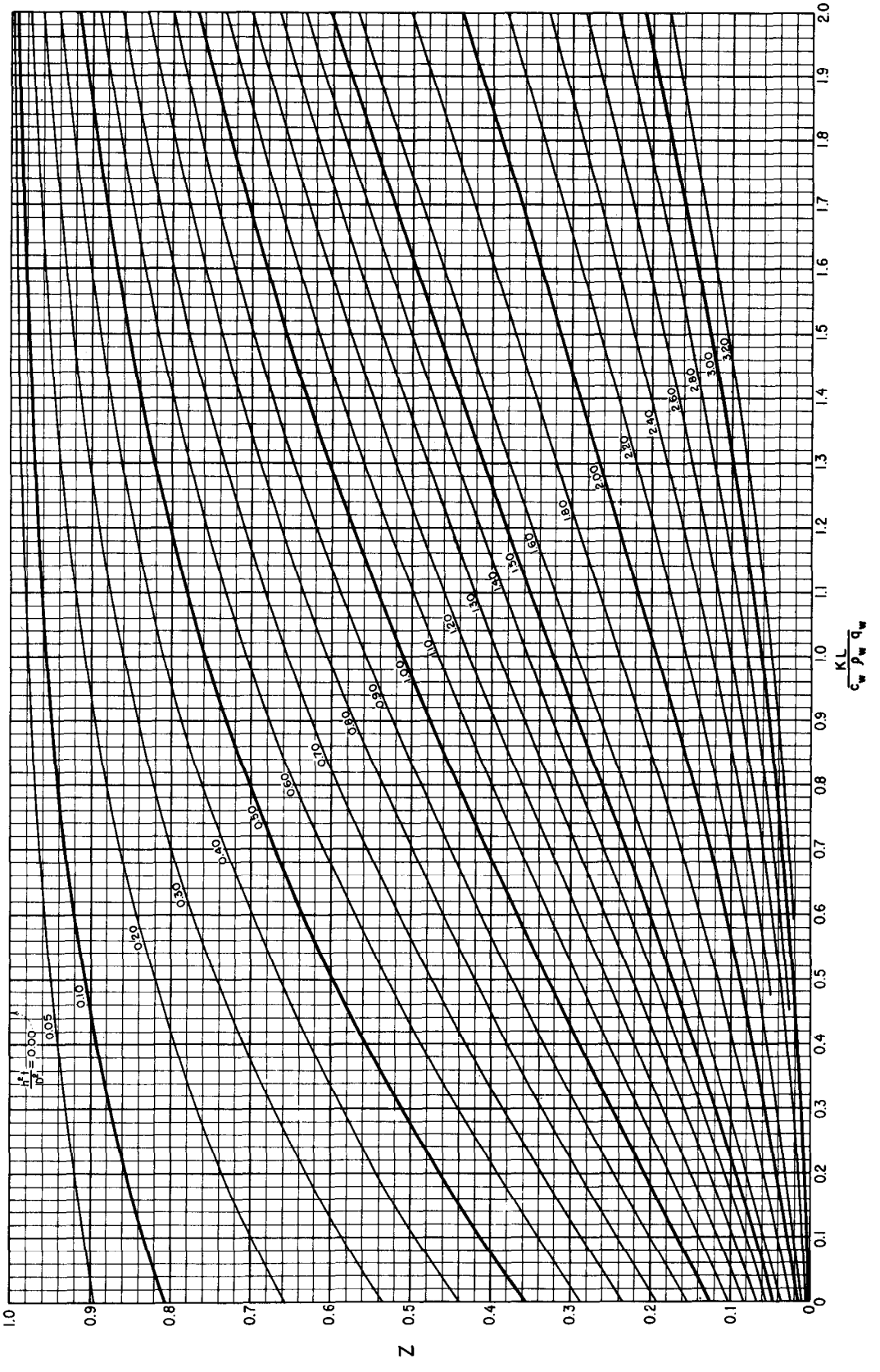


FIGURE 38.—*Pipe cooling of concrete—values of "Z".*

from the inlet. The latter curve is not normally used because the normal field practice is to reverse the flow of the cooling water each day.

In Figures 36, 37, and 38, the following nomenclature is followed:

K=conductivity of the concrete to be cooled in B.t.u./ft /hr /°F.
L=length of the cooled cylinder measured along the cooling pipe in feet
c_w=specific heat of the cooling water in B.t.u./lb /°F.
ρ_w=density of the cooling water in lb /ft ^{3}
q_w=volume of water flowing through the cooling pipe in ft ^{3}/hr
D=diameter of the cooled cylinder in feet
h^2=diffusivity of the concrete in ft ^{2}/hr
t=total elapsed time the cooling has been in operation in hours.

The curves were computed for a ratio of b/a of 100, where b is the radius of the cooled cylinder and a is the radius of the cooling pipe. Actual cooling pipe spacings are nominal spacings and will seldom if ever be spaced so as to obtain a b/a ratio of 100. In order to take the actual horizontal and vertical spacings into consideration, a fictitious diffusivity constant should be used. This is acceptable between certain limits as the cooling rate is proportional to the diffusion constant. Table V gives the values of D, D^2, and h^2_f for various spacings of cooling pipe. The b/a ratios of the spacings shown vary from about 34 to 135. Within these limits, the values of h^2_f may be used with sufficient accuracy.

The curves given in Figures 36, 37, and 38 are used in a straight-forward manner as long as no appreciable heat of hydration is occurring in the concrete during the period of time under consideration. When the effect of artificial cooling is desired during the early age of the concrete, that is, when a high rate of heat generation is occurring, a step-by-step computation is required which takes into consideration heat increments added at uniform time intervals during the period. These heat increments are obtained from the results of a separate study which takes into consideration the adiabatic temperature rise, the loss or gain of heat to the surface of the lift due to exposure conditions, and the loss or gain of heat from an underlying lift or to the foundation. Such a study may be done by Schmidt's Method, or by the method outlined in Chapter V of the Boulder Canyon report "Cooling of Concrete Dams."

TABLE V.—*Values of D, D^2, and h^2_f for pipe cooling*

Spacing		D	D^2	h^2_f
Vertical (feet)	Horizontal (feet)			
2½	2½	2. 82	7. 95	1. 31h²
5	2½	3. 99	15. 92	1. 19h²
5	3	4. 35	18. 92	1. 16
5	4	5. 02	25. 20	1. 12
5	5	5. 64	31. 81	1. 09
5	6	6. 18	38. 19	1. 07
7½	2½	4. 88	23. 81	1. 13h²
7½	4	6. 15	37. 82	1. 07
7½	5	6. 86	47. 06	1. 04
7½	6	7. 54	56. 85	1. 02
7½	7½	8. 46	71. 57	1. 00
7½	9	9. 26	85. 75	0. 98
10	10	11. 284	127. 33	0. 94h²

Several difficulties are encountered in this latter method when attempting to use the formulas for heat losses or gains from a heat-generating lift, from an inert lift, and from an inert semi-infinite solid. For example, the theoretical equation for the adiabatic temperature rise is given as $T=T_0(1-e^{-mt})$ and T_0 and m are selected to make the theoretical curve fit the actual laboratory data. This is not easily done at times, and can result in some error in the theoretical heat loss in the heat-generating lift. The loss from the inert lift cannot take into consideration a varying surface temperature which also introduces an error. The third error is introduced when a new lift is placed on an older lift which is still generating heat. Unless the older lift is quite old, the heat being generated may be enough to be taken into consideration. Table VI gives an example of the computation using Figure 36. In this case, the average temperature during the interval and the temperature at the end of the interval, both without cooling water, were obtained by a Schmidt's Method computation.

