CHAPTER 8

TEMPERATURE STUDIES

8-1. Introduction. Temperature studies for arch dams fall into two distinct categories. The first category is the operational temperature study which is used to determine the temperature loading in the dam. This study is performed early in the design process. The second category includes the construction temperature studies which are usually performed after an acceptable layout has been obtained. The construction temperature studies are needed to assure that the design closure temperature can be obtained while minimizing the possibility of thermally induced cracking. The details of each of these studies are discussed in this chapter. Guidance is given on when the studies should be started, values that can be assumed prior to completion of the studies, how to perform the studies, and what information is required to do the studies.

8-2. Operational Temperature Studies.

a. General. The operational temperature studies are studies that are performed to determine the temperature distributions that the dam will experience during its expected life time. The shape of the temperature distribution through the thickness of the dam is, for the most part, controlled by the thickness of the structure. Dams with relatively thin sections will tend to experience temperature distributions that approach a straight line from the reservoir temperature on the upstream face to the air temperature on the downstream face as shown in Figure 8-1. Dams with a relatively thick section will experience a somewhat different temperature distribution. The temperatures in the center of a thick section will not respond as quickly to changes as temperatures at the faces. The temperatures in the center of the section will remain at or about the closure temperature,[1] with fluctuations of small amplitude caused by varying environmental conditions. The concrete in close proximity to the faces will respond quickly to the air and water temperature changes. Therefore, temperature distributions will result that are similar to those shown in Figure 8-2.

(1) Before describing how these distributions can be obtained for analysis, a description of how the temperatures are applied in the various analysis tools is appropriate. During the early design stages, when a dam layout is being determined, the trial load method is used. The computerized version of the trial load method which is widely used for the layout of the dam is the program ADSAS. ADSAS allows for temperatures to be applied in two ways. The first represents a uniform change in temperature from the grout temperature. The second is a linear temperature load. This linear load can be used to describe a straight line change in temperature from the upstream to downstream faces. These two methods can be used in combination to apply changes in temperature from the grout temperature as well as temperature

[1] The terms grout temperature and closure temperature are often used interchangeably. They represent the concrete temperature condition at which no temperature stress exists. This is also referred to as the stress-free temperature condition.

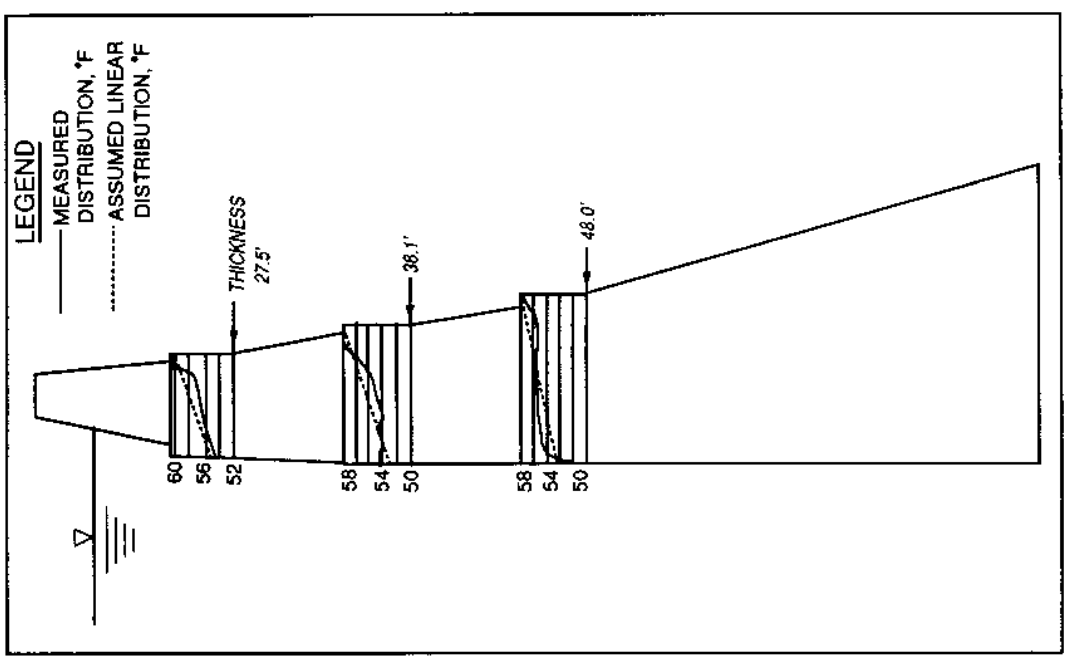

Figure 8-1. Measured temperature distributions in March for a relatively thin dam

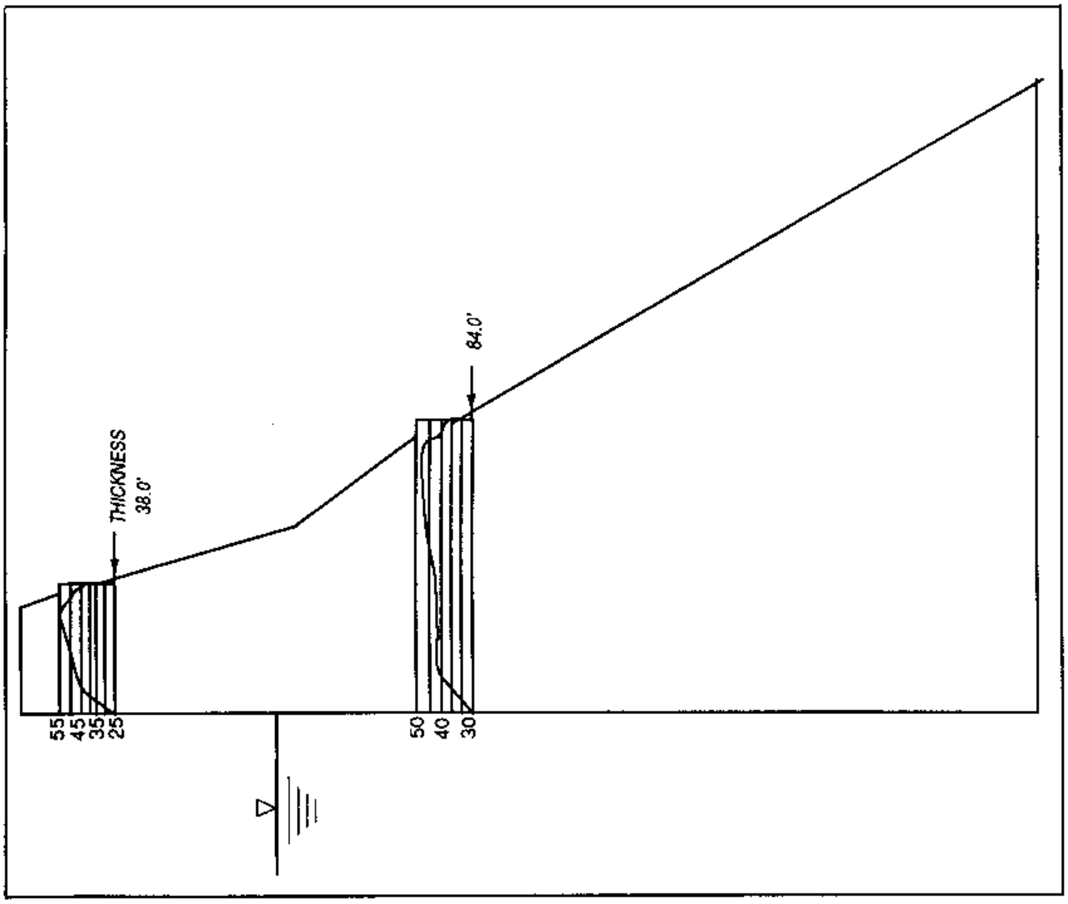

Figure 8-2. Measured temperature distributions in December for a relatively thick dam

differences through the section. The resulting distribution will be a straight line distribution.

(2) During later stages of analysis, usually after the final shape of the dam has been determined, the FEM is used to analyze both the static and dynamic conditions. In most general-purpose finite element programs, temperatures are applied at nodal points. This allows for the application of temperature distributions other than linear if nodes are provided through the thickness of the dam as well as at the faces.

(3) Keeping in mind the method of stress analysis to be used, one can now choose the method of determining the temperature distributions. There are two methods available for determining the distributions. The first method involves determining the range of mean concrete temperatures that a slab of concrete will experience if it is exposed to varying temperatures on its two faces. This method can be performed in a relatively short time frame and is especially applicable when the trial load method is being used and when the dam being analyzed is relatively thin. When the dam being analyzed is a thick structure, the FEM can be used to determine the temperature distributions.

(4) The temperature distributions are controlled by material properties and various site specific conditions, including air temperatures, reservoir water temperatures, solar radiation, and in some instances, foundation temperatures. The remainder of this section will discuss how the site conditions can be estimated for a new site and how these conditions are applied to the various computational techniques to determine temperature distributions to be used in stress analysis of the dam.

b. Reservoir Temperature. The temperature of a dam will be greatly influenced by the temperature of the impounded water. In all reservoirs the temperature of the water varies with depth and with the seasons of the year. It is reasonable to assume that the temperature of the water will have only an annual variation, i.e., to neglect daily variations. The amount of this variation is dependent on the depth of reservoir and on the reservoir operation. The key characteristics of the reservoir operation are inflow-outflow rates and the storage capacity of the reservoir.

