DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

CECW-EG

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Technical Letter
No. 1110-2-542

30 May 1997

Engineering and Design
THERMAL STUDIES OF MASS CONCRETE STRUCTURES

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1. Purpose

This engineer technical letter (ETL) provides guidance for performing thermal studies of mass concrete structures (MCS) as required by Engineer Manual (EM) 1110-2-2000.

2. Applicability

This ETL applies to HQUSACE elements and USACE commands having responsibilities for the design of civil works projects.

3. References

References are listed in Annex 4.

4. Discussion

a. *Background*. The effects of heat generation in mass concrete were first recognized in the 1920's and 1930's, including the development of artificial cooling of mass concrete using chilled water flowing through embedded pipe. Early thermal analysis of mass concrete made use of very simple concepts and various stepwise hand calculation methods of determining temperature changes. Later development of finite element (FE) techniques made possible more accurate and realistic thermal analysis, culminating in the current development of nonlinear incremental structural analysis (NISA). Current mass concrete thermal analysis practice may employ a variety of methodologies, varying from simple hand calculations and charts using broad assumptions for conditions and concrete properties, to computer spreadsheet temperature balance methods, to FE temperature and stress/strain analysis, and finally NISA.

b. *Types of mass concrete structures*. Three types of MCS are commonly used in civil works projects: (1) gravity structures such as dams and lock walls; (2) thick shell structures such as arch dams; and (3) thick reinforced structures such as U-frame locks, large pumping stations, powerhouses, large foundations, and massive bridge piers. MCS constructed using the roller-compacted concrete (RCC) construction method are treated in this ETL identically to structures constructed using traditional construction methods.

c. *ETL content*. Thermal studies for MCS have been categorized into three levels of increasing complexity to provide a convenient frame of reference. This ETL specifically provides information and guidance for thermal studies of MCS and provides methodology for the first two levels of thermal studies. The methodology for the more complex third level is provided by ETL 1110-2-365, which includes many subjects pertinent to Level 1 and Level 2 thermal analyses. EM 1110-2-2201 contains explicit procedures for preliminary temperature studies for arch dams that eventually lead to NISA.

(1) Appendix A provides detailed information and practice for mass concrete thermal studies.

(2) Annex 1 presents current practice for determination of concrete tensile strain capacity for use in cracking analysis.

(3) Annex 2 provides a stepwise procedure for simple, Level 1 thermal analysis, including an example.

(4) Annex 3 provides a procedure for more intensive Level 2 thermal analysis, including an example using simple FE, one-dimensional (1-D) strip models and an example using more complex two-dimensional (2-D), FE methodology.

5. Guidance

a. *Descriptions and applications of thermal analysis methods*. Thermal analysis is categorized into three levels of complexity. These levels are identified to provide a convenient frame of reference for the analytical processes available to the designer. The level of thermal analysis selected should be appropriate for the size, type, function and risk, and stage of design of the structure, as well as the potential for cost savings resulting from the analysis. Appendix A provides a suggested process for selecting and conducting thermal analysis appropriate for MCS. Small, low-head MCS may require no more than a very simplified thermal analysis. A larger structure, such as a concrete gravity dam, may need only a simplified thermal study at the feasibility level of design, but a more thorough study during preconstruction engineering and design (PED) phase. Certain MCS such as complex lock walls, high gravity dams, and arch dams, may require a NISA during PED. Cost savings may be realized through an adequate thermal study when unnecessary joints can be eliminated or construction controls, such as concrete placing temperatures, can be relaxed. Each higher level of analysis may provide more detailed information but, generally, at a price of increasing complexity and cost of the analytical effort.

(1) Level 1 analysis. This is the simplest level of thermal analysis, using very basic methodology, requiring little or no laboratory testing, and incorporating broad assumptions for site conditions and placement constraints. This level of analysis should be used in thermal evaluations of a general nature, where the consequences of thermal cracking are a concern but pose little safety or stability concerns. The method is appropriate for the project feasibility stage to determine if higher level analysis is necessary for PED and for initial verification of construction controls and structural features such as joint spacing and lift heights. It is applicable to small and low-head structures and those structures where thermal cracking poses little risk of loss of function. These structures may include diversion structures for irrigation canals, low-head flood protection structures, low-head MCS that impound water on an infrequent basis for short durations, and thick reinforced structures such as foundations and massive bridge piers. Annex 2 of Appendix A illustrates this level of analysis.

(2) Level 2 analysis. Level 2 thermal analysis is characterized by a more rigorous determination of concrete temperature history in the structure and the use of a wide range of temperature analysis tools. This level of analysis should be applied to thermal evaluations of more critical structures where the consequences of thermal cracking may pose a significant risk to people or property, may present stability concerns or loss of function, or may result in significant cost savings. This level of analysis is recommended to better identify thermal cracking potential and minimize specific requirements necessary for thermal crack control that can add significant cost to construction. Level 2 analysis may be appropriate for the feasibility study phase of significant structures and may be used to determine if higher-level analysis is necessary during PED. Level 2 thermal analysis is also appropriate for PED for significant MCS. It is applicable to medium to high-head flood protection structures and other significant MCS. These structures may include complex lock walls, medium to high gravity dams, tunnel plugs involving postcooling and grouting, pumping stations, powerhouses, and low-head arch dams. Annex 3 of Appendix A illustrates this level of analysis.

(3) Level 3 analysis. This level is the most complex level of thermal analysis. ETL 1110-2-365 describes the computational methodology and application of Level 3 (NISA) analysis, and ETL 1110-2-536 presents an example of NISA application to the Zintel Canyon Dam. This level of analysis is suitable for very critical structures where cracking poses significant risks. The designer must weigh the high costs of NISA evaluation against the potential benefits of increased analysis detail and capability of simultaneously analyzing thermal and

other structure loading. The method is applicable to critical, high-risk projects, complex or unprecedented structures with little or no previous experience, and structures subject to stress interaction from several simultaneous loading conditions. This level of analysis may also be appropriate for normal thermal studies of more ordinary MSC to optimize thermal controls and potentially reduce construction costs. Candidates for NISA include high gravity dams, arch dams, large and complex lock walls.

b. *Cracking analysis methods*. Analysis of cracking for Levels 1 and 2 MCS thermal analysis is performed based on the computed concrete temperature distributions, using simplified procedures to relate thermal changes in volume of the MCS to estimate cracking potential. The procedures involve approximations and require assumptions regarding conditions of restraint. Cracking analysis methodology for Levels 1 and 2 thermal analysis is described in Appendix A. For NISA, the cracking analysis is integral with the incremental FE thermal stress-strain analysis as described in ETL 1110-2-365.

6. Action

a. *Thermal analysis needs.* As required in EM 1110-2-2000, concrete thermal studies are to be performed for any important concrete structure where thermal cracking potential exists. The design team must evaluate the necessity of a thermal study and select the appropriate level of analysis in accordance with the criteria outlined herein. Guidance for performing thermal studies is given in Appendix A.

b. *Stage of project development.* Evaluation of the thermal study requirements should be done during the Feasibility Phase of project development. Necessary design studies and resources should be included in the Project Management Plan. Proper identification of objectives is the key to determining the required scope of studies. Contact CECW-EG and CECW-ED for assistance in determining appropriate levels of investigation and the necessary resources. Thermal studies are usually performed during the PED phase when project concrete materials and mixtures have been identified. However, the most basic studies may be performed during a feasibility study for a major project or for a complex structure where thermal cracking issues may control subsequent design changes and more complex analysis. Testing requirements should be coordinated to ensure test data are ready at the appropriate time of the study. Appendix A contains more detailed information related to thermal analysis and stages of project development.

c. *Testing.* The material properties for thermal studies should be based on test results of proposed concrete mixtures for the project, if appropriate to the level of study, the phase of project study, and requirements of the particular project. If concrete properties testing is not appropriate for a specific project, data will be obtained from various published sources and from consultation with concrete specialists at various Field Operating Activities (FOA) and CEWES, and with outside technical specialists.

d. *Responsible parties.* The materials or structural engineer primarily responsible for the thermal study must ensure that adequate input is obtained from materials, structural, geotechnical, and construction engineers. Coordination is required for selection of environmental conditions, concrete properties, foundation properties, and construction parameters. Review of the thermal study should be conducted at levels commensurate with the scope of the thermal study to ensure that the plan of action being pursued is appropriate. Concrete specialists at various FOA and CEWES, or outside technical specialists, should be consulted for guidance during Level 2 or 3 thermal analysis of MCS.

e. *Construction.* If construction conditions or requirements change significantly from that assumed during the thermal analysis, the designer should evaluate the need to conduct additional thermal studies. Instrumentation should be installed in important MCS to verify design assumptions and analysis.

f. *Documentation.* Results of the thermal study should be documented in an appropriate design report.

FOR THE COMMANDER:

1 Appendix
App A - Techniques for Performing
Concrete Thermal Studies

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APPENDIX A: TECHNIQUES FOR PERFORMING CONCRETE THERMAL STUDIES

LEVEL 1 AND LEVEL 2

A-1. Introduction

a. Content. This appendix presents general techniques for performing a thermal analysis for mass concrete structures (MCS), with more detailed procedures and examples provided in the annexes. The appendix discusses the general process for thermal studies, thermal analysis concepts, available analytical methods for temperature calculation, data collection, temperature analysis, cracking analysis, documentation of thermal analysis, limitations of thermal analysis, and references. Annex 1 presents current practice for determination of concrete tensile strain capacity for use in cracking analysis. Annex 2 provides a stepwise procedure for simple, Level 1 thermal analysis, including an example. Annex 3 provides a procedure for more intensive Level 2 thermal analysis, including an example using simple finite element (FE), one-dimensional (1-D) strip models and an example using more complex two-dimensional (2-D), FE methodology.

b. Purpose. MCS are constructed using the principles and methods defined for mass concrete by American Concrete Institute (ACI) Committee 207, and Engineer Manual (EM) 1110-2-2000. There are three types of MCS commonly used for civil works projects. Gravity structures are used for dams and lock walls; thick shell structures are used for arch dams; and thick, reinforced plate structures are used for U-frame locks, large pumping stations, powerhouses, large foundations, and massive bridge piers. Arch dam thermal analysis is described in detail in EM 1110-2-2201, which contains specific procedures and considerations that may require a Level 3 nonlinear incremental structural analysis (NISA) analysis.

(1) Thermal analysis considerations. A thermal analysis should account for the environmental conditions at the site, the geometry of the structure, the behavior properties of plain or reinforced concrete members, construction conditions, and should provide a basis for comparing thermal generated strain in the structure with strain capacity of the concrete. An analysis may also need to account for the nonlinear behavior of the concrete members, the interaction of the structure, foundation, and backfill, and the effects of sequential construction, thermal gradients, and other loadings on the structure. Very accurate prediction of temperature distribution, resulting strain and stress, and the prediction of cracking in mass concrete is often difficult, if not impossible, due to the complexity of conditions and the many uncertainties in materials, properties, and construction conditions. However, the information, tools, and methods for thermal analysis described in this document provide a basis for thermal analysis that is sufficiently accurate for sound engineering purposes.

(2) Thermal cracking. While cracking is inherent and of little consequence in some concrete structures, other structures may require a relatively uncracked monolithic condition to function as designed. Subsequent cracking, in the latter case, may render such a structure unstable under design conditions or may allow unnecessary or damaging seepage of water. Cracking in some MCS may increase deterioration rates, the results of which, while not structurally damaging, may introduce significant increases in long-term maintenance or repair costs. In many structures with high public visibility, control of cracking may also be desirable for esthetic reasons.

(3) Thermal analysis objectives. A thermal analysis is necessary and cost effective to attain any of the following design objectives:

- To develop materials and structural and construction procedure requirements for use in feasibility evaluation, design, cost engineering, specifications, and construction of new MCS. Thermal studies provide a rational basis for specifying construction requirements. A thermal study provides a guide for formulating advantageous design features, optimizing concrete mixture proportions, and implementing necessary construction requirements.

- To provide cost savings by revising the structural configuration, material requirements, or construction sequence. Construction requirements for concrete placement temperature, mixture proportions, placement rates, insulation requirements, and schedule constraints that are based on arbitrarily selected parameters can create costly operations. Cost savings may be achieved through items such as eliminating unnecessary joints, allowing increased placing temperatures, increased lift heights, and reduced insulation requirements.

- To develop structures with improved performance where existing similar structures have exhibited unsatisfactory behavior (such as extensive cracking) during construction or operation. Cracking which requires remedial repairs would be considered unsatisfactory behavior. Cracking which does not affect the overall structural behavior or some function of the structure would not be classified as unsatisfactory behavior.

- To more accurately predict behavior of unprecedented structures for which limited experience is available, such as structures with unusual structural configuration, extreme loadings, unusual construction constraints, or severe operational requirements.

(4) Counteracting thermal cracking. Provisions to counteract predicted thermal cracking are discussed in ACI 207 documents, and typically include:

- Changes in construction procedures, including placing times and temperatures.

- Changes in concrete materials and thermal properties.

- Precooling of concrete materials and controls on concrete placement temperature.

- Postcooling of concrete.

- Construction of joints (with waterstops where necessary) to control location of cracks.

- Construction of water barrier membranes to prevent water from entering cracks.

- Alteration of structure geometry to avoid or control cracking.

- Use and careful removal of insulation.

c. *Project design process.* A thermal analysis should be performed as early in the design process as possible, but it is preferable that the actual performance of a thermal analysis not take place until test data are available which will typically occur during the preconstruction engineering and design (PED) phase. EM 1110-2-2201 provides project design process considerations for Arch Dams.