TABLE VI.—*Effect of artificial cooling pipe*

GIVEN CONDITIONS

7½-foot lifts
6-foot horizontal spacing
800-foot coil
50° F. placing temperature

40° F. initial cooling water temp.
$h^2=1.248$ ft ^{2}/day
$K=1.7$ B.t.u./ft /hr /F.
$c_w\rho_w q_w=2{,}000$

$$\therefore \frac{KL}{c_w\rho_w q_w}=0.68;\ D^2=56.85;\ h_f^2=1.02h^2=1.273$$

t (days)	$\frac{h^2 t}{D^2}$	X	1−X	Avg. temp. during interval w/o cooling water	Heat increment	Heat loss	Temp. at end of interval w/o cooling water	Temp. at end of interval with cooling water
0								50.0
0.5	0.011	0.99	0.01	62.0	[1] 22.0	0.2	65.5	65.3
1.0	0.022	0.98	0.02	68.5	6.5	0.5	70.3	69.8
1.5	0.034	0.96	0.04	71.6	3.1	1.0	72.8	71.8
2.0	0.045	0.94	0.06	73.8	2.2	1.7	74.7	73.0
2.5	0.056	0.93	0.07	75.2	1.4	2.1	76.0	73.9
3.0	0.067	0.91	0.09	76.5	1.3	2.8	77.1	74.3
3.5	0.078	0.90	0.10	77.4	0.9	3.3	77.9	74.6
4.0	0.090	0.89	0.11	78.3	0.9	3.7	78.7	75.0
4.5	0.101	0.88	0.12	79.0	0.7	4.1	79.3	75.2
5.0	0.112	0.86	0.14	79.6	0.6	4.7	79.9	75.2

[1] Avg. temperature of concrete minus initial temperature of cooling water (first interval only).

Design Considerations

Length of Construction Block

For given site and loading conditions, the overall thickness of a dam is determined by gravity or arch analyses. Thickness, therefore, is a given condition at the time the length of construction block is considered. Where this thickness is large, a decision must be made whether to construct the section by means of one or more construction blocks separated by longitudinal joints, or to apply rigid temperature control requirements and construct it as a single block.

With modern construction equipment capable of continuous concrete placement around-the-clock, the length of a construction block is not normally related to the size or capacity of the construction plant. More generally, the length of construction block is governed by the stresses which develop within the block. These tensile stresses occur between the time the block is first placed and the time when it has cooled to its minimum temperature and are related to many factors. Factors subject to at least some degree of control include the overall temperature drop from the maximum concrete temperature to the closure temperature, the rate of temperature drop, and the age of the concrete when the concrete is subjected to the temperature change.

Beyond the control of the designer are other factors, such as the thermal coefficient of expansion, the effective modulus of elasticity,[5] the elastic and inelastic properties of the concrete, and the restraint factor. The actual stresses will further vary between quite wide limits because of conditions occurring during the construction period which introduce localized stress conditions. Tensile stresses and resulting cracks may occur because the larger blocks cover a greater area of the foundation and are thereby subjected to a greater number of stress concentrations due to the physical irregularities and variable composition of the

[5] The effective modulus of elasticity is often given as:

$$E_{eff} = \frac{1}{1+0.4\frac{E_c}{E_f}} E_c$$

where E_c is the modulus of elasticity of concrete and E_f is the modulus of elasticity of the foundation rock. By inspection, it is evident that lower concrete stresses will result on a foundation of low elasticity.

foundation. Also, cracks may occur because of delays in the construction schedule and construction operations.

Longer blocks are more likely to have cold joints occurring during placement of the concrete, and these cold joints form definite planes of weakness. The longer blocks may also be exposed for longer periods of time, allowing extreme temperature gradients to form near the surfaces. The stresses caused by these steep temperature gradients may then cause cracks to form along any planes of weakness which exist as a result of construction operations.

The size of block does not affect the stress in the normal equation for temperature stress, $S=Ee(T_2-T_1)$, where S is the unit stress, E is the effective modulus of elasticity, e is the thermal coefficient of expansion, and (T_2-T_1) is the temperature drop. This equation is valid only where there is a direct restraint against movement, such as would exist along the rock-concrete contact surface. The equation for temperature stress in a large concrete block is better expressed as $S=REe(T_2-T_1)$, where R is a restraint factor. The restraint factor is a function of the length of block and the height above the foundation.

The most commonly used values of the restraint factor are given in Figure 39.[6] Stresses obtained by the above are uniaxial stresses. Higher stresses than these can occur under biaxial or triaxial restraint conditions. Bringing into consideration these additional restraints, the maximum stresses can be estimated by multiplying the above results by $\frac{1}{1-\mu}$ for biaxial restraint and $\frac{1}{1-2\mu}$ for triaxial restraint. Values of Poisson's ratio, μ, for concrete vary with age and with different concretes, with most values ranging from 0.15 to 0.20 at age 28 days and from 0.16 to 0.27 at age 1 year.

Although subject to theoretical determination by means of the above stress relation, the variations in the values of E and e make such a determination of questionable value. For example, values of the modulus of elasticity of concrete at age 2 days have varied from 1.4×10^6 to 2.8×10^6, at age 7 days from 2.1×10^6 to 4.2×10^6, at age 28 days from 3.3×10^6 to 5.7×10^6, and at age 1 year from 3.3×10^6 to 6.8×10^6. The coefficient of thermal expansion, while not varying appreciably with age, has had values ranging from 4.3×10^{-6} to 5.7×10^{-6}. Because of these variations and because the other stress-producing factors are either unknown or susceptible to change during construction, it is difficult in the design stages to determine the actual stresses which will occur in a given size block.

The strength of the concrete and the creep properties of the concrete both vary with age and make it all the more difficult to determine whether or not the stresses will be acceptable at the time they are imposed upon the structure. For example, studies of the creep properties of concrete used in five dams showed that stresses varied to such an extent that, for a temperature drop of 1° per day starting at age 4 days, the theoretical temperature stresses were 12, 20, 65, and 75 percent higher in Flaming Gorge, Ross, Hungry Horse, and Glen Canyon Dams, respectively, than those in Monticello Dam. Similarly, for a temperature drop of 1° per day starting at age 180 days, the stresses were 2, 10, 33, and 75 percent higher, in the same order, as compared to Monticello Dam.

Lacking data to compute stresses, field experiences on other jobs should guide the designer in determining the degree of temperature control for a new structure. Such experiences are reflected in Table VII, which can be used as a guide during the early stages of design to determine the primary means of temperature control.