(1) When a structure is being designed there is obviously no data available on the resulting reservoir. The best source of this data would be nearby reservoirs. Criteria for judging applicability of these reservoirs to the site in question should include elevation, latitude, air temperatures, river temperatures and reservoir exchange rate.[1] The USBR has compiled this type of information as well as reservoir temperature distributions for various reservoirs and has reported the data in its Engineering Monograph No. 34 (Townsend 1965). Figure 8-3 has been reproduced from that publication.

(2) If data are available on river flows and the temperature of the river water, the principle of heat continuity can be used to obtain estimates heat transfer across the reservoir surface. Determination of this heat transfer requires estimates of evaporation, conduction, absorption, and

[1] The reservoir exchange rate is measured as the ratio of the mean annual river discharge to the reservoir capacity.

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°-59°	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

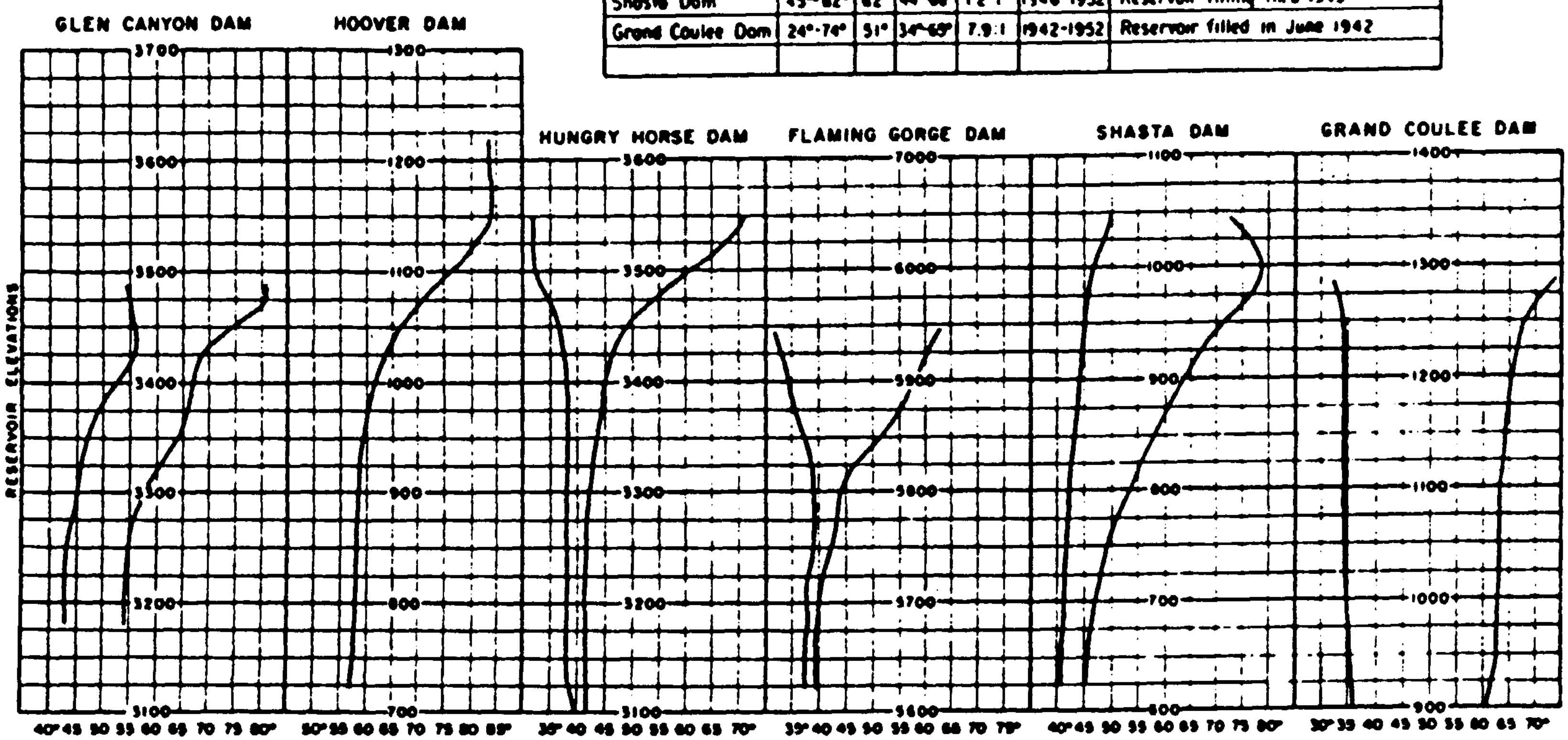

Figure 8-3. Typical reservoir temperature distributions (USBR)

reflection of solar radiation and reradiation, which are based on estimates of cloud cover, air temperatures, wind, and relative humidity. Since so many parameters need to be assumed, this method may be no better than using available reservoir data and adapting it to the new site.

(3) The designer should recognize that the dam's temperatures will be influenced significantly by reservoir temperatures. Therefore, as additional data become available, the assumptions made during design should be reevaluated. Also, it is good practice to provide instrumentation in the completed structure to verify all design assumptions.

c. Air Temperatures. Estimates of the air temperatures at a dam site will usually be made based on the data at nearby weather stations. The U.S. Weather Bureau has published data for many locations in the United States, compiled by state. Adjustments of the data from the nearest recording stations to the dam site can be used to estimate the temperatures at the site. For every 250 feet of elevation increase there is about a 1 ^{o}F decrease in temperature. To account for a positive 1.4-degree latitude change, the temperatures can be reduced by 1 ^{o}F. As with the reservoir temperatures, it is prudent to begin compiling air temperature data as early in the design process as possible to verify the assumed temperatures.

(1) During the discussion of reservoir temperatures, it was pointed out that daily water temperature fluctuations were not of significant concern; however, daily air temperature fluctuations will have a significant effect on the concrete temperatures. Therefore, estimates of the mean daily and mean annual air cycles are needed. A third temperature cycle is also used to account for the maximum and minimum air temperatures at the site. This cycle has a period of 15 days. During the computation of the concrete temperatures, these cycles are applied as sinusoidal variations. The air cycles are not truly sinusoidal, however, this assumption is an acceptable approximation. The pertinent data from the weather station required for the analyses are:

(a) Mean monthly temperatures (maximum, minimum, and average temperatures)

(b) Mean annual temperature

(c) Highest recorded temperature

(d) Lowest recorded temperature

(2) Paragraph 8-2e describes how these cycles are calculated and applied in the computations for concrete temperatures.

d. Solar Radiation. The effect of solar radiation on the exposed surfaces of a dam is to raise the temperature of the structure. Most concrete arch dams are subjected to their most severe loading in the winter. Therefore, the effect of solar radiation generally is to reduce the design loads. However, for cases where the high or summer temperature condition governs the design, the effect of solar radiation worsens the design loads. Also, in harsh climates where the dam is oriented in an advantageous direction, the effect of solar radiation on the low temperature conditions may be significant enough to reduce the temperature loads to an acceptable level.

(1) The mean concrete temperature requires 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 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. Figures 8-4 to 8-7 summarize the results 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 8-8.

(2) If a straight gravity dam is being considered, the orientation 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 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.

(3) The curves shown in Figures 8-4 to 8-7 are based on data obtained by the USBR. The data are based on the weather patterns and the latitudes of the western portion of the United States. A USBR memorandum entitled "The Average Temperature Rise of the Surface of a Concrete Dam Due to Solar Radiations," by W. A. Trimble (1954), describes the mathematics and the measured data which were used to determine the curves. Unfortunately, the amount of time required to gather data for such studies is significant. Therefore, if an arch dam is to be built in an area where the available data is not applicable and solar radiation is expected to be important, it is necessary to recognize this early in the design process and begin gathering the necessary data as soon as possible.

e. Procedure. This section will provide a description of the procedures used to determine the concrete temperature loads.