(1) Project feasibility. Early in the feasibility phase of project design, the need to perform a thermal analysis should be evaluated, based on the objectives stated above. Any potential construction savings, historical problems related to structural behavior, or special unprecedented structural features should be identified. Proposed solutions requiring thermal analysis should be presented, and the necessary design studies along with their associated costs and schedule should be included in the Project Management Plan as described in Engineer Regulation (ER) 1110-2-1150. A thermal analysis more complex than Level 1 should be performed during the feasibility phase only for very significant or unprecedented structures, and/or those with requirements for unusual construction procedures, and when it has been determined that these factors will significantly affect project costs. A Level 1 thermal analysis during the feasibility phase is primarily to provide insight and information as to whether or not design features and construction requirements for the structure are viable.

(2) PED. The initial investigations needed to verify the potential cost savings, functional improvements, or predicted behavior should be performed in the early stages of the PED. The thermal analysis should include project specific material properties based on test data if appropriate. Initial analyses should be used to investigate 1-D portions of the structure. These analyses should be used to evaluate the need for more advanced thermal analysis, as well as the potential changes needed in design, material properties, or construction parameters.

d. *Thermal analysis concepts*. Mass Concrete is defined by ACI as “any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change to minimize cracking.” When portland cement combines with water, the ensuing exothermic (heat-releasing) chemical reaction causes a temperature rise in the concrete mass. The actual temperature rise in an MCS depends upon the heat-generating characteristics of the mass concrete mixture, its thermal properties, environmental conditions, geometry of the MCS, and construction conditions. Usually the peak temperature is reached in a few days to weeks after placement, followed by a slow reduction in temperature. Over a period of several months to several years, the mass eventually cools to some stable temperature, or a stable temperature cycle for thinner structures. A change in volume occurs in the MCS proportional to the temperature change and the coefficient of thermal expansion of the concrete. If volume change is restrained during cooling of the mass, by either the foundation, the previously placed concrete, or the exterior surfaces, sufficient tensile strain can develop to cause cracking. Cracking generally occurs in the main body or at the surface of the MCS. These two principal cracking phenomenons are termed mass gradient and surface gradient cracking, respectively. ACI 207.1R, contains detailed information on heat generation, volume change, restraint, and cracking in mass concrete.

A-2. General Process, Analysis, and Coordination for Thermal Studies

a. *Process*. The thermal study process at any level consists of several steps which are summarized in Table A-1. These steps are similar for all levels of analysis. The steps can be subdivided amongst three general tasks: data collection, temperature analysis, and cracking analysis. The specific efforts within each of these tasks can vary considerably, depending upon the level of analysis selected for the thermal study. Data collection includes those steps that provide input data and preparation of input for subsequent analysis tasks. Data collection may include information retrieval and testing. Temperature analysis generates the temperatures or temperature histories for the MCS, which are possible scenarios of thermal loadings during construction and subsequent cooling. Cracking evaluation uses temperature data from the temperature analysis, other sources of loading, material properties, concrete/ foundation interaction, geometry, construction parameters, etc., to compute strains and evaluate the potential for cracking in the MCS. This process is directly applicable for evaluating mass gradient and surface gradient cracking for thermal studies (Levels 1 and 2) and for advanced FE thermal studies such as NISA (Level 3). At all levels of thermal analysis, parametric studies are an important part of thermal analysis and are used to assist the engineer in making proper decisions for design and construction.

b. *Thermal analysis levels.*

(1) Level 1 analysis. This type of analysis is the least complex. It is a simplified or “quick and dirty” methodology, using little or no laboratory testing, and incorporating broad assumptions for site conditions and placement constraints. The approach is to estimate the worst reasonable combination of material properties and site conditions, so that if conditions are acceptable, no further analysis is necessary. If conditions are not acceptable, then more accurate data and possibly a more detailed

Table A-1
Thermal Study Process

Data Collection	Temperature Analysis	Cracking Analysis
Levels 1-3	Levels 1-3	Levels 1 and 2
• Determine Ambient Conditions	• Prepare Temperature Model	• Determine Restraint
Climatological Conditions Foundation Temperature Water Temperatures Solar Radiation	Compute Surface Heat Transfer Coefficients and Other Boundary Conditions Establish Calculation Increments Prepare FE Model (mesh) or Prepare Step-By-Step Method (spreadsheet)	Compute *Kf* and *Kr* for: Mass Gradient Analysis Surface Gradient Analysis
• Determine Material Properties		• Determine Thermal Strains
Concrete Foundation		Strain = $(C_{th})(\Delta T)(Kr)$ for: Mass Gradient Analysis Surface Gradient Analysis
• Determine Construction Parameters	• Compute Temperature Histories	• Estimate Cracking
Geometry/Lift Height Lift Placement Rate Concrete Placement Temperature Concrete Postcooling Construction Start Date(s) Formwork and Insulation Usage	Mass Gradient Analysis: Determine Peak and Ultimate Stable Temperatures Surface Gradient Analysis: Determine Temperature History at Surfaces Determine Depth of Tensile Zone for K_R	Mass Gradient Cracking: Use Mass Gradient Strain & Slow Load TSC Surface Gradient Cracking: Use Surface Gradient Strains & Age-Modified TSC
		Level 3 - NISA
		FE Method: ABAQUS w/ ANACAP-U
		• Conclusions & Recommendations

analysis are necessary. Temperature calculations are limited to simple determinations of peak concrete temperature based on summation of placement temperature and temperature rise produced by heat from the concrete mixture. Cooling from the peak temperature is assumed to progress to the ambient average annual temperature or a cyclic temperature range. Strain, length change, and cracking are computed based on temperature change in the MCS from peak to average ambient, using simple methods for determination of restraint. Other MCS loading conditions are evaluated separately from the thermal analysis at this level. A detailed description of a Level 1 thermal analysis using average monthly temperatures is shown in Annex 2.

(2) Level 2 analysis. Level 2 thermal analysis is characterized by a more rigorous determination of concrete temperature history in the structure and the use of a wide range of temperature analysis tools. Placement temperatures are usually determined based on ambient temperatures and anticipated material processing and handling measures. The temperature history of the concrete mass is approximated by using step-by-step iteration using the Schmidt or Carlson methods or by FE analysis using simple 1-D models, termed "strip" models, or using 2-D models representing cross sections of a structure. Evaluation of thermal cracking within the interior of an MCS, termed mass gradient cracking, and cracking at the surface of MCS, termed surface gradient cracking, are appropriate at this level. Detailed cracking evaluation of complex shapes or loading conditions other than thermal loads is not performed at this level.

(3) Level 3 analysis. Engineer Technical Letter (ETL) 1110-2-365 describes the computational methodology and application of Level 3 (NISA) analysis. ETL 1110-2-536 presents an example of NISA application to the Zintel Canyon Dam. NISA is performed using the FE method, exclusively, to compute incremental temperature histories, thermal stress-strain, stress-strain from other loading, and

cracking prediction results. Significant effort is necessary to collect environmental data, assess and implement applicable construction parameters, acquire foundation materials properties, determine appropriate construction scenarios, and perform testing required for thermal and nonlinear material properties input. Preparation of FE models and conducting temperature and thermal stress analyses which generate significant volumes of data are generally extensive and costly efforts.

c. *Parametric studies.* A parametric study is a rationally planned set of analyses used to gain a better understanding of thermal performance through the identification and understanding of the effects that critical parameters have on the structure. The effects of a parameter on the structure can be determined by varying that parameter in a set of analyses while holding the other parameters constant. Likely candidates for a parametric study are, but are not limited to, determination of the critical material properties, critical lift sequence or configuration, construction start time, insulation requirements, and placement temperatures. Results from single analyses within the parametric study should be interpreted separately to gain an understanding of the thermal response in each analysis. Then comparisons of results from each analysis in the parametric study can be made and the influence of each parameter identified. Once identified and documented, results and conclusions from parametric studies can be used in subsequent thermal analysis phases. For example, assume a goal of a current thermal study is to reduce construction costs through relaxing controls on concrete placement temperatures. A parametric study is devised, permitting only the lift placement temperature to vary. Results are analyzed, and the highest acceptable placement temperature is selected for subsequent use.

d. *Coordination.* A design team consisting of structural, materials, geotechnical, cost, and construction engineers should be established prior to performing a thermal analysis. Interdisciplinary coordination is essential to ensure that the analysis is based on reliable concrete and foundation properties and realistic construction techniques. The structural, materials, and construction engineers should predict an appropriate set of construction conditions (e.g., time between lifts, lift heights, type of formwork, formwork removal, construction start date, insulation requirements, etc.) which will approximate actual field conditions and which can be adequately modeled. Concrete properties should be provided for the proposed concrete mixtures by the materials engineer. The structural and geotechnical engineer should develop appropriate foundation material properties. The engineer should obtain the monthly average ambient air temperatures and other climatological information. The engineer must ensure that the specified parameters are properly modeled for the numerical analysis. The engineer performing the thermal analysis may be the materials engineer or the structural engineer, depending on the structure and expertise available in the design organization.

A-3. Data Collection

a. *General.* Data collection for the thermal analysis includes acquiring information on ambient weather conditions, concrete properties, foundation properties, and construction parameters. The following are descriptions of these data requirements. Data needs and acquisition costs should always be measured against the level of thermal analysis and requirements of the analysis.

b. *Ambient environmental conditions*. Environmental parameters, including air temperatures, wind, impounded water, and solar radiation can affect cracking in mass concrete.

(1) Climatological conditions. The ambient temperature conditions and variations from ambient temperature during the course of a year at a construction site will affect the need and extent of temperature controls implemented to reduce thermal cracking. The effects of the annual ambient temperature cycle on placement temperatures, short-term and long-term cooling rates, foundation temperatures, and potential starting dates for construction must be considered. Weather data can be acquired from National Oceanic and Atmospheric Administration (NOAA) summaries, from airport or other local weather stations, or from project weather

stations. NOAA data are available on average daily, monthly, and annual temperatures, maximum and minimum daily and monthly average temperatures, humidity, precipitation, and wind velocity. Ambient temperature data will also be used in the computation of concrete placement temperatures. Depending on the project site location, site weather conditions may depart significantly from even local weather stations, necessitating some judgement in weather data usage, and/or some project collection of site-specific data. Adjustments of data from the nearest recording stations to the site can be used to estimate site temperatures. For every 76 m (250 ft) of elevation increase, there is about a 0.5-deg C (1 deg F) decrease in temperature. To account for a positive 1.4-deg lattitude change, temperatures can be reduced 0.5 deg C (1 deg F). Temperature cycles used in thermal analysis may include:

- *A normal annual temperature cycle* is a sinusoidal-like variation of temperatures for a locale obtained from multiyear daily average temperatures.

- An *extreme ambient temperature cycle* can also be used. The extreme ambient temperature cycle can be developed as a sine wave with a 1-year period which captures the coldest and hottest of the extreme monthly average temperatures. The extreme ambient temperature is used to account for the possibility of seasons (months) having much higher or lower temperatures than the average ambient conditions based on multiyear averages.

- *Daily temperature cycles* may be used in areas where daily temperature variation can be 28 deg C (50 deg F) or more. Extreme daily temperature variation can cause significant surface temperature gradients.

The effects of cold fronts may cause significant cracking within an MCS and should be considered when evaluating the MCS. This winter protection evaluation is required mainly to assess the need, duration, and R-value for possible insulation of the structure. Cold fronts have not been commonly included in thermal studies due to their sporadic and unpredictable occurrences. Yet, they do occur and are commonly the cause of cracking during construction. The design team must use the thermal analysis results coupled with experience and engineering judgement to develop the final requirements for insulation during construction.

(2) Water temperatures. The presence of impounded water is generally not necessary in thermal studies, because water impoundment generally occurs long after construction. When needed for unusual analyses, the temperature of the water can be assumed to have an annual variation and may have little variation with great depth. Nearby similar projects are the best source of data.

(3) Solar radiation. The effects of solar radiation during and following construction have often been ignored in thermal analyses. Some thermal analyses have incorporated an increase in ambient temperature of 0.5 to 1.0 deg C (1 to 2 deg F) to account for solar radiation heating of concrete surfaces during construction. EM 1110-2-2201 and ACI 207.1R provide charts allowing approximate estimates of solar radiation effects. Due to the approximate nature of Level 1 analyses, solar radiation should be ignored for Level 1 analysis.

c. *Concrete properties*. Concrete thermal, mechanical, and physical properties needed for thermal analysis are defined and discussed below. These concrete properties are dependent upon the materials used and upon the proportions of these materials in the concrete mixture. Many of these properties are time- and temperature-dependent. Some of the properties will be determined by laboratory testing and some will be assigned by the engineers. Properties that are determined in laboratory tests should be representative of concrete mixtures containing project specific materials. The test data should be included in the concrete materials documentation. When testing of actual concrete mixtures is not possible, data can be acquired from published data in ACI documents, technical publications, and engineering handbooks, and from prior laboratory testing. Consultation with materials engineers is essential for determining all of the following properties. Variations in material properties due to scatter of test data, differences in behavior of the material between actual and that predicted by the numerical model, and expected differences between the laboratory mixture and the actual mixture used during construction can be accounted for by performing parametric studies using combinations of the upper and lower bound values of critical properties. Drying shrinkage is generally ignored for analysis of thermal cracking, except for possible application to surface gradient cracking. Test methods identified as ASTM are American Society for Testing and Materials, Philadelphia, PA, methods. Test methods identified as CRD-C (Concrete Research Division-Concrete) are Corps of Engineers methods found in the Handbook for Concrete and Cement published by the U.S. Army Engineer Waterways Experiment Station (WES) (1949). Test methods identified as RTH (Rock Testing Handbook) are Corps of Engineers methods found in the Rock Testing Handbook (USAEWES 1990). Concrete materials and properties are discussed in EM 1110-2-2000, EM 1110-2-2200, EM 1110-2-2201, and ACI Committee 207 documents.