Width of Construction Block

Contraction joints are normally spaced about 50 feet apart, but may be controlled by the spacing and location of penstocks and river outlets, or by definite breaks and irregularities of the foundation. A uniform spacing is desirable so that the contraction joints will have uniform openings for contraction joint grouting. Spacings have varied in dams designed by the Bureau of Reclamation from 30 feet to 80 feet as measured along the axis of the dam. When the blocks are 30 feet or less in width, a problem arises in regard to obtaining a good groutable opening of the contraction joint. In these cases, it may become necessary to cause a larger temperature drop to take place than would otherwise be desirable. This temperature drop is

[6] Derived from test data reported in 1940 by R. W. Carlson and T. J. Reading to the Portland Cement Association. These data were plotted in the TVA Technical Monograph No. 67, "Measurements of the Structural Behavior of Norris and Hiwassee Dams."

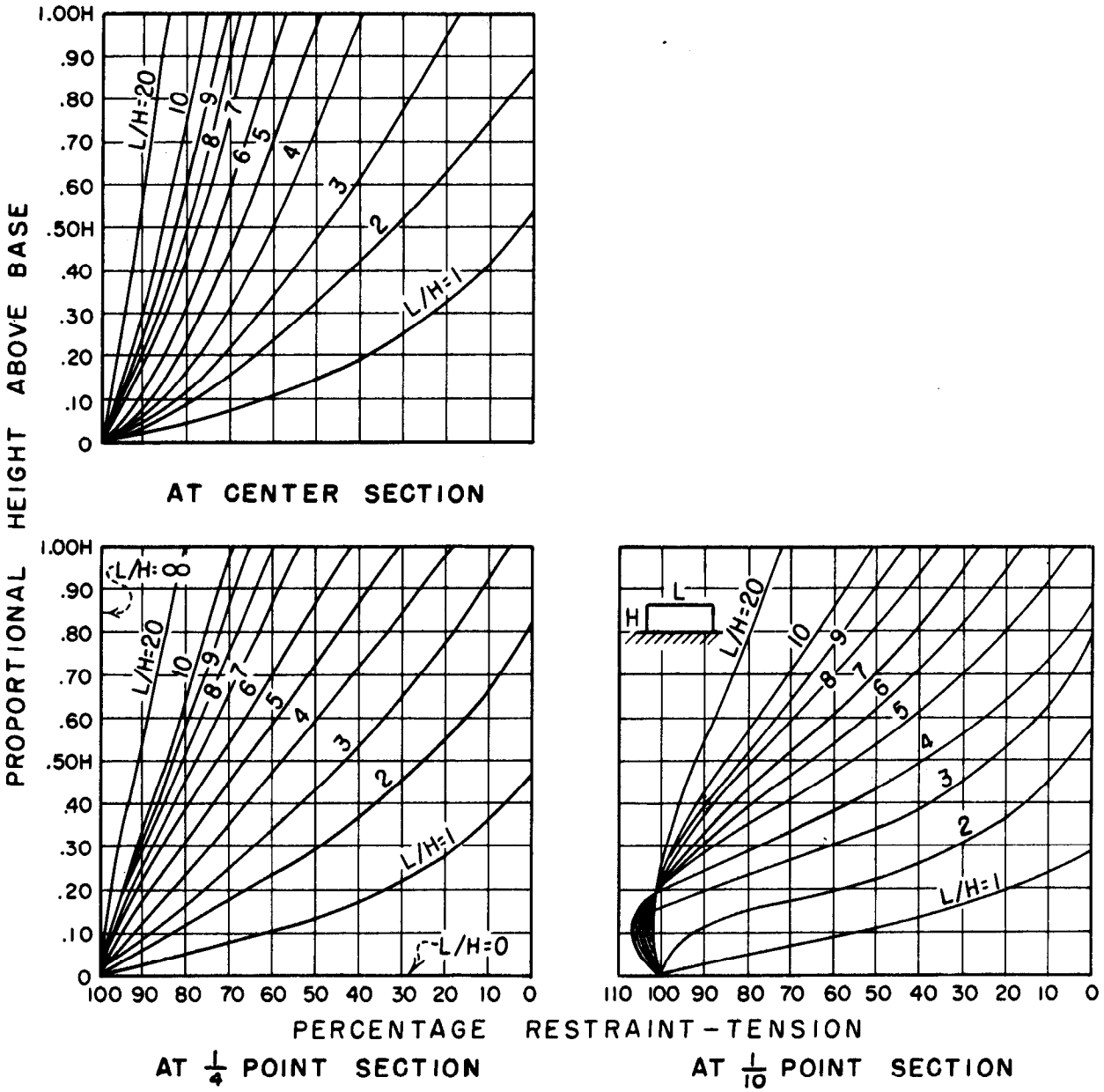


FIGURE 39.—*Foundation restraint factors.*

not without limit, however, it must be compatible with the drop allowed for the longer dimension of the block. A further consideration is the maximum length-to-width ratio of the block which will exist at the top of the dam. If the ratio of the longer dimension to the shorter dimension is much over 2.5, cracking at approximate third points of the block can be expected. Ratios of 2.0 to 1 or less are desirable, if practicable.

It is theoretically possible to compute the required temperature drop for a given size of construction block to obtain a desired joint opening. This theoretical temperature drop from maximum temperature to grouting temperature must be modified because some compression is built up in the block as the temperature increases during the first few days after placement. A 2° to 4° temperature drop from the maximum temperature,

TABLE VII.—*Temperature treatment versus block length*

Block length	Treatment		
Over 200 feet	Use longitudinal joint. Stagger longitudinal joints in adjoining blocks by minimum of 30 feet		
	Temperature drop from maximum concrete temperature to grouting temperature—°F.		
	Foundation to $H=0.2L$ [1]	$H=0.2L$ to $0.5L$ [1]	Over $H=0.5L$ [1]
150 to 200 feet	25	35	40
120 to 150 feet	30	40	45
90 to 120 feet	35	45	No restriction
60 to 90 feet	40	No restriction	No restriction
Up to 60 feet	45	No restriction	No restriction

[1] H=height above foundation; L=block length.

depending on the creep properties of the concrete, may be required to relieve this compression before any contraction joint opening will occur. It must be further modified with time, due to the weight of the concrete in the block and the creep properties of the concrete.

Measured joint openings in Hungry Horse Dam averaged 75 percent of the theoretical. Other experiences with arch dams having block widths of approximately 50 feet have indicated that a minimum temperature drop of 25° from the maximum temperature to the grouting temperature is desirable, and will result in groutable contraction joint openings of 0.06 to 0.10 inch. For the wider blocks with 70 feet or more between contraction joints, a 20° temperature drop will be sufficient.

For planning purposes, the maximum width of block permitted by plant capacity is given by

$$W=0.21\frac{Pt}{\sqrt{bd}(D+1)}$$

where

W=width of block in feet
P=plant capacity in cubic yards per day
t=maximum time (in hours) allowed between layers to prevent cold joints (1 to 3 hours normally, depending on mix)
b=bucket capacity in cubic yards
d=depth of layer in feet
D=number of layers placed to reach full height of lift.

For example, given a plant capacity of 4,600 cubic yards per day, bucket capacity of 8 cubic yards, depth of layer of 20 inches, time allowable between layers of 1 hour, and 5-foot lifts, the maximum width of block would be:

$$W=\frac{0.21(4.600)(1)}{\sqrt{(8)(1.67)}(3+1)}=66 \text{ feet } \pm.$$

Control of Temperature Drop

Control of the temperature drop from the maximum concrete temperature to the grouting temperature is the one factor relating to stress development over which a measure of control can be achieved. A reduction in the total temperature drop may be accomplished by controlling the temperature rise which occurs immediately after placement of concrete, by reducing the placing temperature of the concrete, or by both measures.