(1) The first method involves the calculation of the range of mean concrete temperatures. This method will result in the mean concrete temperatures that a flat slab will experience if exposed to: a) air on both faces or b) water on both faces. These two temperature calculations are then averaged to determine the range of mean concrete temperatures if the slab is exposed to water on the upstream face and air on the downstream face. A detailed description and example of this calculation is available in the USBR Engineering Monograph No. 34 (Townsend 1965). This process has been automated and is available in the program TEMPER through the Engineering Computer Program

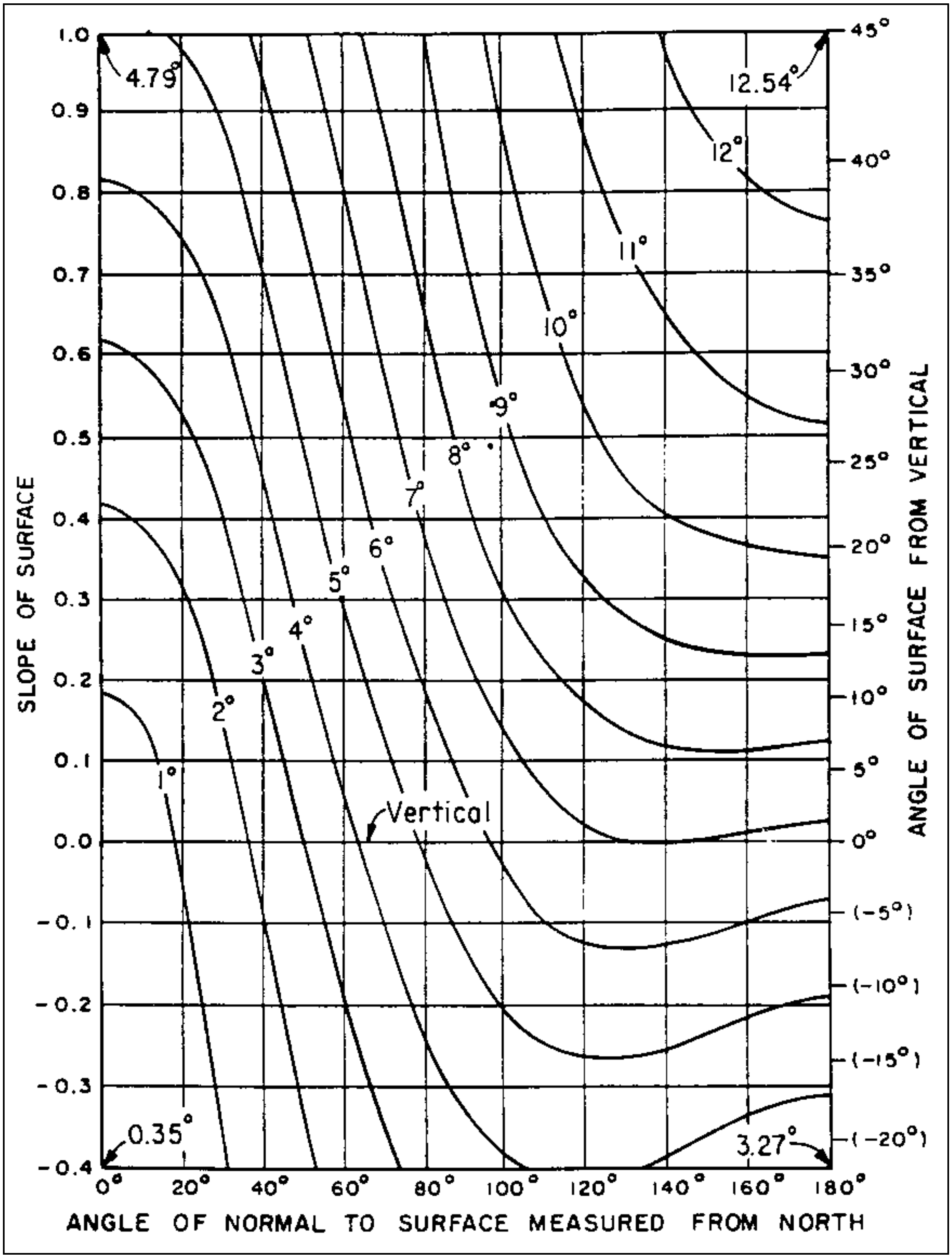

Figure 8-4. Increase in temperature due to solar radiation, latitudes 30° - 35° (USBR)

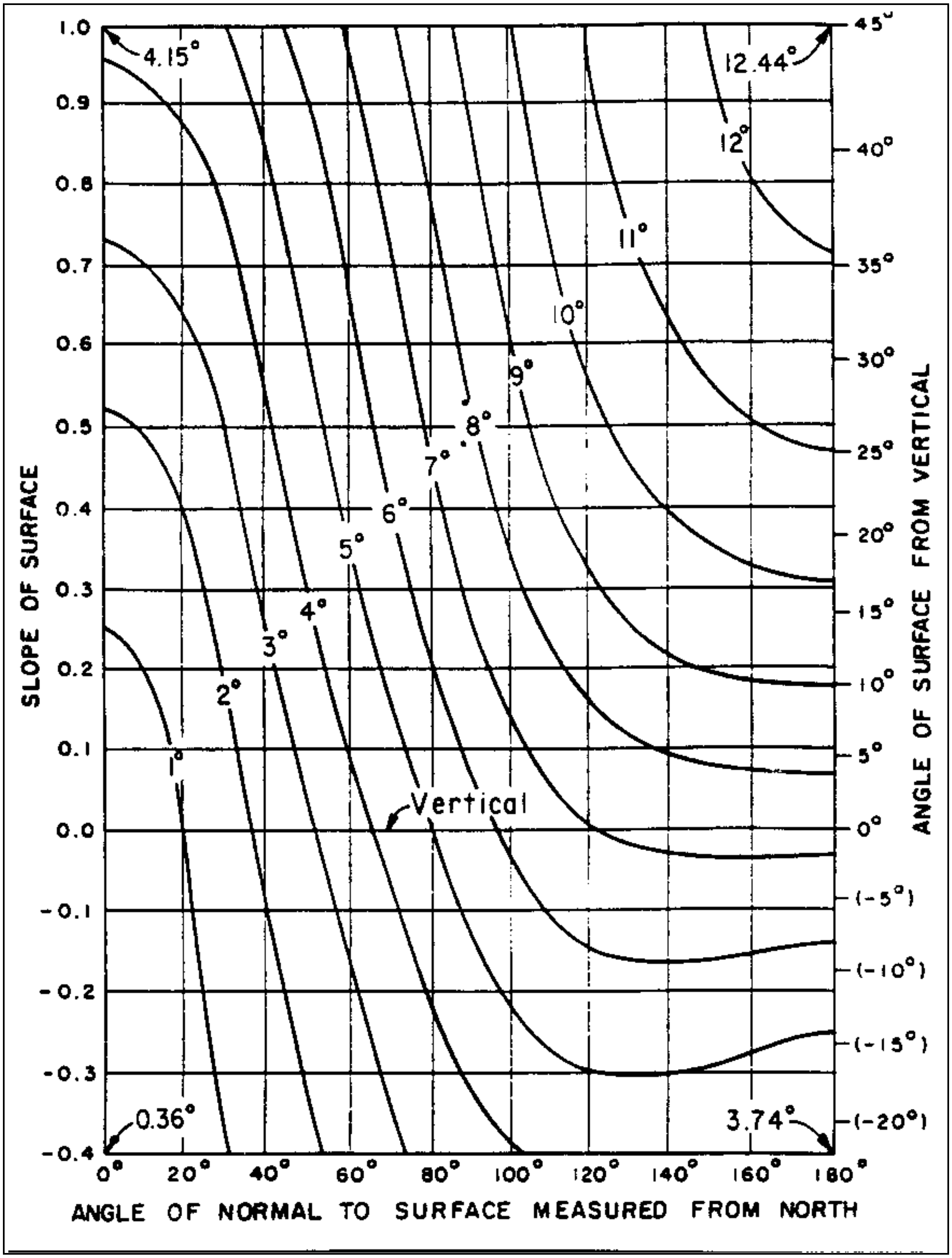

Figure 8-5. Increase in temperature due to solar radiation, latitudes 35° - 40° (USBR)