(1) Concrete thermal properties. ACI reports 207.1R, 207.4R, and 207.5R, many WES published thermal studies, and others listed in the related references provide a wide range of laboratory determined concrete thermal properties.

(a) Adiabatic temperature rise ($T_{ad)}$. An adiabatic system is a system in which heat is neither allowed to enter or leave. The adiabatic temperature rise, therefore, is the change in temperature in concrete due to heat of hydration of cement under adiabatic conditions. It is the measure of heat evolution of the concrete mixture in a thermal analysis. In very large masses of concrete, temperatures near the center of the mass will peak near the sum of the placement temperature and the adiabatic temperature rise. Nearer the surface of the placement, the peak temperature will be lower and will be near ambient air temperature. The magnitude of the adiabatic temperature rise and the shape of the curve can vary significantly for different concrete mixtures. Adiabatic temperature rise is determined according to CRD-C 38 (USAEWES 1949). If testing is conducted, generally only for large projects, the concrete mixture tested should be representative of the mixture proportions and constituent materials that will be used for the project. The placement temperature for the test should represent the temperature at which the bulk of concrete is likely to be placed for the MCS. Typical values for adiabatic temperature rise for mass concrete range from 11 to 19 deg C (20 to 35 deg F) at 5 days to 17 to 25 deg C (30 to 45 deg F) at 28 days. For projects where adiabatic temperature rise tests can not be justified, generic adiabatic temperature rise curves in ACI 207.1R can be used. These curves can also be used to develop parametric adiabatic temperature rise curves for use in thermal analysis.

(b) Specific heat (c). Specific heat is the amount of heat required per unit mass to cause a unit rise of temperature. It is affected by temperature changes but should be assumed to be constant for the range of temperatures in MCS. Specific heat is determined according to CRD-C 124 (WES 1949). For mass concrete mixtures, specific heat is not substantially affected by age. Typical values for specific heat of mass concrete range from 0.75 kJ/kg-K (0.18 to 0.28 Btu/lb-deg F).

(c) Thermal diffusivity (h^2). Thermal diffusivity is a measure of the rate at which temperature change can occur in a material and is the thermal conductivity divided by the product of specific heat and unit weight. It is determined according to CRD-C 36 (WES 1949) for concrete with up to 75-mm (3-in.) nominal maximum aggregate size and CRD-C 37 (WES 1949) for concrete with larger nominal maximum aggregate size and is usually conducted between ages of 7 and 28 days. For mass concrete, thermal diffusivity is not substantially affected by temperature or age. Diffusivity is influenced by aggregate type and concrete density. Diffusivity is directly input to the Carlson and Schmidt methods. Thermal diffusivity is used to calculate thermal conductivity used for FE analysis. Typical values for thermal diffusivity of mass concrete range from 0.003 to 0.006 m^2/hr (0.03 to 0.06 ft^2/hr).

(d) Thermal conductivity (K). Thermal conductivity is a measure of the ability of the concrete to conduct heat and is defined as the rate at which heat is transmitted through a material of unit area and thickness when there is a unit difference in temperature between the two faces. For concrete, thermal conductivity is calculated from the product of thermal diffusivity, specific heat, and density according to CRD-C 44 (WES 1949). Thermal conductivity of mass concrete is not significantly affected by age or by changes in temperature over typical ambient temperature ranges but is influenced by aggregate type. Typical values for thermal conductivity of mass concrete range from 1.73 to 3.46 W/m-K (1 to 2 Btu/ft-hr-deg F).

(2) Concrete mechanical and physical properties. Tests and descriptions of concrete mechanical and physical properties used in thermal studies are described below. Test programs to develop these data can be relatively expensive. Modulus of elasticity, creep, and, to some degree, tensile strain capacity are difficult to estimate without testing. When laboratory tests cannot be performed, the best approach is to use results of more easily performed laboratory tests in conjunction with published information for similar concrete materials and mixtures from other projects.

(a) Modulus of elasticity (E_c). The modulus of elasticity is defined as the ratio of normal stress to corresponding strain below the proportional limit. For practical purposes, only the deformation which occurs during loading is considered to contribute to the strain in calculating the instantaneous modulus of elasticity. Subsequent strain due to sustained loading is referred to as creep. The modulus of elasticity is a function of the degree of hydration and is time and strength dependent. The temperature dependence of the modulus of elasticity is negligible for the range of temperatures of concern in MCS and is ignored. The modulus of elasticity is determined according to CRD-C 19 (WES 1949), which is described as a "chord" modulus. Three other methods of modulus measurement are seen in the literature. Hence, for critical analyses, the engineer may need to determine which method has been used when using published data. Generally, the differences between the methods is small compared to the overall variations in material properties and uncertainties in thermal analysis. ACI formulas for the modulus are not based on mass concrete mixtures and are generally not accurate estimates of mass concrete modulus. To model the time dependency of the modulus of elasticity, tests should span the duration of analysis. Test ages of 1, 3, 7, 28, 90, 180, and possibly 365 days, as well as at the design age, may be considered. Modulus of elasticity of mass concrete is about 6.9 GPa (1×10^6 psi) at 1 day, and ranges from about 21 to 38 GPa (3 to 5.5×10^6 psi) at 28 days, and from about 30 to 47 GPa (4.3 to 6.8×10^6 psi) at 1 year. Tensile E_c is assumed to be equal to the compressive E_c. Sustained modulus of elasticity (E_{sus}) includes the results of creep and can be obtained directly from creep tests by dividing the sustained load on the test specimen by the total deformation. ACI 207.4R includes values of instantaneous and E_{sus}. E_{sus} for tests conducted on specimens loaded at early ages for a period of 1 year will be about one-half that of the instantaneous E_c. E_{sus} for tests conducted on specimens loaded at 90 days or later ages for a period of 1 year will be a slightly higher percentage of the instantaneous E_c. Early age creep information is more important for thermal studies.

(b) Creep. Creep is defined as time-dependent deformation (strain) due to sustained load. Specific creep is creep under unit stress or strain per MPa (psi). Creep results in an increase in strain, but at a continually decreasing rate, under a state of constant stress. Creep is closely related to the modulus of elasticity and compressive strength of the concrete and is thus a function of the age of the concrete at loading. Concrete with a high modulus of elasticity will generally have relatively low creep. Creep is determined according to CRD-C 54 (WES 1949). Creep tests for mass concrete should always be conducted with sealed specimens. So called "drying creep" testing is not appropriate for mass concrete. The test method recommends five ages of loading between 2 days and 1 year to fully define creep behavior. For Level 2 FE thermal analysis, creep data are generally used only in surface gradient analysis, thus, loading ages should span the time during which surface gradients are developing. Loading ages of 1, 3, and 14 days are generally adequate. Creep is not generally used in Level 1 thermal analysis. The effects of creep can be considered by using the sustained modulus of elasticity of the concrete measured during the period of surface gradient development.

(c) Tensile strain capacity (ϵ_{tc}). Tensile strain capacity is the change in length per unit length that can be sustained in concrete prior to cracking. This property is used with the results of temperture analysis to determine whether an MCS will crack and the extent of cracking. Tensile strain capacity is discussed in detail in Annex 1. Tensile strain capacity is time-and rate-of-loading dependent and is strongly dependent on strength. Tensile strain capacity tests are conducted on large concrete beams instrumented to measure strain to failure for strain-based cracking analysis. Tensile strain capacity is determined according to CRD-C 71 (WES 1949).

(d) Tensile strength (F_t). Tensile strength may be used with the results of stress-based thermal analysis to determine if cracking is probable in an MCS. ACI 207.2R discusses tensile strength in some detail. Tensile strength can be measured by several methods, including the splitting tensile method (CRD-C 77 (WES 1949)), direct tension (CRD-C 164 (WES 1949)), and by the flexural test or modulus of rupture method (CRD-C 16 (WES 1949)). The splitting tensile test is more commonly run for mass concrete, due to the simplicity of the test, and because it can be less sensitive to drying than other tests. All tensile strength tests are age dependent, load rate dependent, and moisture content dependent. Prediction of tensile strength based on compressive strength is generally not particularly reliable. For preliminary thermal analysis, the split tensile strength relationship to compressive strength is discussed in ACI 207.2R.

(e) Coefficient of thermal expansion (C_{th}). The coefficient of thermal expansion is the change in linear dimension per unit length divided by the temperature change. The coefficient of thermal expansion is determined according to CRD-C 39 (WES 1949). The value of this property is strongly influenced by the type and quantity of coarse aggregate in the mixture and is not dependent on age or strength. Typical values for the coefficient of thermal expansion for mass concrete range from 5 to 14 millionths/deg C (3 to 8 millionths/ deg F).

(f) Autogenous volume change. Autogenous volume change, commonly called "autogenous shrinkage," is a decrease in volume of the concrete due to hydration of the cementitious materials without the concrete gaining or loosing moisture. This type of volume change occurs in the interior of a large mass of concrete and can be a significant factor. Autogenous shrinkage occurs over a much longer time than drying shrinkage, the shrinkage due to moisture loss that affects only thinner concrete members or a relatively thin layer of the mass concrete near the surface. Although no specific test method exists, autogenous shrinkage can be determined on sealed creep cylinder specimens with no load applied in accordance with CRD-C 54 (WES 1949).

(g) Density (ρ). Density is defined as mass-per-unit volume. It is determined according to CRD-C 23 (WES 1949). Typical values of density for mass concrete range from 2,240 to 2,560 kg/m^3 (140 to 160 lb/ft^3).

d. Foundation properties. The thermal, mechanical, and physical properties of the foundation are dependent on the type of soil or rock, the moisture content, and any discontinuities in the foundation. In situ properties may vary significantly from those obtained from laboratory testing of small samples obtained from borings or test pits. Rock may exhibit anisotropic properties. Exact thermal properties are seldom necessary for the foundation materials, and adequate values for use in a thermal analysis may be obtained from Jumikis (1977) or Kersten (1949). Likewise, exact mechanical properties are not required, and adequate values can be estimated from foundation test data or from Hunt (1986). The structural and geotechnical engineers should jointly select foundation properties based on any in situ properties available and varied based on information from the above referenced texts and past experience.

(1) Thermal properties of foundation rock.

(a) Specific heat (c_{fdn}). Specific heat varies within a narrow range of values. Specific heat for soil foundations ranges from 0.80 kJ/kg-K (0.19 Btu/lb-deg F) for sand to 0.92 kJ/kg-K (0.22 Btu/lb-deg F) for clay. Specific heat for foundation rock generally ranges from 0.80 to 1.00 kJ/kg-K (0.19 to 0.24 Btu/lb-deg F). Specific heat can be determined according to CRD-C 124 (WES 1949).

(b) Thermal conductivity (K_{fdn}). The thermal conductivity of the foundation material is affected by density and moisture content and the degree of jointing and fracture in rock. The thermal conductivity of foundation materials may range from 4.15 W/m-K (2.4 Btu/ft-hr-deg F) for clay, to 4.85 W/mm-K (2.8 Btu/ft-hr-deg F) for sand, to 5.19 W/m-K (3.0 Btu/ft-hr-deg F) for gravel, and can range from 1.73 to 6.23 W/m-K (1 to 3.6 Btu/ft-hr-deg F) for rock. Thermal conductivity can be determined according to one of several applicable ASTM procedures.

(c) Diffusivity (h^2). Diffusivity of the foundation is direct input to the Carlson and Schmidt step-by-step temperature analysis methods and is sometimes assumed equal to the concrete diffusivity for simplicity. Diffusivity is influenced by material type, rock type, and density. Typical values for thermal diffusivity of rock range from 0.003 to 0.006 m^2/hr (0.03 to 0.06 ft^2/hr). Rock diffusivity can be determined according to CRD-C 36 (WES 1949), or may be calculated according to CRD-C 158 (WES 1949), using test values of thermal conductivity, specific heat, and density.

(2) Mechanical and physical properties of foundation rock.

(a) Modulus of elasticity (E_{fdn}). The modulus of elasticity of foundation materials varies greatly with the grain size, moisture content, and degree of consolidation for soil, and with the degree of jointing and fracture of a rock foundation. Adequate values can be estimated by the geotechnical engineer. Values for foundation rock can be determined by ASTM D 3148; typical values from intact small specimens range from 28 to 48 GPa (4 to 7 × 10^6 psi) for granite and between 14 to 41 GPa (2 to 6 × 10^6 psi) for limestone.

(b) Coefficient of thermal expansion ($C_{th\text{-}fdn}$). The coefficient of thermal expansion for soil foundations is not needed for thermal analysis. The coefficient of thermal expansion for rock foundations can be determined according to ASTM D 4535. The coefficient can vary widely based on rock type; typical values can be found in the references. Measurements have been recorded ranging from 0.9 to 16 millionths/deg C (0.5 to 8.9 millionths/deg F).

(c) Density and moisture content. The density and moisture content of the foundation material must be determined by the geotechnical engineer.

(d) Initial temperature. For Levels 1 and 2 thermal analyses, the initial temperatures for the foundation may be assumed to be at the annual average temperature at the site.

e. *Construction parameters*. Differences in the way an MCS is constructed will impact the behavior of the structure significantly. The response of the structure to changes of the construction parameters in the analysis will often dictate whether or not cost reducing measures can be taken in the field. Construction parameters can also be varied in an attempt to improve the performance of a structure. The paragraphs below describe the primary construction parameters that can be considered for changes during the thermal analysis for accomplishing cost reductions or improved behavior. Values for the following parameters, depending on the level of thermal analysis, must be selected by the design team prior to the initial analysis. The requirements for construction parameters in a Level 1 analysis are minimal. Levels 2 and 3 thermal analyses depend on specific data regarding the construction operation.