The temperature rise is normally controlled by the selection of the type and amount of cement used in the concrete, the replacement of part of the cement by a pozzolan, and the artificial cooling of the concrete through an embedded pipe system during the first few days after placement. Minor benefits may also accrue due to evaporative cooling when the concrete is water cured. In most instances, Type II cement will produce concrete temperatures which are acceptable. In the smaller structures, Type I cement will usually be satisfactory. Other factors being equal, Type II is usually selected because of its better resistance to sulfate attack, better workability, and lower permeability.

Type IV cement, although originally developed for use in mass concrete to reduce the temperature rise, is now used only when other methods of control will not accomplish the desired result. For example, it may be used near the base of long blocks where a high degree of restraint exists. Concrete with Type IV cement requires more curing than concrete with other types of cement, and extra care is required at early ages to prevent damage to the concrete from freezing or from too early a form removal.

Concrete technology has reached the point where the old standard of four or more sacks of cement per cubic yard for mass concrete is no longer the criterion. Present-day mix designs can usually obtain the required concrete strengths with about three sacks of cementing materials per cubic yard of concrete for the interior mass of the dam. The cementing materials may be entirely cement or may be a mixture of cement and a pozzolanic ma-

terial. As the temperature rise in concrete is directly related to the amount of cement and to the amount of pozzolan, and as the heat generation of the pozzolanic replacement is roughly 50 percent that of cement, the subsequent temperature rise in present-day concrete has been reduced by as much as 40 percent over that obtained in the past. Figure 40 shows typical temperature rise curves for concrete placed in Monticello and Hungry Horse Dams.

Maximum concrete temperatures can be lowered by reducing the placing temperature of the concrete. In addition to the direct reduction in temperature, this also results in a slower rate of hydration of the cement. When no special provisions are employed, concrete placing temperatures will be very close to the mean monthly air temperature, ranging from 4° to 6° higher than the mean air temperature in the winter time and this same amount lower than the mean air temperature in the summer time. The actual concrete mix temperature depends upon the batch weights, specific heats, and the temperatures of the individual materials going into the concrete mix.

Table VIII shows a sample computation for estimating the concrete placing temperature of an assumed mix. By dividing the total B.t.u.'s in the mix by the total B.t.u.'s required to vary the temperature of the material 1° F., the temperature of the concrete as mixed can be estimated. In the sample computation, this would be $\frac{89{,}060}{1{,}074}$ or 82.9° F. To estimate how much any one ingredient would have to be changed to lower the mix temperature 1° F., the total B.t.u.'s required to vary the temperature of the material 1° F. would be divided by the B.t.u.'s required for the individual mix ingredient. For example, the mix water to be added would have to be cooled $\frac{1{,}074}{140}$ or 7.67° F. to lower the concrete temperature 1° F.

Various methods have been used to reduce concrete placing temperatures, and the method or combination of methods will vary with the degree of cooling required and the construction contractor's previous experience. Cooling of the coarse aggregate has varied from shading and sprinkling the aggregate in the stockpiles to chilling the aggregate in large tanks where the aggregate is immersed in refrigerated water for a given period of time. Relatively complete cooling of coarse aggregate has also been obtained by forcing refrigerated air through the aggregate, while the aggregate is draining in a storage bin after being immersed, while it is draining on a conveyor belt after being sprayed with cold water, or while it is passing through the bins of the batching plant. The sand is usually cooled by passing it through vertical, tubular heat exchangers. Cold air jets in conjunction with conveyor belts have also been used. Immersion of sand in cold water has not been successful because of the difficulty in removing the free water from the sand after cooling.

Cooling of the cement is a definite problem. Bulk cement is almost always obtained at relatively high temperatures, and the best that can be hoped for, in some cases, is that it will cool naturally and lose a portion of the excess heat before it is used. In some instances, the vertical tubular heat exchangers may be used for cement cooling.

Mix water has been cooled to varying degrees, the more common temperatures being from 32° to 40° F. Adding slush or crushed ice to the mix is an effective method because it takes advantage of the latent heat of fusion of ice. The addition of large amounts of ice, however, may not be too practicable in some instances. First, the batching hopper must be designed in such a way as to prevent the ice from adhering to the sloping sides of the batcher. In part, this may be overcome by batching the ice with the sand. Second, if the coarse aggregate and sand both contain appreciable amounts of free water, the additional water to be added to the mix may be so small that replacement of part of the free water by ice would not be of appreciable value.

An embedded pipe cooling system has two primary functions: To reduce the maximum temperature of the concrete, and to reduce the temperature of the concrete to the required grouting temperature during the construction period. An embedded pipe cooling system embedded on the top of each 5- or 7½-foot lift of concrete cannot remove the heat as fast as it is generated in the concrete during the first 2 to 5 days, but can remove the heat faster than it is generated after that time. The maximum temperature of concrete cooled by an embedded pipe cooling system will, therefore, be reached during this period and subsequent temperatures can be maintained below this initial maximum temperature. Without an embedded pipe