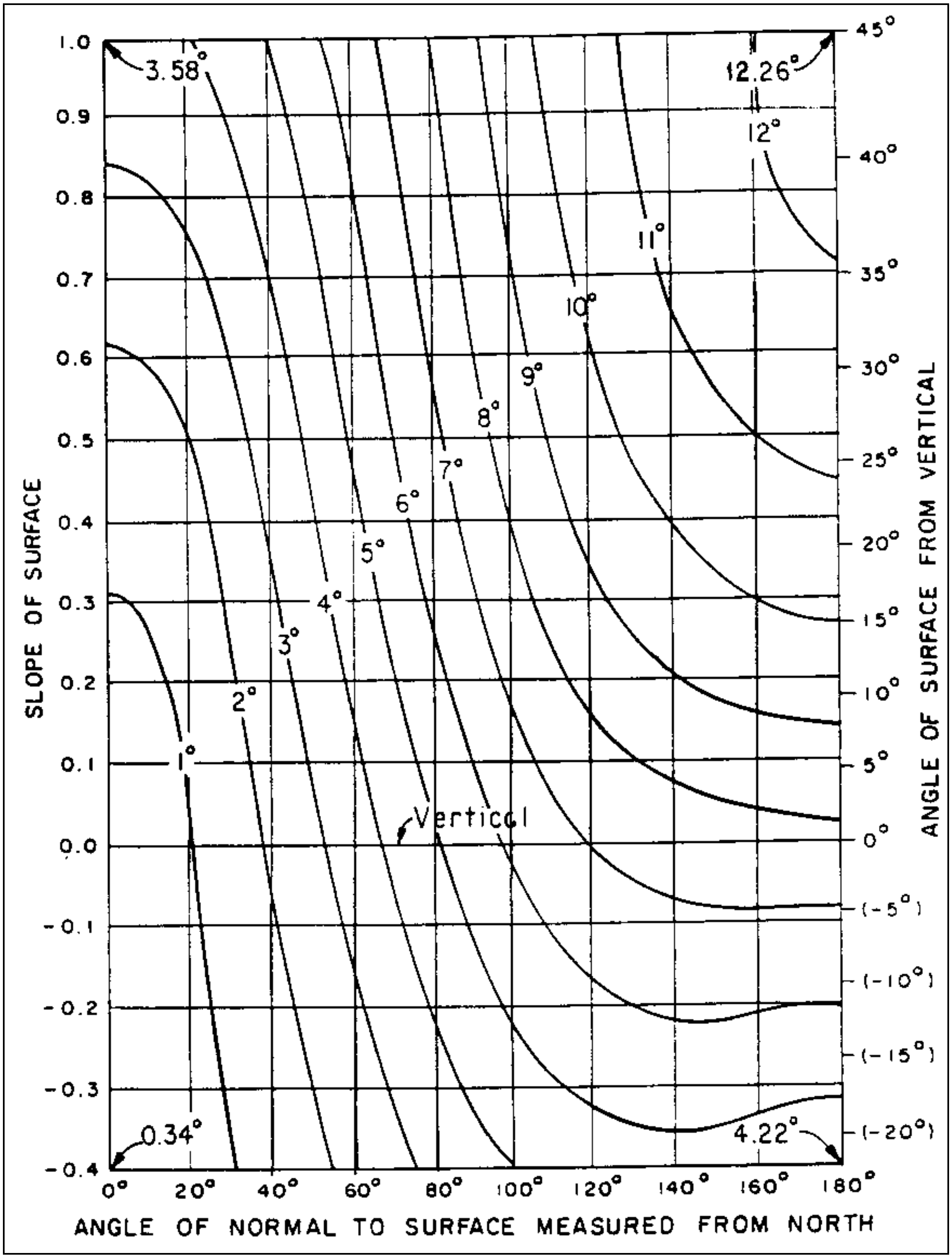

Figure 8-6. Increase in temperature due to solar radiation, latitudes 40° - 45° (USBR)

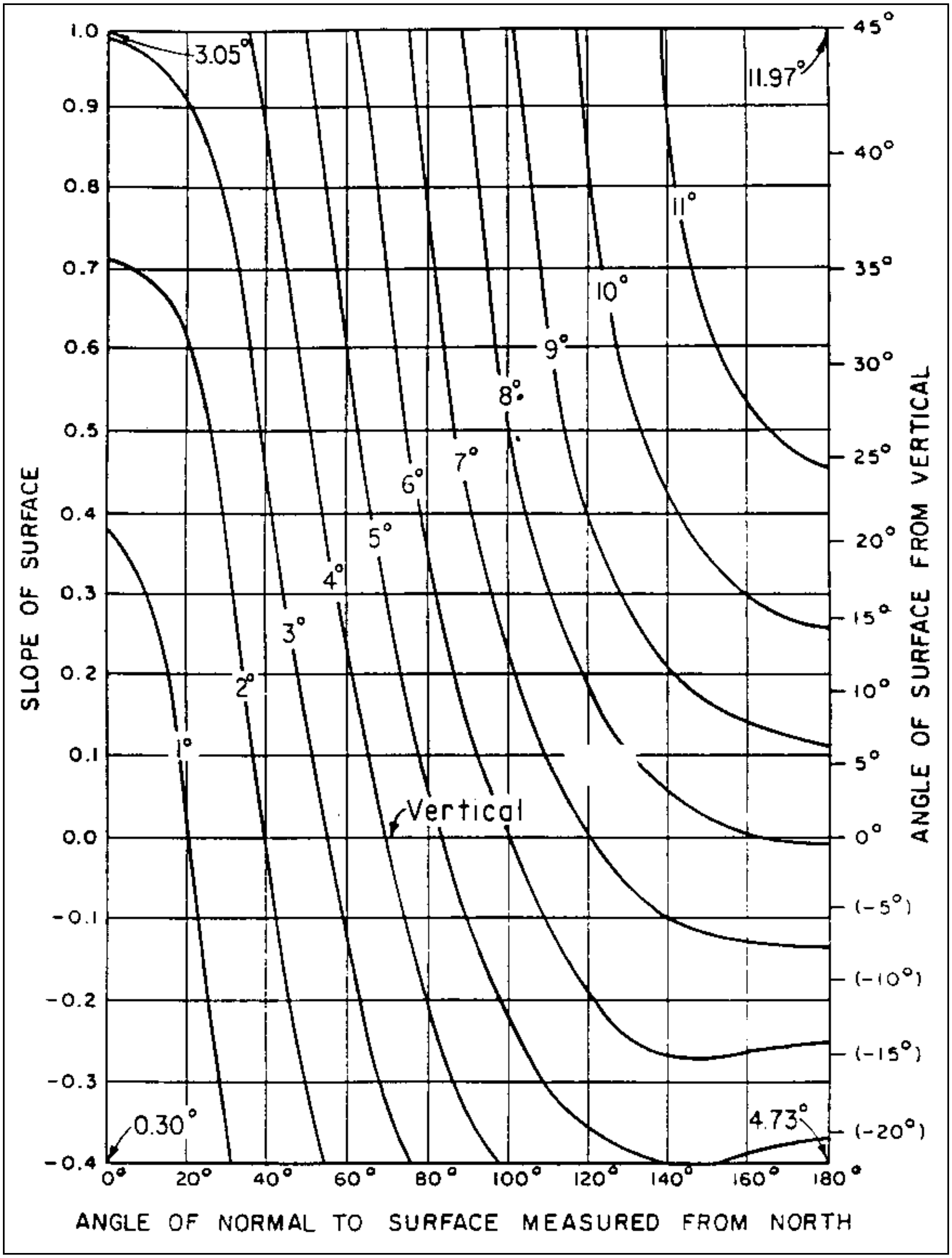

Figure 8-7. Increase in temperature due to solar radiation, latitudes 45° - 50° (USBR)

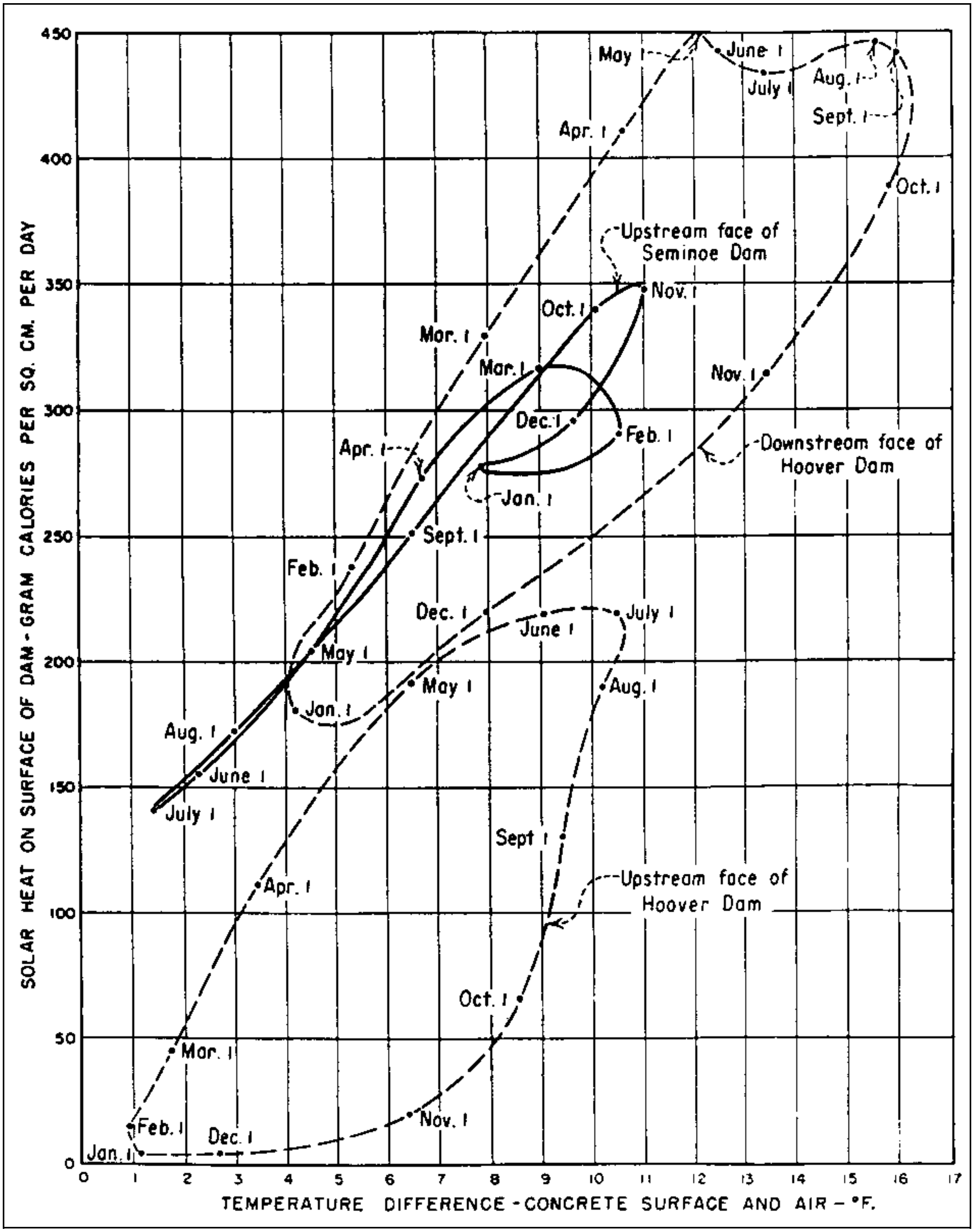

Figure 8-8. Variation of solar radiation during a typical year (USBR)

Library at the U.S. Army Engineer Waterways Experiment Station. Using the computer program will save a great deal of time; however, it would be very instructive to perform the calculation by hand at least once. The steps involved in this process are:

(a) Determine the input temperatures. An explanation of the required data has already been given in paragraphs 8-2b through d.