(1) Geometry. The geometry of the structure is a major factor in the thermal behavior of the structure. This information includes section thickness, monolith length, and location and size of section changes such as galleries or culverts. A Level 2 or 3 thermal analysis should not be performed until the structural geometry is at a stage where only minor changes to the geometry are expected. A change in

the geometry will generally require some type of revision to the temperature analysis model.

(2) Lift height. Since the heat escape from a mass is inversely proportional to the square of its least dimension and since the height of a lift will usually be the smallest dimension, the height of a lift can become an important factor in the thermal behavior of an MCS. Lift heights to be used in initial analyses will typically be selected by the engineer based on previous experience and practical limits. If the initial analyses indicate that the behavior of the structure is satisfactory, then analyses may be performed with increased lift heights as a measure for reducing cost. Likewise, if results indicate unacceptable behavior, a decrease in lift height may be considered to alleviate problems in the structure.

(3) Lift placement rate. The time between the placement of lifts has an effect on the thermal performance of the structure due to the insulating effect a new lift has on the previous lift(s). The time between placement of lifts must be included in the thermal analysis. Usually, shorter time intervals between lifts, i.e., higher placement rates, cause higher internal temperatures in an MCS. A 5-day interval between lift placements is typically assumed for traditional concrete. For RCC, the time interval will depend on the placing rate anticipated and the lift surface area, which often varies during construction. The longer the interval between placement of lifts, the longer each lift will have to dissipate the heat that has built up within the lift. When considering the aging characteristic of concrete, however, longer placement intervals may not be desirable, since the previous lift will be much stiffer than the new lift providing more restraint to the new lift. Lift placement interval can have an effect on the construction cost if the change increases the length of the contract.

(4) Concrete placement temperature. For many mass concrete structures, the temperature of the concrete at the time of placement is limited to control the temperature level within the mass due to the heat of hydration, as well as to control temperature at the MCS surface. Without control measures implemented, concrete placement temperature is a function of the annual ambient temperature cycle. In thermal analysis, the placement temperature is the starting point for concrete temperature rise. Placement temperatures are affected by concrete constituent materials temperatures, heat added or lost due to ambient conditions, and heat added or lost from material processing and handling. The placing temperature for the initial analysis should be established by the materials engineer. As with lift heights, if behavior is acceptable then consideration may be given to increasing the placing temperature. Increasing the allowable placing temperature can lead to cost savings due to decreased cooling requirements. EM 1110-2-2201 and ACI 207.4R contain information and guidance on precooling of mass concrete.

(5) Construction start date. The time of year when construction is started can have a significant effect on the MCS temperatures. The selection of start dates is structure and site dependent and should be evaluated by the design team based on past experience and engineering judgement. The objective in selection of start dates is to chose those among the possible start dates that may produce the worst conditions in the MCS. Usually a single start date is inadequate for identifying the worst conditions at most locations within the structure, especially since the structure is built in lifts over a significant period of time. Different start dates may yield temperature problems at different locations in the MCS. The start dates should be chosen to create the largest temperature gradients. Often a start date representing each of the four annual seasons is selected for preliminary analysis.

(6) Formwork. Removal times of formwork must be established for Levels 2 and 3 thermal analyses, because the insulating effects of formwork can significantly affect surface temperature gradients and surface cracking. This information is used to calculate the surface heat transfer coefficient, a thermal boundary condition for surface gradient thermal analysis.

(7) Insulation. Insulation of the concrete during cold weather may be necessary during construction and, if used, must be accounted for in the analysis. The time that insulation is in place and the amount

of insulation (the *R* value) to be used will depend on the project location and should be selected by the engineer for the initial analysis. Both of these parameters may be varied during subsequent analyses to achieve cost savings or to improve performance. The effects of insulation are included in the surface heat transfer coefficient calculations.

(8) Postcooling. Embedded cooling coils to control heat generation within an MCS have been used in some large gravity and arch dam projects, as well as some smaller specialized placements such as tunnel plugs (to shorten time for joint grouting), but have typically not been needed on navigation-type structures. Postcooling of mass concrete is very costly in terms of both installation and maintenance and has seldom been used in recent years. If placing temperatures have been reduced to their lower limit, lift heights have been reduced to a practical minimum, and temperatures within the structure remain excessive, then the addition of cooling coils may be considered. Because postcooling is so seldom used, it's use is not included in the thermal analysis procedures. Guidance on postcooling is provided in EM 1110-2-2201 and in ACI 207.1R.

(9) Reinforcement. Reinforcement is generally not used in the MCS being analysed for thermal concerns but may be used in smaller structures such as powerhouses and large foundations. Since excluding reinforcing from an analysis provides conservative results, initial analyses can be performed without the effects of reinforcement. The effects of reinforcing on resulting structural behavior are small, if no cracking occurs, but if cracking does develop, modeling of the reinforcement can be very beneficial for control of the cracking. ACI 207.2R provides information on thermal analysis and reinforcement.

(10) Roller-Compacted Concrete (RCC). Techniques and design of RCC structures are discussed in EM 1110-2-2006 and ETL 1110-2-343. Although concrete placement using RCC is fundamentally different than traditional mass concrete placement, similar construction parameters are used for thermal analysis, although the individual numbers may differ.

A-4. Analytical Methods For Temperature Calculation

All thermal studies require computation of temperature or temperature distribution changes in a structure. Depending upon the type and function of a structure, less rigorous thermal studies may be adequate for "acceptable" evaluation of thermal performance. Temperature calculation requirements for thermal studies may range from very simple to reasonably complex. ACI 207.1R discusses several approximate methods that are appropriate for simple evaluations. The Carlson (Carlson 1937) and Schmidt (Rawhouser 1945) methods are step-by-step integration techniques, adaptable to spreadsheet solutions on personal computers, that can be used for computing temperature gradients when 1-D heat flow and reasonably simple boundary conditions are assumed. FE programs for computing temperatures (Wilson 1968; Polivka and Wilson 1976; Hibbitt, Karlsson, and Sorensen 1994) are appropriate for thermal studies when aspects of the analysis exceed the capabilities of simpler methods or when application of the FE method is as easy to implement as the simple methods. The following are descriptions of the range of analytical methods that can be used for Levels 1 and 2 thermal analyses.

a. Simple maximum and final temperature calculations. This "quick and dirty" method is used to compute peak temperatures due to heat of hydration and final stable temperature in the MCS. Computation usually results in a conservative approximation of peak temperature. Peak temperature is simply the sum of the placing temperature and the adiabatic temperature rise of a concrete mixture less heat (+ or -) due to ambient conditions. The structure cools over a long period of time to a stable temperature dependent primarily on annual ambient air temperature. In small structures, internal temperatures may not stabilize at a single temperature but at a temperature cycle dependent upon the annual air temperature cycle. Computation of temperature variation in an MCS as a function of depth and ambient temperature cycle is discussed in ACI 207.1R. This method is appropriate for a Level 1 analysis and is described in Annex 2.

b. Heat dissipation methods. The time required for dissipation of heat and the resultant cooling of MCS can be calculated by the use of heat loss charts or by simple computation as described in ACI 207.1R for solid bodies, such as slabs, cylinders, and spheres. These charts provide an approximate method of calculating the time for the concrete to cool from a peak temperature to some stable temperature. Peak concrete temperature must be determined using other means. Strain and resultant cracking analysis must also be performed by other methods. These heat dissipation methods can be of use in Level 1 analyses.

c. Step-by-step integration methods.

(1) Carlson method. The Carlson method is a step-by-step integration method for determining temperature distribution in a concrete structure. Carlson (1937)(Department of the Interior, U.S. Bureau of Reclamation (USBR) 1965) provides detailed discussions for implementing this method. It is readily adapted to modern computer spreadsheet computations and provides reasonable approximations of temperature distributions in simple structures. Properly applied, this method permits modeling of incremental construction, heat flow between dissimilar materials such as foundations and concrete, and adiabatic temperature rise of concrete. This method can be used in Level 2 analysis.

(2) Schmidt method. The Schmidt or Schmidt-Binder method is one of the earliest computation methods for incrementally determining temperature distributions in a structure. Rawhouser (1945), ACI 207.1R, and USBR (1965) provide comprehensive and illustrated discussions of the method. Although most easily adapted for 1-D heat flow, the simplicity of this method permits adaptation to 2-D and three-dimensional (3-D) thermal analysis. Because of the iterative approach, the method is time-consuming when performed manually. Especially when used in 1-D analyses, this method is easily adapted to modern computer spreadsheet computations. This method also provides for incorporating internally generated heat into the process. The Schmidt Method can be used in Level 2 analyses.

d. FE methods. An FE analysis can be described as a numerical technique for the determination of temperature distribution or stress analysis in which structures are mathematically represented by a finite number of separate elements, interconnected at a finite number of points called nodes, where behavior is governed by mathematical relationships. All the boundary conditions are then applied to the model, including material thermal properties, ambient conditions, and construction schedule. The model is run, and a temperature history for the model is generated. Temperature is calculated for specified times for each node. The FE method is the preferred methodology for computing temperatures in mass concrete structures. Information on building a data file to run an FE analysis must be obtained from manuals provided by the developer of the FE code being used. To use the FE method, an FE model must first be prepared. The model is divided into a grid of finite elements in which element boundaries coincide with material interfaces, lift interfaces, and structural boundaries. Generally, smaller elements are used in areas of greatest thermal gradient. The methodology permits detailed modeling of virtually all applicable parameters. Few FE programs have been written to compute temperature histories modeling incremental construction of MCS. Few, if any, programs have been written to model solar gain on lift surfaces. ETL 1110-2-332 and ETL 1110-2-254 provide guidance on FE analysis.

(1) One of the earliest FE temperature analysis computer programs was developed by Wilson (Wilson 1968) for the U.S. Army Engineer District, Walla Walla, followed by an improved version (Polivka and Wilson 1976). Temperature histories using such programs have compared very favorably with actual measured temperatures. These programs were written to support incremental construction thermal analysis, and they are reasonably easy for new users familiar with FE analysis to implement.

(2) More recently, the U.S. Army Corps of Engineers has developed user-defined subroutines to supplement ABAQUS (Hibbitt, Karlsson, and Sorensen 1994), a modern, general-purpose FE program. ABAQUS is used with associated user-supplied subroutines DFLUX and HETVAL for modeling heat generation in incremental construction thermal analyses, with user subroutine UMAT, or with the ANACAP-U subroutine to implement a time-dependent material/cracking model for thermal stress analysis of MCS. ABAQUS has been used to perform Level 3 NISA and is the basis for ETL 1110-2-365. ABAQUS can also be readily used for performing temperature calculations for Level 2 analyses, especially by experienced ABAQUS users. This program requires a high level of computer experience and expertise, as well as an advanced computer.

A-5. Temperature Analysis

a. *General*. This section provides general methodology for MCS temperature analyses conducted at Levels 1 and 2, once objectives have been developed, input data has been collected, a parametric analysis plan has been prepared for the temperature analysis, and a method of temperature analysis has been selected. Since the FE method is widely used for determination of temperature distribution histories in thermal analyses of MCS, a description of required FE thermal model development is also presented. The information is generic in that it is not directed for use by a specific FE program.

b. *Levels and methods of temperature analysis*. Methods of temperature analysis for each level of analysis are described below.

(1) Level 1 temperature analysis.

(a) Simplified peak temperature analysis. Temperature analysis at this level involves only very basic hand calculations to determine approximate peak temperature and ultimate operating temperature of the MCS. Peak temperature is the sum of the placing temperature and the adiabatic temperature rise of a concrete mixture and a correction for heat lost or gained due to ambient conditions. Peak temperature in most MCS is higher than the average ambient temperature. Thus, the structure cools over a long period of time to a stable temperature equal to the average ambient air temperature. This very simple analysis usually estimates temperatures higher than actual peak temperatures. The exception may be for very hot climates where the peak temperature may be higher than estimated. For small or relatively thin structures, internal temperatures can be assumed to stabilize at an average annual temperature cycle. Computation of temperature variation in smaller MCS as a function of depth and ambient temperature cycle is discussed in ACI 207.1R, including a figure for determining temperature variation with depth. A step-by-step procedure and example of this level of analysis is included in Annex 2.

(b) Heat dissipation methods. Using the above type of peak temperature analysis, simple computations or heat loss charts may be used to evaluate the time required to cool simple mass concrete structures from the peak temperature. The use of heat loss charts is described in detail in ACI 207.1R.

(2) Level 2 temperature analysis. Temperature analyses for Level 2 thermal studies may be implemented in two types of analytical methods, namely, step-by-step integration methods or FE methods.

(a) Step-by-step temperature integration methods. The Carlson (Carlson 1937)(USBR 1965) and Schmidt (USBR 1965) methods of temperature analysis are tabular methods of computing approximate temperature distribution in a structure that can be adapted to modern computer spreadsheets. These similar methods provide temperature distributions that are sufficiently accurate for many noncomplex structures. The methods are limited to temperature distribution; other methods must be used to determine cracking as a result of the temperature distribution. Field measurements have confirmed the validity of these methods for simple structures. The methods divide the concrete into "space intervals," computing the temperature after the completion of one time interval, then computing another temperature after the next time interval, and so on. Time and space intervals are chosen to meet certain criteria, ensuring validity of model assumptions. Using tabular techniques, the tables essentially solve a large number of simultaneous equations, resulting in progressive temperature distribution. The computations require the structure dimensions, ambient temperature, the temperature distribution at some initial time, the material diffusivity, and the adiabatic temperature rise. The methods will accomodate the presence of forms and insulation, if desired. These methods can be used effectively for parametric analysis of thermal conditions. Although these methods are effective temperature analysis techniques for structures with simple geometry and conditions, current FE analysis computer software often allows development of FE temperature analysis with about the same level of effort to perform a step-by-step analysis.