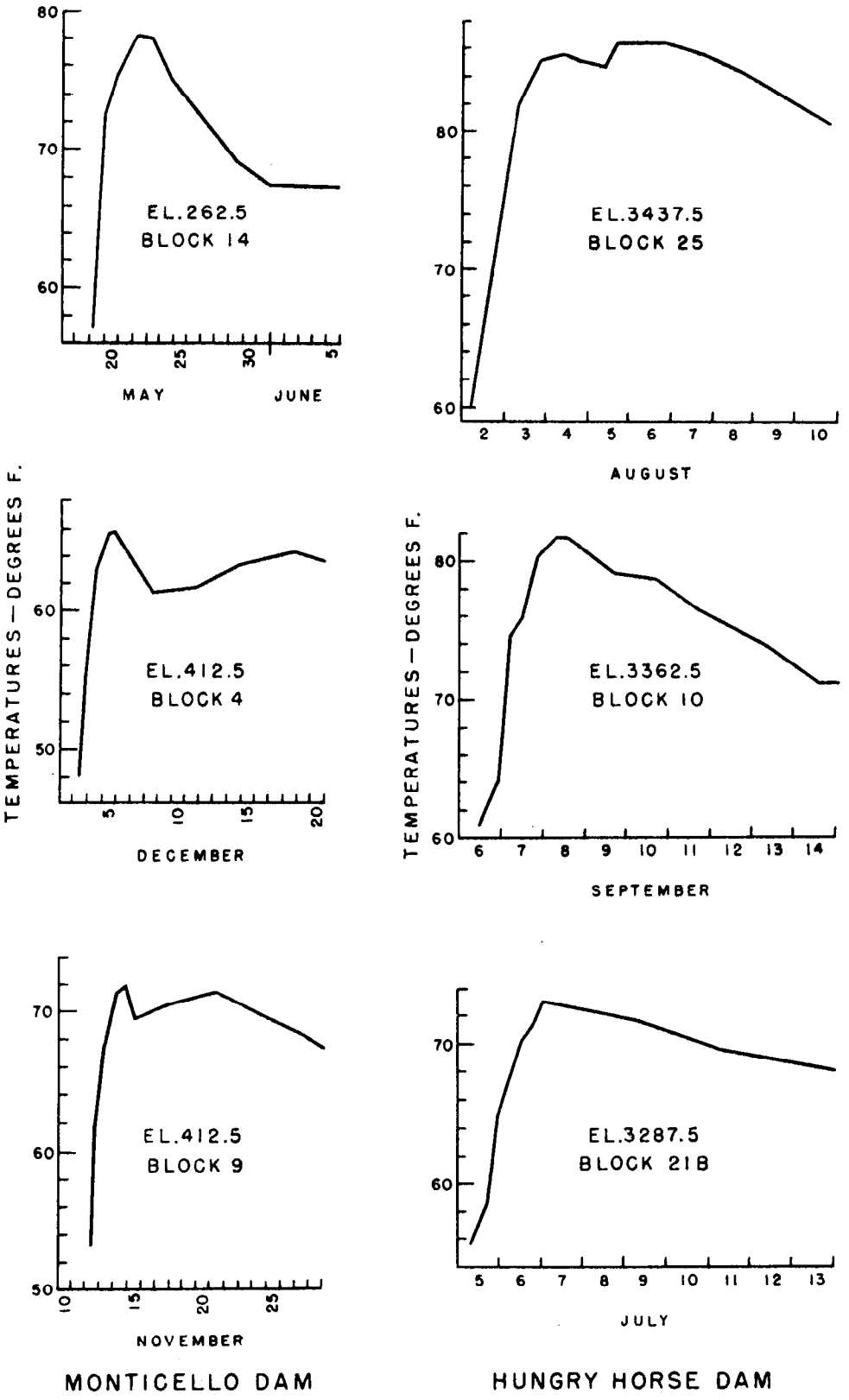


FIGURE 40.—*Typical temperature rise curves.*

TABLE VIII.—*Computation for temperature of concrete mix*

Material	Batch weight, pound	Specific heat	B.t.u. to vary temperature of material 1° F.	Initial temperature of material in °F.	Total B.t.u. in material
Water to be added	140	1. 0	140	60	8, 400
Cement	240	0. 23	55	150	8, 250
Sand	900	0. 22	198	85	16, 830
No. 4 to ¾ in. aggregate	650	0. 22	143	80	11, 440
¾ in. to 6 in. aggregate	2, 220	0. 22	488	80	39, 040
Free moisture:					
Sand	20	1. 0	20	85	1, 700
No. 4 to ¾ in	8	1. 0	8	80	640
¾ in. to 6 in	22	1. 0	22	80	1, 760
Mechanical heat [1]					1, 000
Total			1, 074		89, 060

[1] Estimated heat gain due to motors, friction, etc.

system, concrete temperatures in a 5-foot placement lift would reach a peak temperature at 3 to 5 days and would then drop slowly until that lift was covered by the next placement lift. Another temperature rise would then occur and, depending on the length of exposure period, a new and higher maximum temperature than before could be obtained. In 7½-foot lifts, the same pattern would occur except that the peak temperature would be reached in 5 to 8 days.

The typical temperature history of artificially cooled concrete is shown in Figure 30. The normal use of the embedded pipe cooling system consists of an initial cooling period from 10 to 15 days. During this initial cooling period, the concrete temperatures are reduced from the maximum concrete temperature to such a temperature that, upon stopping the flow of water through the cooling system, the continued heat of hydration of the cement will not result in maximum temperatures higher than that previously obtained. Subsequent to this initial cooling period, an intermediate and a final cooling period are utilized to lower the concrete temperature to the desired grouting temperature. Depending on the temperature drop and final temperature to be obtained, the season of the year when this cooling is accomplished, and the temperature of the cooling water, the intermediate and final cooling periods will require a total of from 30 to 60 days.

Rate of Cooling

Artificial cooling is accomplished by circulating cold water through 1-inch outside diameter pipe or tubing laid grid-like over the top surface of each 5- or 7½-foot lift of concrete. The rate of cooling is controlled so that the tensions set up in the concrete by the drop in temperature will not exceed the tensile strength of the concrete. Cooling of the concrete for 3 to 4 weeks at a rate of temperature drop of about 1° per day can create tensions which are equal to or exceed the tensile strength of the concrete. As shown in Figure 41, a temperature drop of about 2° per day can cause cracking in about 11 days. To prevent the artificial cooling from causing cracks, an initial cooling period of not more than 2 weeks is usually specified, and the cooling systems are so operated that the temperature drop is not more than 1° per day.

Figure 41 shows a typical increase in tensile strength of concrete with age, together with the tensions created in the concrete for different rates of temperature drop. These tensions vary with the creep properties of the concrete. After the initial cooling period is completed, a variable period of time elapses before the intermediate and final cooling periods are started during which the concrete gains tensile strength. Since the modulus of elasticity of the concrete at this time is higher, the rate of temperature drop should again be held