(b) Determine where in the structure temperatures are desired. These locations should correspond to the "arch" elevations in a trial load analysis and element boundaries or nodal locations in a finite element analysis.

(c) Determine air and water temperature cycles. As previously mentioned, the reservoir temperatures may be assumed to experience only annual temperature cycles. At the elevations of interest, the reservoir cycle would be the average of the maximum water temperature and the minimum water temperature, plus or minus one-half the difference between the maximum and minimum water temperatures. As mentioned before, three air temperature cycles are required. Table 8-1 describes how these cycles are obtained.

(d) Perform the computation. As previously mentioned, the details of the computation are described in the USBR Engineering Monograph No. 34 (Townsend 1965). Only a general description will be presented in this manual. The theory involved is that of heat flow through a flat slab of uniform thickness. The basis of the calculations is a curve of the thickness of the slab versus the ratio: variation of mean temperature of slab to variation of external temperature. To apply the curve, the thickness of the slab is an "effective" thickness related to the actual thickness of the dam, the diffusivity of the concrete, and the air cycle being utilized; yearly, 15-day, or daily cycle. Once the effective thickness is known, the graph is entered and the ratio is read from the ordinate. This is repeated for the three cycles and the ratios are noted. Then, using the cycles for air and then water, the maximum and minimum concrete temperatures for air on both faces and water on both faces are determined. These values are then averaged to determine the range of concrete temperatures for water on the upstream face and air on the downstream face.

(e) Correct for the effects of solar radiation.

(f) Apply results to the stress analysis.

(2) Another method to determine concrete temperatures utilizes finite element techniques. Arch dams are truly 3-D structures from a stress standpoint; however, from a heat-flow standpoint, very little heat will be transmitted in a direction which is normal to vertical planes, i.e., longitudinally through the dam. This allows 2-D heat-flow analyses to be performed. Something to keep in mind is that the results from the heat-flow analyses must be applied to nodes of the 3-D stress model. Therefore, for ease of application, it may be worthwhile to use a 3-D heat-flow model. The benefits of ease of application must be weighed against an increase in computational costs and use of a "coarse" 3-D finite element mesh for the temperature calculations.

TABLE 8-1

Amplitude of Air Temperatures

Period	Extreme Weather Condition		Usual Weather Condition	
	Above	Below	Above	Below
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 absolute 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 the mean)
(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)

Example, ^{o}F

Month	Mean	Mean Max	Mean Min	Difference	High/Low
Jan	47.2	58.8	35.7	23.1	
Feb	51.4	63.5	39.3	24.2	21
Mar	57.1	70.6	43.6	27.0	
Apr	66.0	80.4	51.6	28.8	
May	74.8	89.5	60.0	29.5	
Jun	83.3	98.3	68.4	29.9	
Jul	89.1	102.4	75.6	26.8	116
Aug	86.6	99.7	73.6	26.1	
Sep	81.7	95.3	68.2	27.1	
Oct	70.4	84.1	56.7	27.4	
Nov	57.0	70.0	43.9	26.1	
Dec	49.3	60.4	38.3	22.1	
Annual	67.8	81.1	54.6		

(Continued)

TABLE 8-1. (Concluded)

Mean annual	67.8
Highest mean monthly	89.1
Lowest mean monthly	47.2
Highest	116.0
Lowest	21.0
Highest mean max	102.4
Lowest mean min	35.7
Lowest difference	22.1

Period	Extreme Weather Condition		Usual Weather Condition	
	Above	Below	Above	Below
Annual	21.3	20.6	21.3	20.6
15-day	15.9	15.2	9.1	7.8
Daily	11.0	11.0	11.0	11.0

(a) A finite element model of either the entire dam or of the crown cantilever should be prepared. The water and air cycles are applied around the boundaries of the model and the mean annual air temperature can be applied to the foundation.[1] In most general-purpose finite element programs, steady state and transient solutions are possible. When performing these analyses, the transient solution is utilized. An initial temperature is required. By assuming the initial temperature to be the mean annual air temperature of the site, the transient solution will "settle" to a temperature distribution through the dam that is cyclic in nature. The key to this analysis is to let the solution run long enough for the cycle to settle down. The length of time necessary will be dependent on the thickness of the dam and the material properties. By plotting the response (temperature) of a node in the middle of the dam, a visual inspection can be made and a decision made as to whether or not the solution was carried out long enough. A cyclic response will begin at the initial temperature and the value about which the cycle is fluctuating will drift to a final stable value with all subsequent cycles fluctuating about this value. Based on these results, a solution time step can be chosen to represent the summer and winter concrete temperatures. Then the temperatures can be applied directly to the nodes of the 3-D stress model, if the same model is used for the temperature calculations. If a different model is used for the temperature calculations, a procedure must be developed to spread the 2-D heat flow results throughout the 3-D stress model.

[1] If the dam site is in an area of geothermal activity, the mean annual air temperature may not be appropriate for the foundation temperature. In these cases, data should be collected from the site and foundation temperatures should be used based on this data.

f. Summary. Paragraph 8-2 has described the data necessary to determine the operational temperature loads, the methods that can be used to estimate the data which may not be available at a new dam site, and the methods available to calculate the concrete temperatures. It is necessary for the engineer to determine that the methods used are consistent with the level of evaluation being performed and the stress analysis technique to be employed. The thickness of the dam and, therefore, the resulting temperature distribution should be kept in mind while choosing the temperature analysis technique. The premise here is that thinner structures respond faster to environmental temperature changes than thicker structures. USBR Engineering Monograph No. 34 (Townsend 1965) is a good reference for both the techniques used and data that have been compiled for dams in the western portion of the United States. The Corps' program TEMPER is available to use in determining the range of mean concrete temperatures. Finally, it is important to begin an instrumentation program early in the design process to verify the assumptions made during the temperature calculations.

8-3. Construction Temperatures Studies.

a. General. Before the final stages of the design process it is necessary to begin considering how the dam will be constructed and what, if any, temperature control measures need to be implemented. Temperature controls are usually needed to minimize the possibility of thermally induced cracking, since cracking will affect the watertightness, durability, appearance, and the internal stress distribution in the dam. The most common temperature control measures include precooling, postcooling, using low heat cements and pozzolans, reducing cement content, reducing the water-cement ratio, placement in smaller construction lifts, and restricting placement to nighttimes (during hot weather conditions) or to warm months only (in areas of extreme cold weather conditions). This section will cover precooling methods, postcooling procedures, monolith size restrictions, and time of placing requirements. These items must be properly selected in order that a crack-free dam can be constructed with the desired closure temperature. This section also discusses how these variables influence the construction of the dam and how they can be determined.

b. The Temperature Control Problem. The construction temperature control problem can be understood by looking at what happens to the mass concrete after it is placed.

(1) During the early age of the concrete, as the cement hydrates, heat is generated and causes a rise in temperature in the entire mass. Under normal conditions some heat will be lost at the surface while the heat generated at the core is trapped. As the temperature in the core continues to increase, this concrete begins to expand; at the same time, the surface concrete is cooling and, therefore, contracting. In addition, the surface may also be drying which will cause additional shrinkage. As a result of the differential temperatures and shrinkage between the core and the surface, compression develops in the interior, and tensile stresses develop at the surface. When these tensile stresses exceed the tensile strength capacity, the concrete will crack.