(b) FE models. Due to the ease in creating and using FE models for temperature analysis, FE methodology is preferred for a Level 2 thermal analysis and is required for a Level 3 analysis. Level 3 temperature analysis is NISA, described previously, and is not covered further in this document. Even when 2-D or 3-D FE analysis is used for the final thermal analysis, 1-D FE analysis can be a productive screening tool for parametric analyses.

- 1-D strip models. In many larger structures, a model consisting of a "strip" or "line" of elements oriented within the transverse section of a monolith can be used to provide reasonably accurate temperature distributions without complete modeling of the section. The strip is a 1-D heat flow representation. The strip may represent the vertical temperature distribution that models incremental construction used in mass gradient cracking analysis. Horizontal strips produce temperature distributions that may be used to evaluate temperatures for surface gradient cracking. The Schmidt and Carlson Methods may be implemented for these calculations, if a desk-top computer and spreadsheet software are available. Otherwise, an FE code which employs or can be adapted for incremental construction capability is recommended. The FE method provides the best modeling of construction parameters and boundary conditions characteristic of mass concrete construction. A step-by-step procedure and example of this level and type of analysis is included in Annex 3.

- 2-D full-section models. Thermal analysis with full-section models must be performed with one of the FE programs which employs or can be adapted for incremental construction capability. A 2-D, FE model representing 2-D heat flow in an appropriate section(s) of a monolith is used. More complex structure geometry, materials properties, construction parameters, and boundary conditions are used in these analyses. The results of a Level 2 full-section 2-D temperature analysis are temperature distributions in the entire plane of the monolith that was modeled. These data are used as the basis for more refined mass gradient and surface gradient analyses anywhere in the model. A step-by-step procedure and example of this level and type of analysis is included in Annex 3.

- 3-D-full section models. These more complex FE models can be used for MCS with complex geometry and may develop into NISA models.

c. *FE thermal analysis considerations*. Information on developing FE temperature analysis models follows.

(1) FE mesh. Conventional FE modeling techniques apply to most temperature analyses. The meshes comprising the model should be adequately fine to describe 1-D or 2-D heat flow appropriate for 1-D strip or 2-D full-section analysis. ETL 1110-2-332 provides relevant information for modeling MCS for FE analysis. A 1-D strip mesh for vertical temperature distribution and a 2-D full-section mesh must both account for incremental construction by lifts. The meshes should include a depth of foundation so that the lowest elevation remains at the constant foundation temperature for the locale. This is usually 2 to 9 m (10 to 30 ft) depending upon the thermal conductivity of the foundation and size of the structure. Horizontal

strip meshes entirely contained in one lift usually extend from the surface to the middle of the monolith. Lift boundaries and boundaries between different concrete mixtures or other materials must only exist at element boundaries. Various programs are available that may be used to provide preprocessing capabilities in developing a mesh. If a decision is made to use a preprocessor, users should select a preprocessor which is fully compatible with the FE program and with which they are familiar or feel they can learn easily. Element aspect ratios should follow ETL 1110-2-365 recommendations, and element size will generally depend on geometry and temperature gradients. Time increments must be small enough to capture early age temperature changes that occur more rapidly than later cooling, with 0.25 day often used.

(2) Surface heat transfer coefficients. Surface heat transfer coefficients (film coefficients) are applied to all exposed surfaces to represent the convection heat transfer effect between a fluid (air or water) and a concrete surface, in addition to the conduction effects of formwork and insulation. The following equations are taken from the American Society of Heating, Refrigerating and Air Conditioning Engineers (ASHRAE) (1977). These equations may be used for computing the surface heat transfer coefficients to be included in any of the FE codes for modeling convection.

(a) For surfaces without forms, the coefficients should be computed based on the following:

$$\begin{aligned}&\text{for } V > 17.5\ km/h\ (10.9\ mph):\\ &h = aV^{b}\ W/m^{2}\text{–}K\ (\text{Btu/day–in.}^2\text{-deg F})\end{aligned} \tag{A-1}$$

$$\begin{aligned}&\text{for } V < 17.5\ km/h\ (10.9\ mph):\\ &h = c + d(V)\ W/m^{2}\text{–}K\ (\text{Btu/day–in.}^2\text{-deg F})\end{aligned} \tag{A-2}$$

where

a = 2.6362 (0.1132)

b = 0.8 (0.8)

c = 5.622 (0.165)

d = 1.086 (0.0513)

h = surface heat transfer coefficient or film coefficient

V = wind velocity in km/h (mph)

The wind velocity may be based on monthly average wind velocities at the project site. Data can be obtained for a given location and then generalized over a period of several months for input into the analysis.

(b) If forms and insulation are in place, then the values for h computed in the equations above should be modified as follows:

$$\begin{aligned}h' &= \frac{1}{\left(\frac{b}{k}\right)_{\text{formwork}} + \left(\frac{b}{k}\right)_{\text{insulation}} + \left(\frac{1}{h}\right)}\\ h' &= \frac{1}{R_{\text{formwork}} + R_{\text{insulation}} + \left(\frac{1}{h}\right)}\end{aligned} \tag{A-3}$$

where

h' = revised surface heat transfer coefficient

b = thickness of formwork or insulation

k = conductivity of formwork or insulation

R_{formwork} = R value of formwork

$R_{\text{insulation}}$ = R value of insulation

(3) Foundation temperature stabilization. Foundation temperatures at the start of a vertical strip thermal analysis or a 2-D thermal analysis must be defined. The temperature distribution in the foundation for the start of concrete placement can be determined by performing a thermal analysis

on the foundation for an arbitrary time period up to 1 year immediately preceding the construction start date(s). The time period selected is usually a function of the depth of foundation in the model. During this analysis, the lower boundary of the foundation is fixed at the stable foundation temperature, usually mean, annual air temperature. The foundation surface is exposed to the normal, annual ambient temperature cycle. Appropriate adjustments should be made for possible surface thermal conditions during the analysis period, such as snow cover or very hot weather.

(4) Output interpretation. This section is intended to give insight into the various methods that have proven useful in presentation of analysis results. The engineer must sufficiently process results to comprehend the behavior of the structure and provide the necessary data (plots, diagrams, tables, etc.) to support cracking analysis and conclusions based on this understanding.

(a) Temperature contours. Temperature contours should be smooth throughout a lift and across lift interfaces. Temperature contours should never abruptly intersect free surfaces of the model where surface heat transfer coefficients are applied, except for locations where a very low coefficient is used to model an enclosed void. This indicates the application of an incorrect thermal boundary condition. Contour plots of temperature, stress, net strain, and/or crack potential are useful in selecting zones in the structure for more detailed investigation.

(b) Time-history plots. Time-history plots of temperature, stress, and strain results at a single location or multiple points across a section of significance are useful in showing the response of that location throughout the time of the analysis. These are useful in determining the critical material property combination when parametric analyses are performed. To assist reviewers and persons unfamiliar with the model, a locator section is often provided to show the location in the model where the results are presented. Selection of locations for presentation of time-history results may be determined from contour plots, the determination of locations of maximum values of results, or locations of particular interest. The latter may be places where similar structures have experienced problems, places where previous analyses have presented results, or places which help explain the overall response of the structure.

(c) Section plots. Plots of results (i.e., stress, temperature, net strain) across a specified section or location at a specific time are useful in determining the behavior of the section or location. Determination of the maximum value of a specific result (i.e., stress, strain) and its time of occurrence is useful in determining which section or location to plot and the corresponding time.

A-6. Cracking Analysis

a. General. The ability of concrete to resist thermal cracking is dependent on the magnitude of the thermal shrinkage or volume change, the degree of restraint imposed on the concrete, and the tensile strain capacity of the concrete. This section discusses restraint in MCS that leads to strain in the concrete mass or near the MCS surface and possible cracking if the tensile strain capacity of the concrete is exceeded. Strain due to other loading conditions oftens needs to be considered with thermal strain to evaluate cracking potential. The consequences of cracking may be structural instability, seepage, durability, and maintenance problems or may be relatively inconsequential, depending on the MCS design and function. Depending on the orientation of cracking, sliding or overturning stability of a structure may be impaired. Typically, transverse cracking in a gravity dam does not directly affect stability. However, such cracking may affect assumptions concerning uplift by allowing reservoir water under pressure into the interior of the dam along cracks and lift joints. Longitudinal or diagonal crack orientation can separate a dam into separate, unstable sections. Thermal shock, when warm mass concrete is suddenly subjected to much colder temperature, can cause significant surface cracking and occasionally can contribute to cracking in the concrete mass. This can occur with the removal of forms or the filling of a deep reservoir with cold runoff. Abrupt, large drops in temperature at the concrete surface can create steep temperture gradients, leading to high strains and stresses at the surface, and result in cracking if the tensile capacity of the concrete exterior is exceeded.

b. *Thermal volume change.* Volume change in MCS is primarily due to cement hydration heat generation and subsequent cooling. However, additional volume change may result from autogenous shrinkage or other mechanisms. Volume change for analysis of thermal cracking is normally discussed in terms of 1-D length change and is determined by multiplying the coefficient of thermal expansion by the effective temperature change induced by cooling of the mass concrete from a peak temperature. This is discussed further under mass gradient and surface gradient cracking subjects below. If concrete is unrestrained, it is free to contract as a result of cooling from a peak temperature, no tensile strain is induced, and it will not crack. However, since most MCS are restrained to some degree, tensile strain is generally induced, leading to cracking if tensile strain capacity is exceeded.

c. *Restraint in mass concrete*. Cracking in mass concrete is primarily caused by restraint of volume change. Restraint that prevents free volume change or contraction after mass concrete has reached a peak temperature and cools to an ultimate temperature is of primary concern in mass concrete structures. Restraint prevents the free volume change of concrete, which causes tensile strain and stress in the concrete. Restraint may be either external or internal, corresponding to mass gradient and surface gradient strain-stress, respectively. ACI 207.2R discusses restraint in some detail.

(1) Mass gradient restraint. Mass gradient or external restraint is caused by bond or frictional forces between the MCS and its foundation, by underlying and adjacent lifts, or by other portions of a massive concrete section. The degree of external restraint depends upon the relative stiffness of the newly placed concrete, the restraining material, and the geometry of the section. Large variations in mass or thickness which cause abrupt dimensional changes in a structure, such as wall offsets, culvert valve shafts, gallery entrances, and offsets, induce external restraint of volume change that has resulted in cracking. The foundation or lower lift is viewed as a restraining surface, with high strain-stress at the restraining surface, decreasing with increasing distance from that surface.

(2) Surface gradient restraint. Surface gradient, or internal restraint, is caused by changes in temperature within the concrete. This condition exists soon after placement when heat loss from the surface stabilizes the temperature of near-surface concrete, while the temperature of interior concrete continues to rise due to heat of hydration. This temperature gradient also continues later, when the temperature of the surface concrete cools more rapidly than interior concrete. These temperature gradients result in relatively larger volume changes (temperature shrinkage) at the surface relative to the interior. The result is strain-stress at the surface, shown in Figure A-1, decreasing in magnitude with increasing distance from the surface to eventually a zero strain-stress region at some point in the interior. Strain is generated nearer the surface because the adjacent more interior concrete is changing volume at a slower rate. This is sometimes described as the interior concrete "restraining" the exterior concrete. As can be seen in Figure A-1, the interior is not "restraining" the surface as the foundation "restrains" an MCS, since the strain-stress buildup due to surface gradients is at the surface, not in the interior. The restraint formulas used for mass gradient strain calculation are also applied to surface gradient restraint strain calculation, with some differences. In this case, no "restraining" surface exists at the interior. Rather, a point of zero strain-stress exists in the interior, with increasing strain-stress as the concrete surface is approached. The thermal strain important for surface gradient analysis is the net or effective strain due to temperature change at the surface relative to the temperature change in the interior of the mass.

d. *Types of thermal cracking*. The analysis of thermal cracking can be categorized by two general types: mass gradient cracking and surface gradient cracking.

(1) Mass gradient cracking. Mass gradient cracking is generally caused by classical external restraint, discussed previously and in ACI 207.1R. Mass gradient cracking is described as cracking that occurs when the tensile strains of the mass exceed

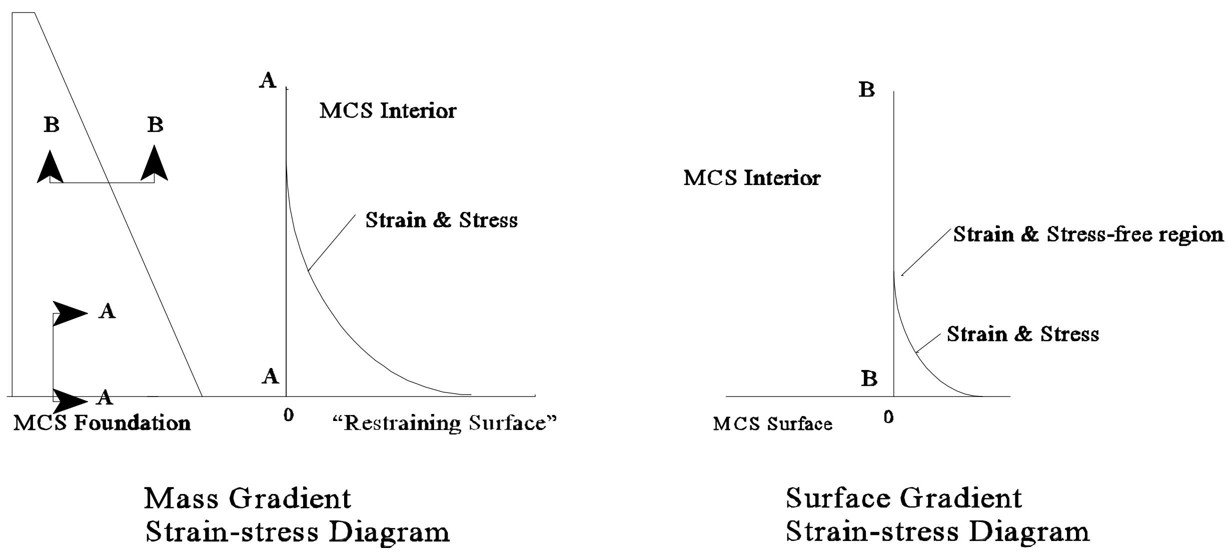

Figure A-1. Mass and surface gradient strain-stress "model" comparison

the tensile strain capacity of the concrete. The orientation of the cracking, if fully developed, can separate the structure into discrete sections. In some cases, cracking in a dam that occurs normal to the monolith joints could affect the stability of a monolith. In dams where monoliths are very wide, this cracking can be longitudinal or parallel to the axis of the dam. This procedure for analysis of external restraint mass gradient cracking is based upon ACI 207.2R, which can be adapted for a stress-or a strain-based methodology, as seen in the two examples at the end of this appendix.