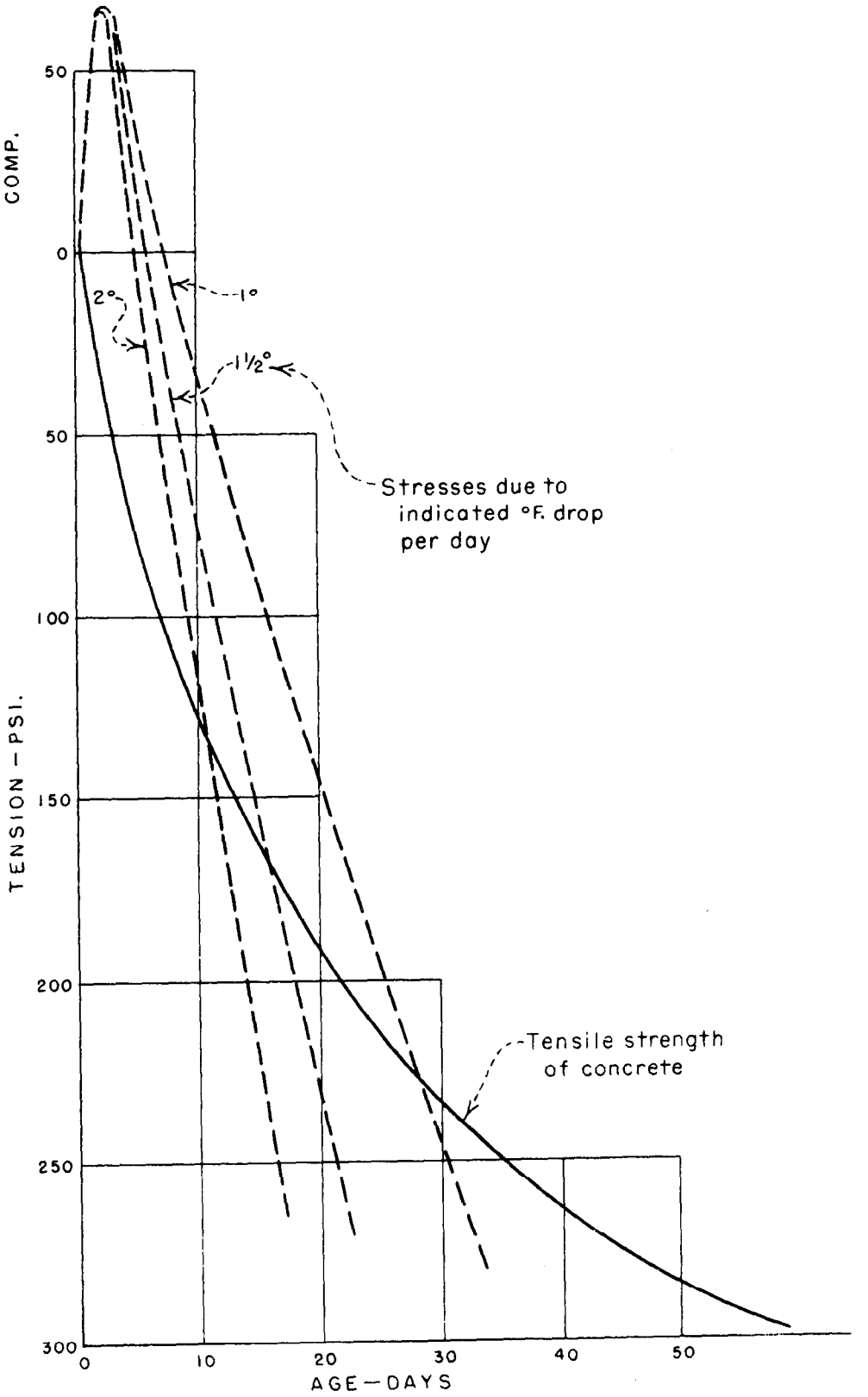


FIGURE 41.—*Tensions versus concrete strength—early age.*

to not more than 1° per day, and a rate of ½° to ¾° per day would be preferable.

The rate of temperature drop forced upon the concrete by an embedded pipe cooling system can be controlled by varying the length of coil, the horizontal spacing of the tubing, the velocity of the cooling water, and the temperature of the cooling water in the cooling systems. The length of coil and the horizontal spacing are determined by the size of the blocks and the purpose of the cooling systems; that is, whether the cooling systems are primarily for lowering the temperature of the concrete prior to contraction joint grouting, for controlling the maximum concrete temperature, for reducing temperature gradients near exposed surfaces, or any combination of these.

The effects of changes during the construction period such as the type or amount of cement used in the concrete, curing methods employed, exposure temperatures varying from those assumed, or any other factors which influence concrete temperatures, are normally taken care of by varying the periods of flow and the rate of flow of the cooling water. If these changes become major changes and vary greatly from those assumed, the temperature effects can be taken care of by varying the horizontal spacing of the tubing and/or length of cooling coil in those portions of the dam then being constructed.

Joints

A construction joint in concrete is defined as a concrete surface, upon or against which new concrete is to be placed and to which the new concrete is to adhere, that has become so rigid that new concrete cannot be made monolithic by vibration with that previously placed. Construction joints are placed in concrete structures to facilitate construction, to reduce initial shrinkage stresses and cracks, to permit installation of embedded metalwork, or to allow for the subsequent placing of other concrete, backfill concrete, or second-stage concrete. Construction joints also occur as a result of inadvertent delays in concrete placing operations. Construction joints are, therefore, construction expedients required to obtain the desired concrete structure.

Expansion and contraction joints are provided in concrete structures to accommodate volumetric changes which occur in the structure after placement. Expansion joints are provided in a unit-structure to allow for the expansion of the unit in such a manner as not to change the stresses in, or the position of, an adjacent unit or structure. Contraction joints are provided in a structure to prevent the formation of tensile cracks in the structure as the structure contracts. In this respect, the contraction joint may be defined as a designed crack. Contraction joints should be so constructed that no bond exists between the concretes separated by the joint. Except for dowels, reinforcement should not extend across a contraction joint.

The location of contraction joints is largely a matter of past experience. Accurate analyses are difficult because of the many variables, the largest of which in mass concrete are the degree of restraint and the temperature distributions and variations. In relatively thin walls and slabs, drying shrinkage also is of concern since its effect could be equivalent to that of a 60° to 100° temperature drop. Any spacing of joints should, therefore, be based upon experimental data and systematic observation. Spacing of joints in mass concrete structures is given in Table VII. Knowing where cracks are likely to occur, the designer and the constructor can alleviate many of the problems arising from random cracking.

Temperature Reinforcement—General

If a concrete structure is free to expand or contract with variations in temperature, no stresses of any significance will be developed. This is true even in reinforced concrete, as the coefficients of thermal expansion of steel and concrete are essentially the same. When, however, the movement of any part of the structure is restrained, either by external restraints or by the restraint exerted by one part of the structure upon an adjacent part, a drop in temperature will cause tensile stresses. Cracks caused by these tensile stresses not only will be unsightly, but could also serve to accelerate the deterioration of the structure to perform its purpose.

Temperature reinforcement does not prevent cracking, but it can distribute cracks and control their width so that the usefulness of the structure will not be seriously affected. Since crack widths and spacings tend to be greater for widely spaced reinforcement bars, smaller bars at closer spacing

should be used for temperature reinforcement in preference to large bars at wide spacing. Reinforcement bars should be placed as close as practicable to the surface. Steel having a high elastic limit is also advantageous.

Temperature Reinforcement—Frames Attached to Mass Concrete

Temperature reinforcement in frame- and box-type structures attached to mass concrete is required to resist the moments and shears induced when temperature expansion or contraction of the structure is prevented by the adjoining mass concrete. In small, flexible members, this moment may be of little concern. In thick, short members, however, the induced moment may be the governing factor for the amount of steel required. The temperature drop or rise in such structures may be computed by the following:

Reservoir full.—The temperature rise or drop to be used for determining temperature reinforcement is one-quarter of the range of reservoir temperature to be experienced at the elevation under consideration. This takes into consideration that the restraint factor of the interior of the mass on the exterior surface is 50 percent and that the rise or drop in temperature is one-half of the mean annual range in temperature at the elevation. Since reservoir temperatures vary with depth, different temperature conditions may occur in those structures which extend vertically for some distance. This temperature loading is applied to the structure with waterload.

Reservoir empty.—The range of temperatures being much greater in air than in water, temperature conditions existing when there is no waterload on the structure may require more reinforcement than the condition with waterload. To determine the design temperature rise and drop when air surrounds the structure, the procedure is as follows:

1. Determine the mean monthly air temperatures and the mean annual air temperature.
2. Determine the difference, R, between the mean annual and the highest mean monthly, and the difference, D, between the mean annual and the lowest mean monthly.
3. Determine the average daily variation, r, in the warmest month, and the average daily variation, d, in the coldest month.
4. For a known or assumed h^2 (diffusivity constant), use Figure 11 to determine the ratio of the variation of the mean temperature in the slab or beam under consideration to the variation of the daily external air temperature. This assumes a sinusoidal variation in temperature which takes place on both sides of the slab or beam. If the beam has a width which is of the same order of magnitude as its thickness exposure of the four sides of the beam should be taken into consideration. (For this condition, the product rule for heat loss should be used, using values of [1–Ratio] from Figure 11. After obtaining total heat loss, the computation should revert back to mean temperature.) For a closed-in box-type structure, a Schmidt's Method computation may be required to take into consideration the internal exposure temperatures.
5. Multiply the daily variations, r and d, by the ratio found in the above step to obtain d_1 and r_1. If Schmidt's Method is used, r_1 and d_1 are the ranges of mean concrete temperature occurring in the concrete structure as it undergoes the daily variations in temperature in the warmest and the coldest months.
6. The temperature rise and drop used for design is then:

$$\text{Temperature rise} = \frac{R + r_1}{2}$$

$$\text{Temperature drop} = \frac{D + d_1}{2}$$

These equations take into consideration the 50-percent restraint factor the interior of the mass exerts on the external surface, and assume that one-half of the daily variation occurs above and below the daily mean. Average air temperatures obtained from the above daily and mean monthly air temperatures are used in most cases. Extreme air temperatures can be assumed for important structures by increasing the daily variations by such an amount as to account for recorded extreme temperatures.