(2) Over a period of time the compressive stresses that are generated in the core tend to be relieved as a result of the creep properties of the

material. As this is happening, the massive core also begins to cool, and it contracts as it cools. This contraction, if restrained by either the foundation, the exterior surfaces, or the previously placed concrete, will cause tensile stresses to develop in the core. As with the previous case, once these tensile stresses exceed the tensile strength capacity of the concrete, the structure will crack.

c. The Ideal Condition. The ideal condition would be simply to eliminate any temperature gradient or temperature drop. This is possible only if the initial placement temperature of the concrete is set low enough so that the temperature rise due to hydration of the cement would just bring the concrete temperature up to its final stable state. For example, if the final stable temperature is determined to be 80 °F and the concrete is expected to have a 30 °F temperature rise, then the initial placement temperature could be set at 50 °F, and the designer could be assured that there would be little chance of thermally induced cracking. This example would result in no volumetric temperature shrinkage. However, it may not always be feasible or economical to place concrete at such a low temperature, especially where the final stable temperature falls below 70 °F. In most cases, it is more economical to set the initial placement temperature slightly above the value that would give the "ideal" condition, thereby accepting a slight temperature drop and a small amount of volumetric temperature shrinkage.

d. Precooling.

(1) Precooling is the lowering of the placement temperature of the concrete and is one of the most effective and positive of the temperature control methods. Precooling can also improve the workability of the concrete as well as reduce the rate of heat generated during the hydration. The initial selection of the placement temperature can be achieved by assuming that a zero-stress condition will exist at the time of the initial peak temperature. A preliminary concrete placement temperature can be selected by using the following expression (American Concrete Institute (ACI) 1980):

$$T_i = T_f + (100*C)/(e*R) - dt \qquad (8\text{-}1)$$

where

T_i = placing temperature
T_f = final stable temperature
C = strain capacity (millionths)
e = coefficient of thermal expansion (millionths/degree of temperature)
R = degree of restraint (percent)
dt = initial temperature rise

In this expression, the final stable temperature is that temperature calculated as described in paragraph 8-2 of this chapter. In the absence of that information, the final stable temperature can be assumed to be equal to the average annual air and water temperatures. By assuming 100 percent restraint (as would occur at the contact between the dam and the foundation), the equation becomes:

$$T_i = T_f + C/e - dt \tag{8-2}$$

As an example, if the average annual air temperature is 45 °F, the slow load strain capacity is 120 millionths, the coefficient of thermal expansion is 5 millionths per °F, and the initial temperature rise is 20 °F, then the maximum placement temperature would be in the order of 49 °F. The material property values for these variables should be obtained from test results and from the other studies discussed in other parts of this chapter. Table 8-2 shows a comparison of the average annual temperature and the specified maximum placement temperature for various arch dams constructed in the United States.

TABLE 8-2

Comparison of Mean Annual and Placement Temperatures (°F)

Dam	Mean Annual	Placement
Swan Lake	45	50
Strontia Springs	52	55
Crystal	37	40-50
Mossyrock	50	60
Morrow Point	39	40-60
Glen Canyon	62	<50

(2) The method or methods of reducing concrete placement temperatures will vary depending upon the degree of cooling required, the ambient conditions, and the contractor’s equipment. The typical methods of cooling concrete are listed in Table 8-3 in approximate order of increasing cost (Waddell 1968).

TABLE 8-3

Precooling Methods

Method of Precooling Concrete	Approximate Temperature Reduction (°F)
Sprinkle coarse aggregate (CA) stockpiles	6
Chill mix water	3
Replace 80% of the mix water with ice	12
Vacuum cool CA to 35 or 38 °F	31
Cold-air cool CA to 40 °F	25
Cool CA by inundation to 40 °F	30
Vacuum cool fine aggregate to 34 °F	12
Contact cool cement to 80 °F	3

e. Postcooling. Postcooling is used both to reduce the peak temperature which occurs during the early stage of construction, and to allow for a uniform temperature reduction in the concrete mass to the point where the monolith joints can be grouted. Postcooling is accomplished by circulating water through cooling coils embedded between each lift of concrete. Following proper guidelines, concrete temperatures can safely be reduced to temperatures as low as 38 ^{o}F. Figure 8-9 shows a typical temperature history for post-cooled concrete. Descriptions of the cooling periods and of the materials and procedures to be used in the postcooling operation are discussed in the following paragraphs.

(1) Initial Cooling Period. During the initial cooling period (see Figure 8-9) the initial rise in temperature is controlled and a significant amount of heat is withdrawn during the time when the concrete has a low modulus of elasticity. The total reduction in the peak temperature may be small (3 to 5 ^{o}F), but it is significant. The initial cooling period will continue to remove a significant amount of heat during the early ages of the concrete when the modulus of elasticity is relatively low. It is preferable, however, not to remove more than 1/2 to 1 ^{o}F per day and not to continue the initial cooling for more than 15 to 30 days. Rapid cooling could result in tensions developing in the area of the cooling coils which will exceed the tensile strength of the concrete.

(2) Intermediate and Final Cooling Periods. The intermediate and final cooling periods are used to lower the concrete temperature to the desired grouting temperature. In general, the same rules apply to the intermediate and final cooling periods as to the initial cooling period except that the cooling rate should not exceed 1/2 ^{o}F per day. This lower rate is necessary because of the higher modulus of elasticity of the concrete. The need for the intermediate cooling period is dependent upon the need to reduce the vertical temperature gradient which occurs at the upper boundary of the grout lift. If an intermediate cooling period is needed, then the temperature drop occurring in the period is approximately half the total required. Each grout lift goes through this intermediate cooling period before the previous grout lift can go through its final cooling.

(3) Materials. The coils used in the postcooling process should be a thin-wall steel tubing. The diameter of the coils is selected as that which will most economically pass the required flow of water through the known length of coil. A small diameter may reduce the cost of the coil, but would increase the pumping cost. Coils with a 1-inch outside diameter are common for small flows. The water used in the postcooling operation must be free of silt which could clog the system. If cool river water is available year round, it usually will be cheaper than refrigerated water provided the required concrete temperature can be obtained within the desired time. The use of river water will usually require more and longer coils and a greater pumping capacity, but it could eliminate the need for a refrigeration plant.

(4) Layout. Individual coils can range in length from 600 to 1,300 feet. However, it is preferable to limit the length of each coil to 800 feet. Wherever possible, horizontal spacings equal to the vertical lift spacings give the most uniform temperature distribution during cooling. With

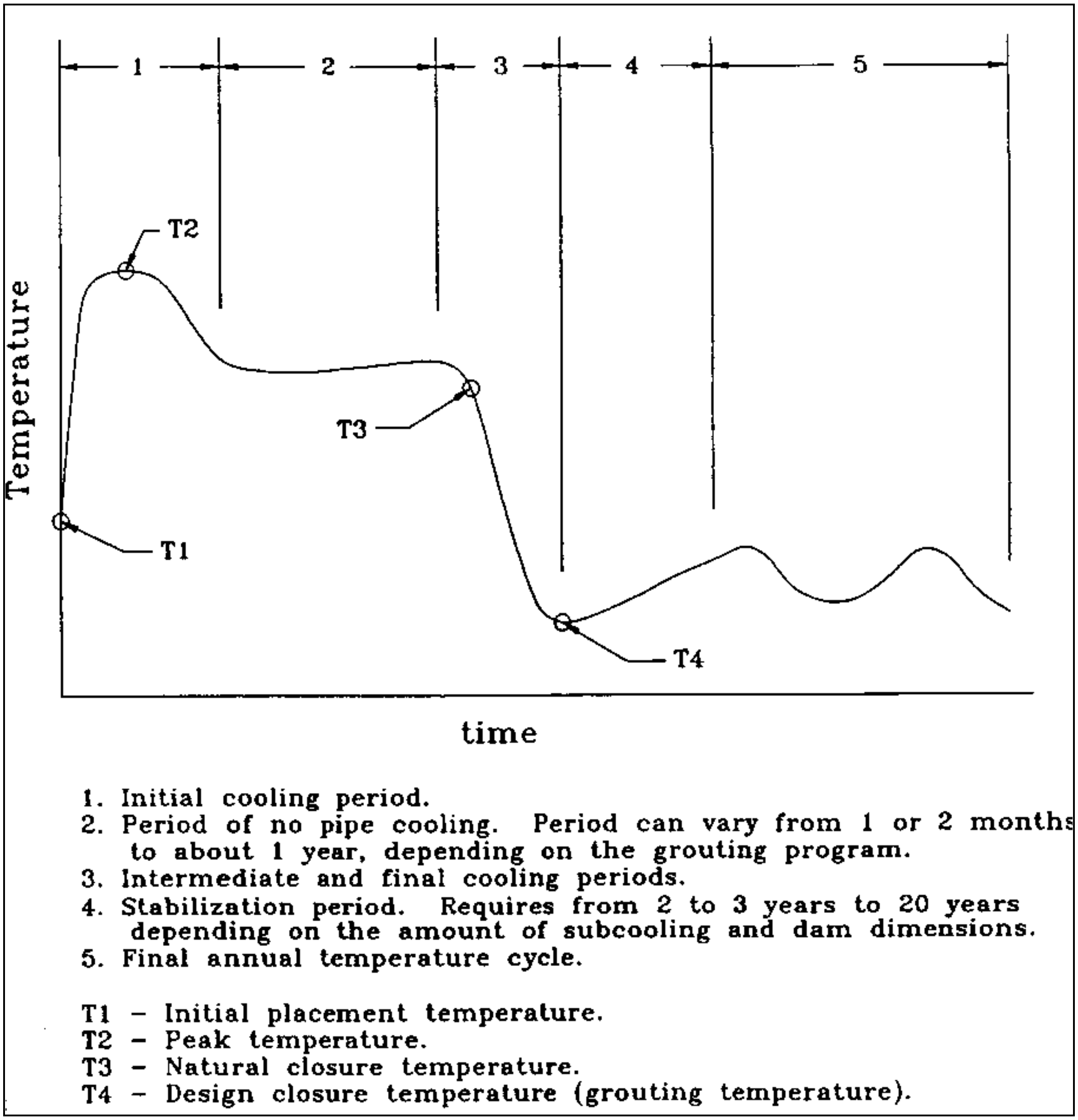

Figure 8-9. Temperature history for artificially cooled concrete where monolith joints are grouted (adapted from Townsend 1965)

lifts in excess of 7.5 feet, this may not be practical. Horizontal spacings from 2 to 6 feet are most common. Coils are often spaced closer together near the foundation to limit the peak temperatures further in areas where the restraint is large.