(2) Surface gradient cracking. When the surface of a structure cools faster than the interior, a temperature gradient exists from the interior to a maximum at or near the surface. This causes a gradient of tensile strain and stress and can cause cracking at the exterior surface. It may also cause tension to develop or reduce the compression across lift joints. Surface cracking may not cause great concern if cracking is localized, but it cannot be assumed that cracking will be localized. Once cracks are initiated, the energy required to propagate cracks is much less than the energy required to initiate a crack. Surface gradient cracking is observable on concrete surfaces as pattern cracking and often extends into the structure from a few inches to several feet. This problem is less prevalent in temperate climates and more exaggerated in locations with greater temperature variations. However, under some circumstances, this cracking can lead to more serious cracking conditions. Thermal shock can induce steep surface temperature gradients leading to cracking. This occurs when warm concrete surfaces are suddenly subjected to considerably lower air or water temperatures, creating steep surface temperature gradients and potential cracking. This can occur when wooden or insulated forms are removed during periods of cold weather. Since steel forms provide less insulation, the concrete surface may be near ambient temperatures already when forms are removed, hence causing smaller surface gradients. Sudden cold fronts can also generate steep surface gradients, potentially causing cracking. The procedure for analysis of internal restraint surface gradient cracking in this ETL is based upon ACI 207.2R and can be adapted to a stress- or a strain-based methodology, as seen in the examples at the end of the appendix.

(3) Mass/surface gradient interaction cracking. Cracking may not occur due to mass or surface gradient cracking alone. However, if the mass has built up significant mass gradient tensile strains and stress near the threshold of cracking, the additional tensile strain or stress from surface gradients may propagate a crack through the mass. Additionally, other loading, such as hydrostatic pressures from a reservoir, temperature effects from unusually cold water in deep reservoirs, or differential settlement of the foundation, may propagate a surface crack through the structure.

(4) Longitudinal cracking. Longitudinal cracking has long been a concern for large dams, since the occurrence of significant longitudinal cracking has the potential to affect the stability of the dam. In traditional dam construction, precooling and postcooling techniques were used to eliminate this concern. With the predominance of RCC in the construction of dams, longitudinal cracking is again a concern for large dams. This is due to the high cost and difficulty with using postcooling in RCC. Hence, precooling of the materials is the primary method of controlling RCC temperature. In large dams, those methods may not be sufficient to prevent longitudinal cracking.

e. *Mass gradient cracking analysis*. Although strain is used as a basis for the following cracking analyses and is the recommended approach, stress has been historically and can still be used to evaluate cracking. The principle of superposition of incremental strains or stress is assumed to apply to these cracking analyses. This means that each increment of strain or stress generated by each incremental change in temperature gradient can be added to each other to determine the total thermal strain or stress at any given time. The following equation may be used to determine the strain due to mass thermal gradients in concrete (ACI 207.2R):

$$\epsilon = (C_{th})(dT)(K_R)(K_f) \qquad \text{(A-4)}$$

where

ϵ = induced strain-millionths

C_{th} = coefficient of thermal expansion-millionths/deg C (millionths/deg F)

dT = temperature change in the mass concrete causing strain - deg C (deg F)

K_R = structure restraint factor

K_f = foundation restraint factor

(1) Mass gradient restraint factors. A concrete mass is commonly restrained by the foundation, other structures, or by previous lifts. Full restraint seldom exists in a structure and then, only at very specific locations. The induced strain in a structure can be calculated using the restraint formula, modified by factors based upon the geometry and relative internal stiffness of the structure, K_R, and upon the relative stiffness of the structure compared to the foundation, K_f.

(a) Structure restraint factor (K_R). The structure restraint factor is determined by Equations A-5 and A-6 from ACI 207.2R. The restraint model (Figure A-2) is a representation of the external restraint geometry which is applied to mass gradient cracking due to foundation restraint. It relates the

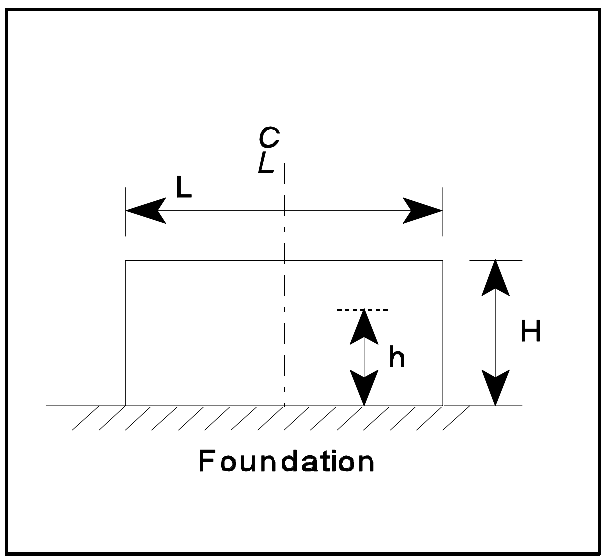

Figure A-2. External restraint model used in mass gradient analysis

magnitude of restraint to the shape of a simple structure where L is length, H is height, and h is the distance from the restraining interface (or restraining plane) at the base of the structure to any location of interest where strain is to be determined. L should be selected with care, since some large structures may be susceptible to mass gradient cracking in more than one direction. This model provides for a structure restraint factor, K_R, for external restraint at locations, h, away from the restraining plane. K_R is determined by one of the following two equations:

for L/H greater or equal to 2.5

$$K_R = \left(\frac{\frac{L}{H} - 2}{\frac{L}{H} + 1} \right)^{h/H} \qquad \text{(A-5)}$$

and for L/H less than 2.5

$$K_R = \left(\frac{\frac{L}{H} - 1}{\frac{L}{H} + 10} \right)^{h/H} \qquad \text{(A-6)}$$

These formulas from ACI 207.2R are reasonable approximations of figures shown in ACI 207.2R, but Equation A-6 is a somewhat inaccurate representation of the ACI figures for values of L/H approaching 1.0, where $h/H > 0.6$. For $L/H \leq 1.0$, of course, the formula breaks down and cannot be used.

(b) Foundation restraint factor (K_f). A second factor for induced mass gradient strain is provided by K_f, the foundation restraint or multiplication factor, used to modify K_R. This factor accounts for the difference in the elasticity of the foundation compared to the elasticity of the concrete mass. This relationship is expressed as:

$$K_f = \frac{1}{1 + \frac{A_g E_c}{A_f E_f}} \qquad \text{(A-7)}$$

where

A_g = gross area of concrete cross section at foundation plane

A_f = area of foundation or zone restraining contraction of concrete (recommended maximum value is 2.5 A_g).

E_f = modulus of elasticity of foundation or restraining element

E_c = modulus of elasticity of mass concrete

f. Surface gradient cracking analysis. Cracking due to temperature gradients from the relatively stable interior temperatures to the exterior of an MCS is analyzed based on the restraint model described below and in ACI 207.2R. This model is similar in nature to that used for mass gradient cracking analysis. Although strain is used as a basis for the following cracking analyses, and is the recommended approach, stress has been historically and can still be used to evaluate cracking. The principle of superposition of incremental strains or stress is assumed to apply to these cracking analyses. This means that each increment of strain or stress generated by each incremental change in temperature gradient can be added to each other to determine the total thermal strain or stress at any given time. Figure A-3 illustrates the concept of surface gradient analysis.

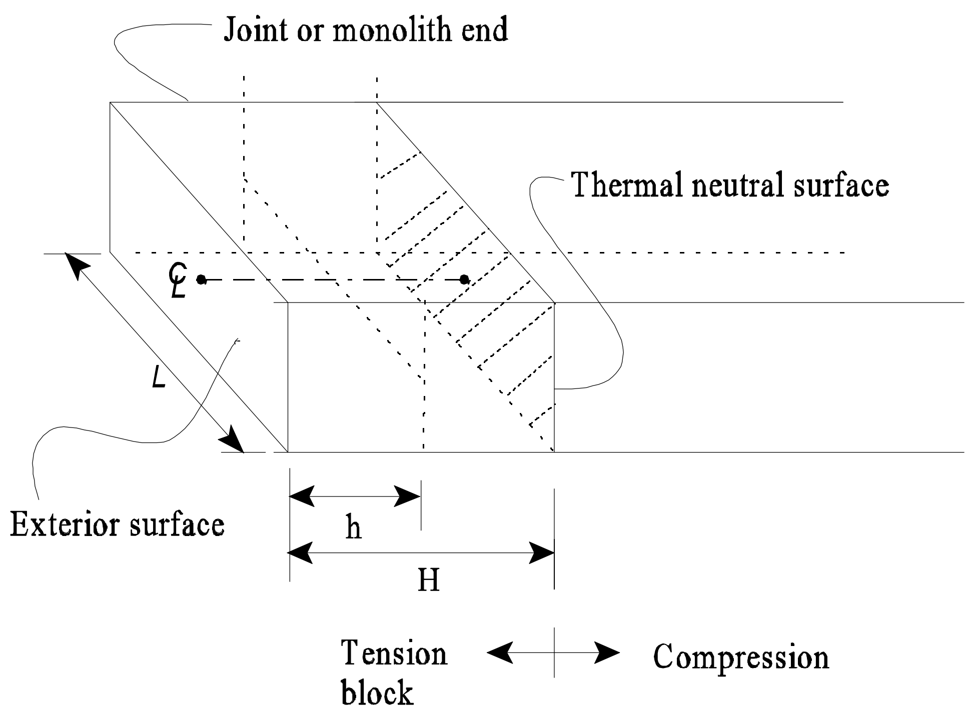

Figure A-3. Internal restraint model used in surface gradient analysis

The following equation may be used to determine the strain due to surface thermal gradients in concrete (based on ACI 207.2R):

$$\epsilon = (C_{th})(dT)(K_R) \qquad \text{(A-8)}$$

where

ϵ = induced tensile strain (millionths)

C_{th} = coefficient of thermal expansion - millionths/deg C (millionths/deg F)

dT = temperature difference with respect to interior temperature difference - deg C (deg F)

K_R = internal restraint factor

Determination of K_R and dT are described in the following.

(1) Surface gradient restraint factor. The degree of restraint is not easily determined but can be estimated based on the thickness of the exterior surface layer being restrained. The restraint factor, K_R, is computed in a manner similar to mass gradient restraint factor, from Equations A-5 or A-6 depending upon the value of L/H, where L is the monolith width (between joints or between ends of the monolith) and H is the distance from the interior strain and stress-free surface (thermal neutral surface) to the exterior surface, as shown in Figure A-3:

for L/H greater or equal to 2.5

$$K_R = \left(\frac{\frac{L}{H} - 2}{\frac{L}{H} + 1} \right)^{h/H} \qquad \text{(A-5bis)}$$

and for L/H less than 2.5

$$K_R = \left(\frac{\frac{L}{H} - 1}{\frac{L}{H} + 10} \right)^{h/H} \qquad \text{(A-6bis)}$$

Values of L/H less than 2.5 will rarely be applied for surface gradient analysis, since the surface gradient tensile region can be visualized as a flat slab lying along the exterior surface, with large L and small H. Values of K_R may be determined at various distances, h, from the interior surface of zero strain-stress, to determine restraint at specific locations. A maximum value of K_R = 1.0 will always exist at the exterior surface.

(2) Determining temperature gradients, the surface gradient tension block and H. Surface gradient strain computations are performed using temperature differences, dT, which is the temperature change at the point of interest in the mass minus the temperature change in the interior. These temperature differences represent the temperature gradient from the surface to the interior of the mass concrete that generates thermal strains and stresses. If the exterior and interior concrete underwent the same temperature change during initial temperature rise and later cooling, no surface gradient strains and stresses would be generated. The fact that the exterior and interior concrete undergo temperature changes at different rates gives rise to surface gradient strains and stresses. The starting temperatures for computing temperature differences are always the temperatures present when the concrete begins hardening and has measurable, but small, mechanical properties.

(a) The temperature differences determine the location of the thermal neutral surface (and "*H*") and are used to compute dT. Figure A-4 shows a graph of temperature differences distributed across a typical mass concrete lock wall characterized by surface concrete that is cooler than the interior concrete. Note the zero temperature difference at the exterior surface. This temperature difference distribution induces tension near the surface and compression in the interior concrete. ACI 207.2R states that for sectional stability, the summation of tensile stresses (and strains) induced by a temperature gradient in a cross section must be balanced by equal compressive stresses (and strains). Assuming that the modulus of elasticity and coefficient of thermal expansion are constant across the section and that stresses and strains are balanced, the implication is that temperature differences contributing to tensile and compressive strain must also be balanced.