Construction Requirements

Surface Treatments

SINCE the large majority of all cracks start at an exposed surface, several provisions for surface treatment can be utilized which will reduce the cracking tendencies at such surfaces. These include surface cooling, time of form removal, and water curing.

If the concrete near the surface of a mass concrete structure can be made to set at a relatively low temperature and can be maintained at this temperature during the early age of the concrete, say, for the first 2 weeks, cracking at the surface can be minimized. Under this condition, tensions at the surface are reduced, or the surface is put into compression, when the interior mass of concrete subsequently drops in temperature. The use of noninsulating steel form which are kept cool by continuous water sprays will tend to accomplish this desirable condition. If, however, water sprays are not used to modify the temperature of the steel forms, the early-age temperature history of the face concrete will be even greater than the daily cycle of air temperature and little, if any, stress benefit will be obtained at the surface. With wood forms which prevent heat from being lost readily to the surface, the temperature near the surface will be close to that obtained in the interior of the mass, and no stress benefit will occur at the surface.

The time of form removal from mass concrete structures is important in reducing the tendency to crack at the surface when wood forms or insulated steel forms are used. If exposure temperatures are low and if the forms are left in place for several days, the temperature of the concrete adjacent to the form will be relatively high when the forms are stripped, and the concrete will be subjected to a thermal shock which will cause cracking. From the temperature standpoint, forms should either be removed as early as practicable or should remain in place until the temperature of the mass has stabilized. In the latter case, a uniform temperature gradient would have been established between the interior mass and the surface of the concrete, and removal of the forms,

except in adverse exposure conditions, would have no harmful results.

Following the removal of forms, proper curing is important if drying shrinkage and resulting surface cracks are to be avoided. Curing compounds which prevent the loss of moisture to the air are effective in this respect, but lack the cooling benefit which can be obtained by water curing. In effect, water curing obtains a surface exposure condition more beneficial than the fluctuating daily air temperature. With water curing, the daily exposure cycle is dampened because the daily variation in the water is less than that of the daily air temperature. A benefit also occurs from the evaporative cooling effect of the water on the surface. The actual effective exposure temperature is difficult to compute because it varies considerably with the rate and amount at which the water is applied to the surface. Intermittent sprays which maintain the surface of the concrete in a wet to damp condition with some free water always available is the most desirable condition.

Surface Gradients

During periods of cold weather, steep temperature gradients between the relatively warm interior of a mass and its surface will cause high tensile stresses to form at the surface. The surface treatments previously described can reduce these temperature gradients, particularly when used in conjunction with artificial cooling, but the use of insulation or insulating-forms will give greater protection. Although steel forms are beneficial from the standpoint of avoiding thermal shock upon the removal of forms, there are times when wooden forms would be preferable. During the fall of the year when placing temperatures are still relatively high, the tendency of the surface of the concrete is to drop rapidly to the exposing temperature. This may occur while the interior concrete is still rising in temperature. In this instance, wooden forms would prevent the rapid drop at the surface and, combined with artificial cooling, would lower the temperature of the mass in a uniform manner.

If extreme exposure temperatures exist soon after placement of the concrete, insulation of the faces may be employed to prevent steep gradients near the exposed surfaces. Such insulation may be obtained by measures varying from simply leaving wood or insulated forms in place to the use of commercial-type insulation applied to the forms or to the surfaces of the exposed concrete. Tops of blocks can be protected with sand or sawdust when an extended exposure period is anticipated.

Whatever the type of insulation, measures should be taken to exclude as much moisture from the insulation as practicable. The insulation should also be as airtight as possible. For a short period of exposure, small space heaters may be used, either by themselves or in conjunction with canvas coverings which enclose the work. Care should be taken, however, when using space heaters in enclosed areas to avoid drying out of the concrete surfaces.

Stresses due to temperature gradients may be of concern not only during the construction period discussed above but during the life of the structure. Stresses across a section due to temperature gradients can be obtained from the expression

$$\sigma_y=\frac{eE}{b^3(1-\mu)}\left[b^2\int_0^b T(x)dx+3(2x-b)\int_0^b (2x-b)T(x)dx-b^3T(x)\right]$$

where e is the thermal coefficient of expansion, E is modulus of elasticity, μ is Poisson's ratio, and b is the thickness of section with a temperature distribution, $T(x)$. Where the temperature variation, $T(x)$, cannot be expressed analytically, the indicated integrations can be performed numerically by the use of Simpson's Rule. For example, using $b=30$ feet, $e=6.0\times10^{-6}$, $E=2{,}500{,}000$ pounds per square inch, $\mu=0.20$, and an assumed $T(x)$ as shown in Table IX, the stresses would be computed as follows:

TABLE IX.—*Computation of temperature stress*

x ft	$T(x)$ °F.	$(2x-30)T(x)$ ft −°F.		σ_y lb/ft²	σ_y lb/in²
0	0.0	0	For the given conditions:	10,584	74
3	8.3	−199		4,125	29
6	15.8	−284	$\frac{eE}{b^3(1-\mu)}=0.1$	−174	−1
9	22.7	−272		−2,853	−20
12	29.1	−175	$\sigma_y=0.1[(900)(1003.8)$	−4,182	−29
15	35.1	0	$+3(2x-30)(8862)$	−4,431	−31
18	40.7	244	$-(30)^3T(x)]$	−3,600	−25
21	46.0	552		−1,959	−14
24	50.9	916		762	5
27	55.6	1334	Simplifying:	4,023	28
30	60.0	1800	$\sigma_y=5317x-2700T(x)$	8,094	56
			$+10,584$		
$\int_0^b$	1003.8	8862			

Similar stress computations made for sudden temperature drops at the surface of a mass concrete structure show that tensile stresses as high as 16 to 18 pounds per square inch per degree temperature drop can be expected to occur. As can also be found from the stress relationship in Table IX, any linear distribution of temperature across a section will result in no temperature stress in the section.