(5) Procedures. The cooling coils should be fixed in position by the use of tie-down wires which were embedded in the lift surfaces prior to final set. Compression type connections should be used and the coil system should be pressure tested prior to placement of concrete. It should also undergo a pumping test at the design flow to check for friction losses. Each coil

should include a visual flow indicator. Circulation of water through the cooling coils should be in process at the time that concrete placement begins. Since the water flowing through the coil is being warmed by the concrete, reversing the flow daily will give a more uniform reduction in temperature and help to prevent clogging. The cooling operations should be monitored by resistance-type thermometers embedded in the concrete at representative locations. When refrigerated systems are used, the flow seldom exceeds 4 gallons per minute (gpm). These are closed systems where the water is simply recirculated through the refrigeration plant. Systems using river water could have flow rates as high as 15 gpm. In these systems, the water is usually wasted after flowing through the system and new river water is supplied at the intake. Once the final cooling has been completed, the coils should be filled with grout.

f. Closure Temperature Analysis. One of the most important loadings on any arch dam is the temperature loading. The temperature loading is obtained by calculating the difference between the operational concrete temperature (paragraph 8-2) and the design closure temperature (Chapter 4). The design closure temperature is sometimes referred to as the grouting temperature, and is commonly obtained by cooling the concrete to the desired temperature and grouting the joints. However, grouting of the joints may not always be necessary, or possible. In some cases, it may be more desirable to select the placement temperature for the concrete so that the natural closure temperature of the structure corresponds to the design closure temperature. This is the "ideal condition" discussed in paragraph 8-3c. The purpose of the closure temperature analysis is to determine how the design closure temperature can be obtained while minimizing the possibility of cracking the structure.

(1) Before performing a detailed closure temperature analysis, a preliminary (simplified) analysis should be performed. The first step in the closure temperature study is to look at the typical temperature cycle for artificially cooled concrete. Artificially cooled concrete is concrete that incorporates the postcooling procedures discussed in paragraph 8-3e. Figure 8-9 shows a typical temperature cycle for artificially cooled concrete when the joints are to be grouted. The temperatures shown in this figure and those discussed in the next few paragraphs should be considered average temperatures. There are many factors that influence the temperature history including the placement temperature, the types and amounts of cementitious materials, the size of the monoliths, the placement rates, and the exposure conditions. As shown in the figure, there are five phases to the temperature history. Phase 1 begins as the concrete is being placed and continues while the cooling coils are in operation. Phase 2 covers the period between the initial postcooling operations and the intermediate and/or final cooling period. Phase 3 is the phase when the postcooling is restarted and continues until the joints are grouted. Phase 4 is the period after the grouting operation in which the concrete temperatures reach their final stable state. Phase 5 is the continuation of the final annual concrete temperature cycle, or the operating temperature of the structure, which is discussed in paragraph 8-2.

(2) There are four important points along this temperature history line which are determined as part of the closure temperature analysis. Temperature T1 is the placement temperature of the concrete. Temperature T2 is the maximum or peak temperature. Temperature T3 is the natural closure temperature,

or the temperature at which the joints begin to open. Temperature T4 is the design closure temperature, or the temperature of the concrete when the contraction joints are grouted. The preliminary analysis can be made to assure that the dam is constructable by evaluating each of these four temperatures. This is done by starting with temperature T4 and working back up the curve.

(a) Temperature T4 is set by the design analysis and is, therefore, fixed as far as the closure temperature analysis is concerned. For the example discussed in the next few paragraphs, a design closure temperature (T4) of 50 °F is assumed.

(b) Temperature T3 can be calculated by selecting a monolith width, using the coefficient of thermal expansion test results and assuming a required joint opening for grouting. An arch dam with a 50-foot monolith, a 5.0×10^{-6} inch/inch/°F coefficient of thermal expansion, and a joint opening of 3/32 inch would require a temperature drop of:

$$\Delta T = \frac{3/32 \text{ inch}}{50 \text{ feet} \times 12 \text{ inches/foot}} \times \frac{1}{5.0 \times 10^{-6}/°F} = 31.25\ °F \qquad (8\text{-}3)$$

For this type of analysis, temperatures can be rounded off to the nearest whole degree without a significant impact in the conclusions. Therefore, a ΔT of 31 °F is acceptable and T3 becomes 81 °F.

(c) The difference between T3 and T2 will vary according to the thickness of the lift and the placement temperature. This variation is usually small and is sometimes ignored for the preliminary closure temperature analysis. If included in the analysis, the following values can be assumed. For lift heights of 5 feet, a 3 °F difference can be assumed. For 10-foot lifts, a 5 °F temperature difference is more appropriate. Therefore, for a 10-foot lift height the average peak temperature (T2) becomes 86 °F (81 + 5 °F).

(d) The placement temperature (T1) can be calculated based on the anticipated temperature rise caused by the heat of hydration. There are many factors that influence the temperature rise such as the type and fineness of cement, the use of flyash to replace cement, the lift height, the cooling coil layout, the thermal properties of the concrete, the ambient condition, the construction procedures, etc. Because of the variety of factors affecting temperature rise, it is difficult to determine this quantity without specific information about the concrete materials, mix design, and ambient conditions. For the example discussed in this section, we will simply assume that a 25 °F difference exists between T1 and T2, which is somewhat typical when Type II cement and flyash are used in the concrete mix and a 10-foot lift height is selected. This 25 °F temperature rise will yield a placement temperature of 61 °F. Allowing for some error in the analysis and some variation during the construction process, a temperature range of 60 $\pm$ 5 °F would be specified for this example.

(3) Using the procedure in the previous paragraphs, the temperatures along the temperature history curve can be estimated. The next step in this preliminary closure temperature analysis is to determine if any of the

temperatures and/or changes in temperatures could result in thermal cracking, or if they represent conditions which are not constructable. Two aspects of the temperature history need to be closely evaluated:

(a) The placement and peak temperatures. To be economical, the placement temperature should be near the mean annual air temperature. If the calculated placement temperature from the preliminary analysis is less than 45 °F or greater than 70 °F, or if placement temperature is 10 °F above the mean annual air temperature, then a more detailed closure temperature analysis should be performed. A detailed analysis should also be performed if the required peak temperature (T2) is above 105 °F.

(b) The temperature drop from the peak to design closure temperature. The strain created during the final cooling period should not exceed the slow load strain capacity of the concrete as determined from test results (see Chapter 9). The maximum temperature drop can be determined by dividing the slow load strain capacity by the coefficient of thermal expansion. For example, if the slow load strain capacity is 120 millionths and the coefficient of thermal expansion is 5 millionths per °F, the maximum temperature drop will be 24 °F. Based on the values assumed in paragraph 8-3f(2)(c) (required temperature drop of 36 °F), the monolith width would need to be increased, or a more detailed closure temperature analysis would be required. In this case, by increasing the monolith width to 80 feet the required temperature drop would be reduced to 24.5 °F. The combination of the large monolith width and excessive temperature drop would usually require that a detailed closure analysis be performed to more accurately determine the construction parameters and temperature values.

(4) If either of the conditions stated in paragraph 8-3f(3) indicate a problem with obtaining the design closure temperature without jeopardizing the constructability of the dam, then a more detailed closure temperature analysis is required. The details of how to perform a detailed closure temperature analysis are presented in the next paragraphs.

(5) To perform a detailed closure temperature analysis, the following assumptions are required:

(a) The principle of superposition must apply. That is, the strains produced at any increment of time are independent of the effects of any strains produced at any previous increment of time.

(b) When the monolith joints are closed, the concrete is restrained from expanding by the adjacent monoliths and compressive stresses will develop in the monolith joints.

(c) The concrete is not restrained from contraction. In other words, no tensile stresses will develop due to contraction of the concrete. Contraction of the concrete will produce either a relaxation of compressive stresses at the joint, or a joint opening.

(d) Joint opening will occur only after all compressive stresses have been relieved.