(b) Figure A-5 shows the temperature differences from Figure A-4 adjusted to provide equal tension and compression in the section, providing a graphical representation of the surface gradient restraint model. This figure shows the locations of negative temperature differences relative to a thermal balance line at $\Delta T = 0$. Areas with negative temperature differences are in tension, corresponding to the tension block shown in Figure A-3. Areas with positive temperature differences are in compression. The location of $\Delta T = 0$ determines the location of the tension block relative to the exterior surface and the distance H for the K_R calculation. A variety of methods are used to determine the temperature differences, the tension block location, and H, some of which are shown in the examples in Annex 3.

(3) Determining dT. To calculate strain, dT must be determined for that location. dT is simply the temperature difference for that location of interest relative to the interior temperature difference where the tension and compression zones are balanced, or where ΔT=0 on Figure A-5.

g. *Cracking calculations*. To evaluate cracking, tensile strains are compared to tensile strain capacity of the concrete. Stress-based comparisons can be made in a similar way, but strain-based evaluations are usually preferred.

(1) General. To evaluate cracking of an MCS, the calculated tensile strains are compared with appropriate values of slow load ϵ_{tc} of the concrete. Where the ϵ_{tc} is exceeded, the portion of the tensile strains exceeding the ϵ_{tc} are distributed through the MCS section as cracks. If mass gradients induce

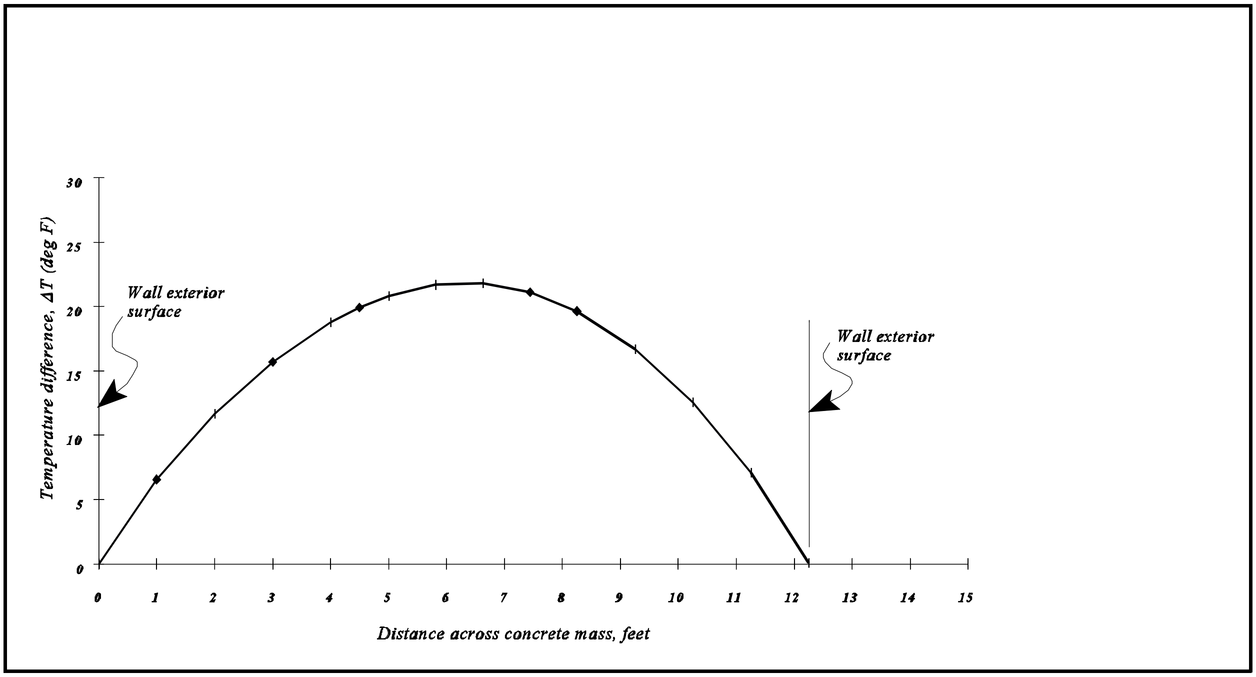

Figure A-4. Example of temperature difference distribution for surface gradient analysis of lock wall

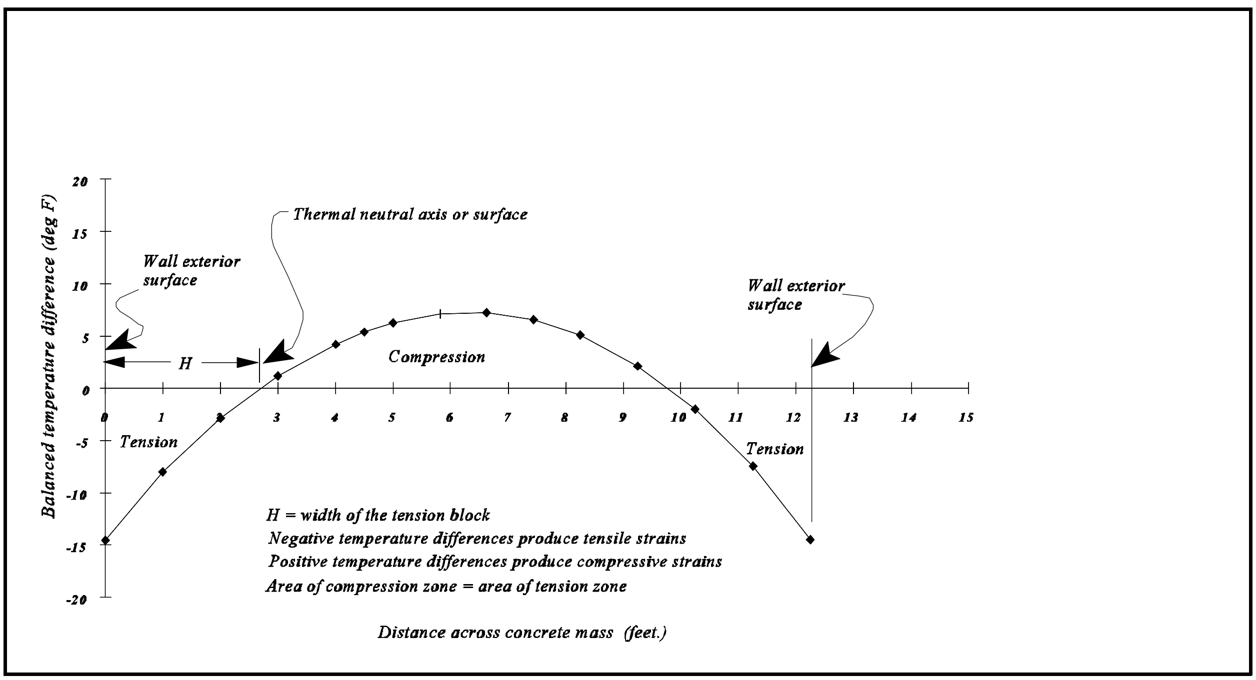

Figure A-5. Example of temperature balance computed from temperature differences in Figure A-4

strains in the mass above allowable ϵ_{tc} values, cracking of that mass is probable. This cracking is typically full cross section transverse cracking of the monolith. However, longitudinal cracking may also occur if the monolith is sufficiently large. If the surface gradient values exceed allowable ϵ_{tc}, surface cracking is probable. The spacing and widths of the cracks depend on restraint conditions and are determined based on judgement and experience.

(2) Cracking calculation. The thermal strain is distributed across the length of the analyzed section. Tensile strain capacity data from slow-loading tests are used to define the capacity of the concrete to "absorb" strain. For example, if a fully restrained dT temperature change occurred over 1 year:

dT = 17 deg C (30 deg F)

C_{th} = 9 millionths/deg C (5 millionths/deg F)

$K_R = K_f = 1$

Using Equation A-4,

$\epsilon_{\text{induced}} = (C_{th})(dT)(K_r)(K_f)$ = 150 millionths.

If

ϵ_{tc} = 100 millionths (when loaded from 7 to 365 days),

then the remaining strain to be distributed as cracks is

$\epsilon - \epsilon_{tc}$ = 50 millionths.

The remaining 50 millionths of strain is distributed into cracks totaling 15mm (0.6 in.) over a structure 305 m (1,000 ft) long.

Cracks = (305 m)(1,000 mm/m)(50 millionths)
= 15 mm

[Cracks = (1,000 ft)(12 in./ft)(50 millionths)
= 0.6 in.]

The example shown resulted in the length change distributed to cracks of 15 mm (0.6 in.). Based on experience, three to six cracks of 2-to 5-mm (0.1-to 0.2-in.) width might be expected, if no joints are installed and the fractured rock foundation is somewhat flexible.

(3) Crack spacing and width. Theoretically, there are an infinite number of combinations of crack spacing and crack widths that will equal a calculated thermal length change. However, there are some general rules of thumb for crack spacing and width based on experience. Foundation conditions of restraint often control the spacing of cracks, and the number of cracks tends to control the crack widths. Mass gradient crack spacing in large MCS usually ranges from 30 to 91 m (100 to 300 ft). Crack widths typically range from 2 to 5 mm (0.01 to 0.2 in.). Surface gradient cracking is highly dependent on the restraint conditions and is usually more closely spaced and narrower than mass gradient cracking. Surface gradient crack widths may range from 0.5 to 2 mm (0.02 to 0.1 in.) (Tatro and Shrader 1992). Hairline cracks of about 0.0005 mm (0.002 in.) may leak initially if water under pressure is available to one side of the crack, but will often heal from calcification. Such leakage is expected to stain the exposed concrete face.

A-7. Limitations of Thermal Studies

a. General. The analytical methods described in this ETL for Levels 1 and 2 thermal studies provide reasonable approaches to the analysis of thermal effects in mass concrete. These thermal analyses do not consider other loading conditions that may be present and that may contribute additional strain and stress leading to cracking. Good engineering judgement must be applied to evaluate the effects of additional loading conditions or of remnant thermal strains contributing to structural strains and stresses. The thermal models discussed in this ETL are based on a number of broad assumptions of conditions and behavior which generally lead to conservative analyses. Good engineering judgement must be applied to these analyses at all stages and levels of thermal evaluation.

b. Verification. All thermal analyses, particularly the temperature model, should be benchmarked or verified in some way to assure the engineer of the appropriateness and accuracy of the methods used. The design team must use every available means to verify the correctness and accuracy of the input data for thermal analysis, including climatological, structural, material, and construction input parameters. The design team should use any means available to help verify the validity of the results. Using the experience and judgement of the materials engineer, an initial check of the results can be made on a qualitative basis. Exploring previously analyzed structures and their results, performing a simple ambient condition analysis (no creep, shrinkage, aging modulus, or adiabatic temperature rise), and performing simplified analyses are all possible methods for providing confidence and a check on the validity of the analysis.

A-8. Documentation of Thermal Study Results

a. General. Thermal studies are performed during various phases of project design. Generally, Level 1 studies are performed during a feasibility study for a major project or for a complex structure where thermal cracking issues may require subsequent design changes and more complex analysis. Detailed thermal analysis is often performed during the feature design phase of the project. The format of the documentation will depend on the design stage and the level of thermal study.

b. Feasibility studies. The thermal study and results should be described in a section of the engineering appendix to the Feasibility Report and not in a separate report. The information should include input data such as geometry, FE model, material properties, parameter combinations, loads, ambient temperature, surface heat transfer coefficients, and other information. Plots of results should be included to illustrate the behavior of the structure. These plots could include temperature, stress and crack potential contours at critical times, plus temperature and stress time-histories at critical locations. There should be a narrative interpretation of the results. This should explain any potential for cracking, whether it is acceptable, what special design or construction procedure changes might be required, and what cost adjustment was made because of these changes.

c. PED studies. PED thermal studies results should be presented in a separate design report and should include a statement of objectives of the study, information on the model(s) used in the analysis, information on all input parameters, presentation of the model and analysis results, verification of the model and analysis results, and conclusions and recommendations for design and construction. Presentation of results is critical in providing the proper understanding of how the structure behaved and for supporting any conclusions or recommendations that will be made as a result of the thermal analysis. Results may be displayed in tables, graphs, contour plots, or color plots. Discussion of results should include cracking potential, acceptability of cracking, and possible corrective measures for thermal problems. The thermal model results must be verified in a manner that illustrates the validity of the model results, either through independent analysis, correlation with field data, or correlation with field experience. Conclusions and recommendations for improved performance or cost savings should be discussed in the thermal studies design report.

ANNEX 1: DETERMINATION OF TENSILE STRAIN CAPACITY

A1-1. Purpose

Tensile strain capacity (TSC) is the change in length per unit length that can be sustained in concrete prior to cracking. This property is used with the results of temperature analysis to determine whether a mass concrete structure (MCS) will crack and the extent of cracking. This annex describes testing to determine TSC, methods to estimate TSC, and methodology for its use in thermal analysis.

A1-2. Background

The Corps of Engineers introduced TSC testing of concrete several decades ago to provide a basis for evaluating crack potential for strain-based thermal studies of MCS (Houghton 1976). This property is also used to compare different aggregates and different concrete mix proportions in MCS. TSC varies primarily based on age, strength, aggregate type, shape, and texture. TSC tests are conducted on large concrete beams instrumented to measure strain to failure. TSC is determined in a series of tests, including rapid and slow loading of beams. The slow-load test was designed to simulate the strain conditions occurring in a mass concrete structure during long-term cooling. By conducting tests at several loading ages, TSC data can be used to evaluate mass gradient cracking resistance in a structure under long-term cooling. Surface gradients generally develop during the first several days or weeks after placement of concrete, particularly following the removal of insulated forms. Hence, strains due to surface gradients develop more rapidly than tested using the slow-load TSC test, and more slowly than a standard TSC test failed at a normal loading rate. This annex describes one method used to estimate TSC for surface gradient analyses.