Foundation Irregularities

Although the trial-load analyses assume, and the design drawings show, relatively uniform foundation and abutment excavations, the final excavation may vary widely from that assumed. Faults or crush zones are often uncovered during excavation, and the excavation of the unsound rock leaves definite depressions or holes which must be filled with concrete. Unless this backfill concrete has undergone most of its volumetric shrinkage at the time overlying concrete is placed, cracks can occur in the overlying concrete near the boundaries of the backfill concrete as loss of support occurs within the area of backfill concrete. Where these areas are extensive, the backfill concrete should be placed and cooled before additional concrete is placed over the area. Similar conditions exist where the foundation undergoes abrupt changes in slope. At the break of the slope, cracks often occur because of the differential movement which takes place between concrete held in place by rock and concrete held in place by previously placed concrete which has not undergone its full volumetric shrinkage.

A forced cooling of the concrete adjacent to and below the break in slope, and a delay in placement of concrete over the break in slope, can be employed to minimize cracking of the concrete at these locations. Although uneconomical to remove sound rock at abrupt foundation irregularities and replace it with concrete, the elimination of these points of high stress concentration is worthwhile. Such cracks in lifts near the abutments very often are the source of leaks which lead to spalling and deterioration of the concrete.

Relaxation of Initial Cooling

In the early spring and late fall months when exposure conditions are severe, the length of the initial cooling period and the rate of temperature drop can be critical in thin concrete sections where pipe cooling, combined with low exposure temperatures, can cause the concrete temperature to drop too fast. During these periods, artificial cooling should be stopped 4 to 5 days after placement and the concrete should be allowed to cool in a natural manner. In structures with thicker sections, the exposure temperatures have less effect on the immediate temperature drop, and the initial cooling period can be continued with the primary purpose of controlling the temperature difference between the exposed faces and the interior. A Schmidt's

Method of computation, using estimated exposure conditions, will usually suffice to determine the conditions existing for intermediate sections.

Final Cooling

Cooling and contraction joint grouting of the mass concrete in concrete dams is performed by grout lifts starting in the lowest part of the dam and progressing upward. These grout lifts are normally 50 to 60 feet in height and extend from abutment to abutment across the full length of the dam. Each contraction joint in each grout lift has its own vertical and horizontal seals so that the grout lift can be grouted as a unit. Cooling is completed in each grout lift just prior to grouting the contraction joints, the program of cooling being dictated by construction progress, method of cooling, season of the year, and any reservoir filling criteria.

As indicated in Figure 21, cooling prior to grouting the contraction joints is normally accomplished from 1 or 2 months to about 1 year after the concrete is placed. In the smaller construction blocks, final cooling may be accomplished in a single continuous cooling period. In the larger blocks, however, the final cooling should be performed in two steps to reduce the vertical temperature gradient which occurs at the boundary of a grout lift due to the grout lift pattern of cooling. The first of these steps is commonly referred to as the intermediate cooling period and the second step then becomes the final cooling period.

In practice, the intermediate cooling period for a grout lift lowers the temperature of the concrete in that lift to approximately halfway between the temperature existing at the start of the cooling period and the desired final temperature. Each grout lift, in succession, undergoes this intermediate cooling period before proceeding with the final cooling of the next lower grout lift. Such a pattern of cooling, once underway, will require little, if any, additional cooling.

Height Differential

For several reasons, a maximum height differential between adjacent blocks is specified in construction specifications for an arch dam. First, from a temperature standpoint, an even temperature distribution throughout the structure will be obtained when all blocks in the dam are placed in conformance with a uniform and continuous placement program. This even temperature distribution is desirable, not only because of the subsequent uniform system of contraction joint openings, but also because the overall stress distribution will be nearer that assumed in the design. Extreme temperature gradients on the exposed sides of blocks can also cause the start of circumferential cracks across the blocks. The chances of such cracks forming will be lessened when each lift is exposed for a minimum length of time.

Secondly, a uniform placement will result in a more uniform loading of the foundation. During that period of construction when the contraction joints are open, the individual blocks react on the foundation as individual columns. If extreme height differentials exist between blocks, unequal settlements in the foundation may result. A third reason for the height differential is that it will cause construction of the dam to progress uniformly up from the bottom of the canyon. Contraction joints can then be grouted in advance of a rising reservoir, thus permitting storage at earlier times than would occur if construction progress was concentrated in selected sections of the dam. An additional maximum height differential, that occurring between the highest and lowest blocks in the dam, should be required if the early storage is desirable.

Where one or two blocks are left low for diversion or as a construction expedient, several adverse conditions may occur as construction progresses. One of these conditions occurs when a large height differential exists during a long period of cold weather. Under such a situation, a definition or tilting of the high block out into the area of the low block occurs because of the temperature differential between the cold face adjacent to the low block and the opposite face which remains relatively warm. This deflection could damage the metal seals installed for construction joint grouting on the opposite face of the block if the deflection becomes too great. The deflection could also prevent continued placement of concrete in the block adjoining the opposite face of the high block. Continued placement in that block, while the high block is deflected out into the opening of the low block, would progressively wedge the high

block out. During the following warm season, when the cold exposed face warms up, the high block would tend to return to its original position, causing a shearing movement upward with the possibility of horizonal cracks being formed in the adjoining block. These problems peculiar to a considerable height differential can be met and corrected by the timely use of the embedded pipe cooling systems.

The actual height differential is a compromise between arriving at the uniform temperature conditions and construction progress desired, and the contractor's placement program. In Bureau of Reclamation practice, the maximum differential between adjacent blocks is usually stipulated as 25 feet where 5-foot lifts are used or 30 feet where 7½-foot lifts are used. The maximum differential between the highest block in the dam and the lowest block in the dam is usually limited to 40 feet where 5-foot lifts are used and to 52.5 feet where 7½-foot lifts are used.

Openings in Dam

Where possible, galleries and large openings in the dam should be located in regions of low stress or where cracks would not be detrimental to structural stability. Because openings concentrate stresses at their edges, all possible means should be used to prevent the beginning of cracks from the surfaces of such openings. Proper curing methods should be used at all times. The entrances to such openings should be bulkheaded and kept closed, with self-closing doors where traffic demands, to prevent the circulation of air currents through the openings. Such air currents not only tend to dry out the surfaces but also can cause the formation of extreme temperature gradients.

Extended Exposure of Horizontal Construction Joints

Whenever irregular placement occurs and tops of construction blocks are left exposed for more than 2 weeks, several crack-producing factors are at work which can start cracks or break bond between lifts. The edges of horizontal construction joints occurring at the tops of lifts left exposed over a winter's season are particularly vulnerable to surface tensions during the annual temperature cycles which occur during and after the construction period. These construction joints are cold joints and require special treatment to obtain an effective bond between the old concrete and the new concrete. Horizontal cracks often occur at the location of these cold joints during subsequent winter exposures, surface tensions causing the crack to extend into the concrete along the construction joint plane as much as 10 or 12 feet.

Preventive measures used are those directed toward obtaining the best possible bond between the old and new concrete. In the early spring, in addition to preparing the surface for placement of fresh concrete, considerable benefit is obtained by circulating warm water through embedded tubing in the top 2 or 3 placement lifts of the previously placed concrete before new concrete is placed. In conjunction with warming the old concrete, several shallow lifts may be placed initially on the old concrete or the horizontal spacing of the cooling coils in the first 2 or 3 lifts of new concrete may be reduced. Both of these measures will hold down the temperature rise in the new concrete and, combined with the lower placing temperatures normally occurring in the early spring months, will create a temperature differential in the concrete which will reduce the tendency to shear or break bond on the horizontal joint plane.

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