(e) Creep is applied only to compressive stresses.

(f) Only the effects of thermal expansion or contraction and added weight are considered.

(6) To perform the closure temperature analysis, the time varying properties of coefficient of linear thermal expansion, rate of creep, modulus of elasticity, and Poisson's ratio will be needed. These material properties will be needed from the time of placement through several months. Chapter 9 furnishes more information on the material testing required.

(7) The first step in the closure temperature analysis is to predict the temperature history of a "typical" lift within the dam. This can be done with a heat-flow finite element program. The details of such a heat-flow analysis are discussed in ETL 1110-2-324. The main difference between the details discussed in the ETL and those discussed in this section is that the information needed for a closure temperature can be simplified such that the entire structure need not be modeled if a "typical" temperature history for each lift can be estimated. This can usually be done with a 2-D model with a limited number of lifts above the base of the dam. Ten lifts will usually be sufficient for most arch dam closure temperature analyses. If the thickness of the dam changes significantly near the crest, then additional heat-flow models may be necessary in that region.

(8) Once the temperature history of a "typical" lift has been estimated, the next step is to calculate the theoretical strain caused by the change in temperature for each increment of time. This theoretical strain is calculated by:

$$\varepsilon_t = e_i * \Delta T = e_i * (T_i - T_{i-1}) \quad (8\text{-}4)$$

where

ε_t = incremental strain due to the change in temperature from time t_{i-1} to t_i
e_i = coefficient of linear thermal expansion at time t_i
ΔT = change in temperature
T_i = temperature at time t_i
T_{i-1} = temperature at time t_{i-1}

(9) In addition, the theoretical strain due to construction loads can be added by the following equation:

$$\varepsilon_{wt} = \frac{\mu_i * \Delta wt}{E_i} = \frac{\mu_i * (wt_i - wt_{i-1})}{E_i} \quad (8\text{-}5)$$

where

ε_{wt} = incremental strain due to added weight from time t_{i-1} to t_i
μ_i = Poisson's ratio at time t_i
Δwt = the incremental change in weight
E_i = modulus of elasticity at time t_i
wt_i = weight at time t_i
wt_{i-1} = weight at time t_{i-1}

(10) The total incremental strain is the sum of the incremental strain due to changes in temperature and added weight, as follows:

$$\varepsilon_i = \varepsilon_t + \varepsilon_{wt} = e_i * (T_i - T_{i-1}) + \frac{\mu_i * (wt_i - wt_{i-1})}{E_i} \qquad (8\text{-}6)$$

where

ε_i = total incremental strain at time t_i

(11) The incremental stress can be calculated by:

$$\sigma_i = \varepsilon_i * E_i \qquad (8\text{-}7)$$

where

σ_i = total incremental stress at time t_i

(12) Once the stress has been determined for each time increment, creep can be applied to the stress to determine how that incremental stress is relaxed over time. The following equation applies to stress relaxation under constant strain:

$$\sigma_{i-n} = \frac{1}{1/E_i + [c_i * \ln(t_n - t_i + 1)]} \text{ per unit strain} \qquad (8\text{-}8)$$

where

σ_{i-n} = stress at time t_n due to an increment of strain at time t_i
c_i = rate of creep at time t_i

(13) To estimate the total stress at any time t_n, the following equation can be used:

$$\sigma_n = \sum_{i=1}^{n} \sigma_{i-n} \qquad (8\text{-}9)$$

where

σ_n = total stress at time t_n

(14) If the total stress in the monolith joint at the end of time t_n is in compression ($\sigma_n \geq 0$), then the temperature drop necessary to relieve the compressive stress can be determined by:

$$dT_n = T_n - T'_n = \frac{\sigma_n}{e_n * E_n} \qquad (8\text{-}10)$$

where

T'_n = natural closure temperature of the structure at time t_n
T_n = concrete temperature at time t_n.

(15) Under normal circumstances, T'_n should not vary significantly after 20 to 30 days after concrete placement and can simply be referred to as T′. In the closure temperature analysis, the steady state value for T′ is the critical value for estimating the monolith width. With T′ and the design closure temperature, the minimum monolith width required to be able to grout the monolith joints can be determined by:

$$\ell_{min} = \frac{x}{e_n * (T' - T_g)} \qquad (8\text{-}11)$$

where

ℓ_{min} = minimum size (width) of monolith that will produce an acceptable joint opening for grouting
x = joint opening needed to be able to grout the joint
T_g = temperature at which the joints are to be grouted (the design closure temperature)

g. The Ungrouted Option. If the preliminary and/or detailed closure temperature analysis indicates a problem with obtaining the design closure temperature because the required placement temperatures are higher than acceptable (greater than 70 ^oF), then the ungrouted option should be considered. The ungrouted option assumes that the "natural" closure temperature is the same as the "design" closure temperature. Figure 8-10 shows the temperature cycle for the ungrouted option. In this option, the concrete is placed at a low enough temperature such that the natural closure temperature falls within a specified value. A detailed closure temperature analysis is required in order to obtain adequate confidence that the dam will achieve the required closure temperature. Design Memorandum No. 21 (US Army Engineer District (USAED), Jacksonville, 1988 (Feb)) provides additional details of the analysis for the ungrouted option.

h. Nonlinear, Incremental Structural Analysis. Once the closure temperature study has been satisfactorily completed, the next step is to perform a nonlinear, incremental structural analysis (NISA) using the construction parameters resulting from the closure temperature study. ETL 1110-2-324 provides guidance for performing a NISA. If the structural configuration or the construction sequence is modified as a result of the NISA, then a reanalysis of the closure temperature may be required.

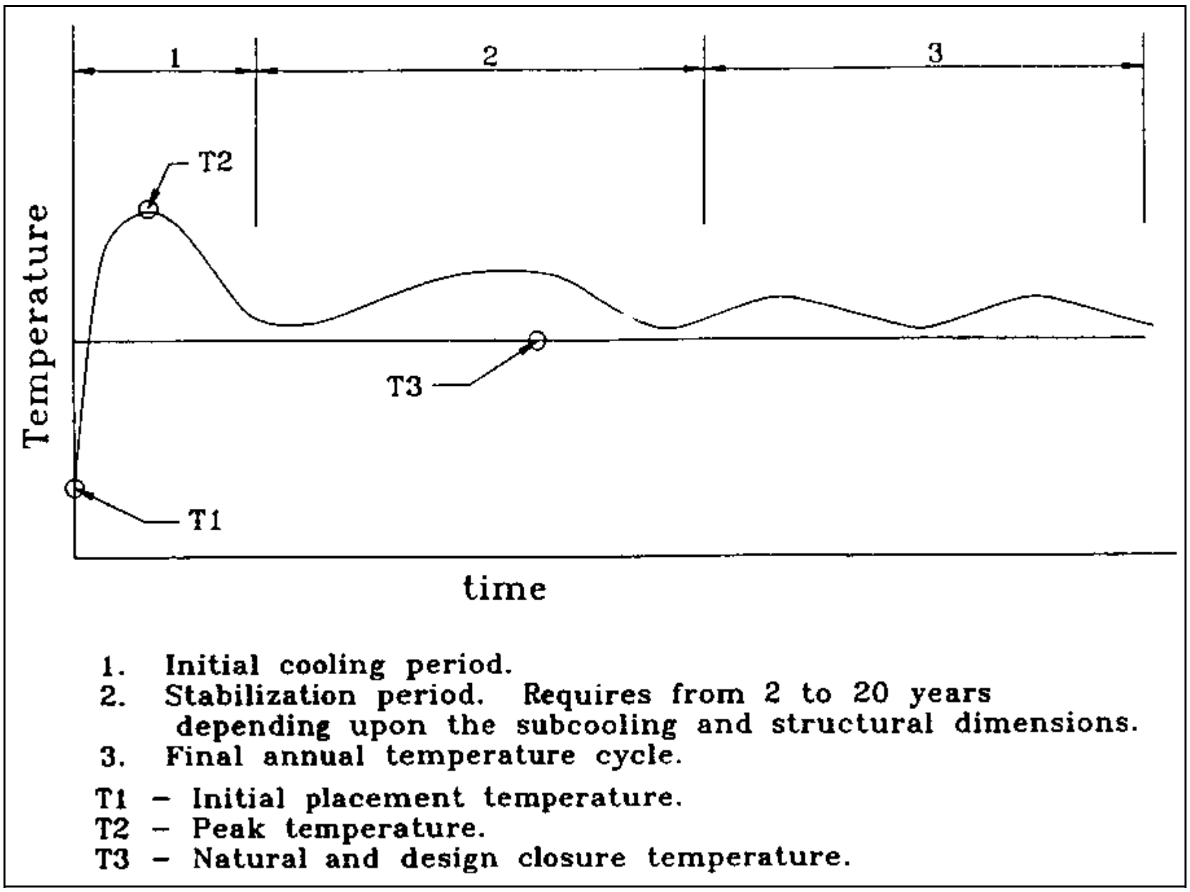

Figure 8-10. Temperature history for artificially cooled concrete where monolith joints are not grouted