A1-3. Description of Test Method

Tensile strain capacity is determined according to CRD-C 71 (WES 1949). The test method requires a minimum of three beams for each test, and generally a minimum of three tests is recommended for each test set to allow for variation in the test results. Rapid-load (0.28 Mpa/min)(40 psi/min) and slow-load (0.17 MPa)(25 psi/week) tests are usually conducted in test series consisting of three beam tests each. TSC test specimens are 300-mm by 300-mm by 1,680-mm-long (12-in. by 12-in. by 66-in.-long) beams tested in third-point loading. Strain gauges are located at or near the top and bottom (compression and tension) surfaces to measure strain during the tests. At the age of test, a rapid-load test is conducted and a slow-load test is begun. Loading continues at the prescribed rate until failure. During the slow-load beam test, strain measurements are made on the beam under load. In addition, measurements of autogenous strain are made on the third beam. The autogenous shrinkage strains are used to correct the strain measurements on the beam under slow load. Upon failure of the slowly-loaded beam, a rapid-load test is performed on the third beam. A TSC test series usually contains a suite of rapid- and slow-load tests typically initiated at 3, 7, 28 days, and/or other ages. The differences in TSC capacity from the slow- and rapid-load beams provide an indication of the cumulative creep strain during the slow-load test. The strains measured in the slow-load beam test containing both elastic and creep strains are expressed in millionths (1×10^{-6} in./in).

A1-4. Tensile Strain Capacity Test Results

TSC test results can vary widely depending on a variety of factors. Use of test results for the specific materials and mixture(s) to be used in an MCS should be used whenever possible. Actual values for TSC of mass concrete for slow-load tests for specimens loaded at 7 days and failing at about 90 days range from 88 to 237 millionths. Corresponding values for rapid-load tests conducted at 7 days range from 40 to 105 millionths. For tests conducted upon failure of the slow-load beam, rapid-load results range from 73 to 136 millionths. Ratios of slow-load tensile strain capacity to rapid-load tensile strain capacity tested at the same age as the slow-load specimens range from 1.0 to 2.0 and averages 1.4. This average is relatively insensitive to age.

A1-5. Use of Tensile Strain Capacity for Mass Gradient Cracking Analyses

Mass gradient tensile loading in an MCS occurs over an extended period of time. The standard slow-load tensile strain capacity test was specifically designed for this condition. Standard slow-load TSC tests provide a reasonable limiting strain in mass gradient cracking analyses for the condition of restrained slow loading of mass concrete which occurs in a slowly cooling mass. Using an appropriate loading time period, the slow-load tensile strain capacity can be used directly for mass gradient cracking analysis.

A1-6. Use of Tensile Strain Capacity for Surface Gradient Cracking Analyses

a. Surface gradient strains. Surface gradient strains can be initiated at a very early age, particularly after the removal of insulated formwork, and can develop over a few days or weeks of loading due to the initial temperature rise and subsequent development of the surface temperature gradient. Because loading under surface gradient conditions is more rapid than the standard tensile strain capacity slow-load test, the results of that test may not well represent surface gradient conditions. Very accurate tensile strain capacity values may not be necessary for surface gradient analysis, except for critical situations. For most situations, the standard test values will suffice for surface gradient cracking analysis as well as mass gradient cracking analysis. In some structures, concrete placed near the surface of the MCS may differ significantly from internal concrete mixtures. Tests for TSC used in surface gradient analysis should be conducted on the appropriate concrete mixture(s).

b. Simulated surface gradient strains. For critical situations, slow-load TSC tests conducted at more rapid rates of loading than the standard slow-load test may be conducted to simulate the development of surface gradient thermal strains. In lieu of such special load rate testing, an estimate can be made of TSC for use in preliminary surface gradient TSC determinations, using the ratio of 1.4 described above. An estimate of TSC for surface gradient analysis is determined by testing TSC at the rapid load rate and at the age of interest. This value is then multiplied by 1.4, to determine a TSC under the slow loading reflective of surface gradient strain development. This estimate is believed to be reasonably conservative at ages from 1 to 14 days. Because creep rates are greatest at early ages, it is possible that slow-load TSC may be considerably higher especially from 1 to 7 days. Until test data are available, this may be used for developing surface gradient tensile strain capacity values.

ANNEX 2: LEVEL 1 THERMAL STUDY MASS GRADIENT ANALYSIS PROCEDURE AND EXAMPLE

A2-1. Procedure

a. General. This Annex summarizes each step in a Level 1 thermal study mass gradient analysis of a mass concrete sheetware (MCS) and provides an example of how this procedure was applied for a modest-size MCS. Although alternative approaches can be used, this method is in common use for this level MCS thermal analysis. Surface gradient thermal analysis is seldom conducted at this level of analysis.

b. Input properties and parameters.

(1) Step 1: Determine ambient conditions. Simple analyses conducted for a Level 1 analysis are typically based on average monthly temperature data.

(2) Step 2: Determine material properties. Laboratory test results on material properties are seldom available for this level of thermal analysis. Material properties are generally estimated from published data in sources such as American Concrete Institute (ACI) documents, technical publications, and engineering handbooks. Often known information such as compressive strength and aggregate type is used to predict other material properties from published data. The minimum properties required are the coefficient of thermal expansion (C_{th}), the adiabatic temperature rise (ΔT_{ad}), and the tensile strain capacity (ϵ_{tc}).

(3) Step 3: Determine construction parameters. Concrete placement temperature is the essential construction parameter needed for this level of thermal analysis. A first approximation is to assume that concrete placement temperatures (T_p) directly parallel the average monthly temperature. A more accurate method is to modify the average monthly temperature based upon production time period and extent of production or to use actual placement temperature data from similar projects.

c. Temperature analysis.

(1) Step 4: Mass gradient temperature analysis. For Level 1 mass gradient analysis, no elaborate "model" is used to develop temperature history. The long-term temperature change is simply calculated as the peak concrete temperature minus the ultimate stable concrete temperature.

(a) Determine peak temperature. This is the sum of the concrete placement temperature and the adiabatic temperature rise.

(b) Determine ultimate stable temperature. Large structures cool to a stable temperature equal to the average ambient temperature. However, smaller concrete structures cool to a stable annual temperature cycle, since there is insufficient mass to provide complete insulation of the interior. ACI 207.1R provides a figure relating temperature variation with depth to determine this internal temperature cycle. It is assumed that the concrete temperature cycles about the average annual temperature.

(c) Determine long-term temperature change. The sum of the placing temperature plus adiabatic temperature rise provides a quick peak temperature of the MCS. Then subtracting the ultimate stable temperature provides the long-term temperature change used for strain and cracking evaluation.

d. Cracking analysis.

(1) Step 5: Mass gradient cracking analysis. Using long-term temperature change and ACI formulas, mass gradient strain is approximated. These strains are compared to estimates of tensile strain capacity to determine if and when cracking may occur.

(a) Determine mass gradient restraint conditions. The structure restraint factor (K_R) and the foundation restraint factor (K_f)(in ACI 207.2R termed "Multiplier for foundation rigidity") are determined as described in Appendix A, and in ACI 207.2R.

(b) Determine mass gradient thermal strain. The total induced strain is the product of the long-term temperature change, the coefficient of thermal expansion and restraint factors. Use Equation A-4 (Appendix A).

$$\text{Total strain} = (C_{th})\,(dT)\,(K_R)\,(K_f) \qquad \text{(A-4bis)}$$

where

Total strain = induced strain (millionths)

C_{th} = coefficient of thermal expansion

dT = temperature differential

K_R = structure restraint factor

K_f = foundation restraint factor

Cracking strain is computed by subtracting tensile strain capacity from the total strain. The remainder is the strain that must be accomodated in cracks at some spacing and width across the MCS.

(c) Estimate mass gradient cracking. Foundation conditions (restraint) control the spacing of cracks and the crack width. If the foundation is stiffer, tightly spaced cracks of small width can be expected. If the foundation is relatively soft (low restraint), widely spaced and wider cracks can be anticipated. Multiply the MSC length by the cracking strain to determine the total width of cracking to be accomodated in the MCS. Estimate a crack width based on foundation conditions and divide the total width of cracking by the assumed crack width to determine the total number of cracks.

e. Conclusions and recommendations. These typically include expected maximum temperatures for starting placement in different seasons, expected transverse and longitudinal cracking without temperature or other controls, recommended concrete placement temperature limitations, anticipated concrete precooling measures, need for adjustment in concrete properties, joint spacing, and sensitivity of the thermal analysis to changes in parameters.

A2-2. Example

a. Introduction. This example, based on a thermal study for the Cache Creek Detention Basin Weir, illustrates one way to estimate concrete placing temperature based on ambient air temperatures and material processing schemes and schedules. The study evaluates mass gradient cracking only. The Cache Creek Detention Basin in California is a roller-compacted concrete (RCC) overflow weir section in a levee system. The structure is 8 m (15 ft) high, 3.6 m (12 ft) wide at the top, has 0.8 to 1 slopes upstream and downstream, and is 530 m (1,740 ft) long. Compacted sands and silts were placed against the full height of the upstream face. The purpose of the study was to determine the adequacy of contraction joints spaced at 30-m (100-ft) intervals and, if necessary, provide recommendations for alternate configurations. Also addressed is the adequacy of a maximum placing temperature of 29 deg C (85 deg F) for the RCC. The following paragraphs provide explanation on the selection criteria and determination of the parameters used to summarize thermal study.

b. Input properties and parameters.

(1) Step 1: Determine ambient conditions. Data were provided from climatological data summaries for Woodland, CA, prepared by the National Oceanic and Atmospheric Administration (NOAA), shown in Table A2-1. The average annual temperature used was 16.1 deg (61 deg F), and monthly mean and average monthly maximum and minimum temperatures were used for other computations.

(2) Step 2: Determine material properties.

(a) Coefficient of thermal expansion. Coefficient of thermal expansion was estimated using handbook data (Fintel 1985) for the sandstone and

Table A2-1
NOAA Temperature Data, Woodland, CA

Month	Monthly avg. max. - deg C (deg F)	Monthly avg. min. - deg C (deg F)	Monthly avg. -deg C (deg F)
Jan	11.7 (53)	2.8 (37)	7.2 (45)
Feb	15.5 (60)	4.4 (40)	10.0 (50)
Mar	18.9 (66)	5.5 (42)	12.2 (54)
Apr	23.3 (74)	7.2 (45)	15.0 (59)
May	27.8 (82)	10.0 (50)	18.9 (66)
Jun	32.2 (90)	12.8 (55)	22.8 (73)
Jul	35.5 (96)	13.9 (57)	25.0 (77)
Aug	34.4 (94)	13.3 (56)	23.9 (75)
Sep	32.2 (90)	12.2 (54)	22.2 (72)
Oct	26.1 (79)	9.4 (49)	17.8 (64)
Nov	18.3 (65)	5.5 (42)	11.7 (53)
Dec	12.2 (54)	2.8 (37)	7.8 (46)
Annual	-	-	16.1 (61)

meta-sandstone aggregate concrete planned for the project:

C_{th} = 9.9 millionths/deg C (5.5 millionths/ deg F)

(b) Adiabatic temperature rise. The study was performed using an RCC mixture with a Type I/II cement content of 119 kg/m^3 (200 lb/cy) and a Class F pozzolan content of 39 kg/m^3 (66 lb/cy). ACI 207.1R suggests that pozzolan can be assumed to have a heat generating capacity about one-half that of cement. Using ACI 207.1R adiabatic temperature rise curves and an equivalent cement content of 138 kg/m^3 (233 lb/cy), this mixture should produce an adiabatic temperature rise of about 22.2 deg C (40 deg F). From ACI 207.1R:

Δt_{ad} for 223 kg/m^3 (376 lb/cy) cement at 28 days = 36.1 deg C (65 deg F)

Δt_{ad} for 138 kg/m^3 (233 lb/cy) equiv. cement at 28 days = (36.1 deg C)(138)/(223) = 22.2 deg C (40 deg F)

(c) Tensile strain capacity. ACI 207.5R suggests that values of tensile strain capacity ranging from 50 to 200 millionths are achievable for early age, slow-load testing. Lean RCC mixes typically range from 60 to 90 millionths. Since the cement content of 119 kg/m^3 (200 lb/cy) is higher than most lean RCC mixes and the coarse aggregate is crushed, a value of 80 millionths was selected.

(3) Step 3: Determine construction parameters. RCC placing temperature was calculated using the average annual temperature modified by rule-of-thumb temperature effects during construction, as shown in Table A2-2. In Table A2-2, the placing temperature is the composite temperature of the aggregate source, (assumed to be the average annual temperature), plus the added heat during aggregate production, plus the added heat during RCC production. Stockpile aggregate temperatures are the base temperature, plus the ambient addition, plus crushing and production energy. Similarly, RCC production temperatures are the stockpile temperature plus ambient additions and mixer energy additions. The ambient temperature additions are calculated as 0.67, an empirical correction factor, times the differential temperature of the aggregates and the air. The complete thermal study is summarized in Table A2-3. A May placing temperature was used for following calculations:

T_p = 18.9 deg C (66 deg F)

c. Temperature analysis.

(1) Step 4: Mass gradient temperature analysis.

(a) Determine peak temperature. This is the sum of the initial RCC placement temperature and the adiabatic temperature rise:

$T_p + \Delta T_{ad}$ = 18.9 + 22.2 = 41.1 deg C (106 deg F)

(b) Determine ultimate stable temperature. Since the weir is a relatively thin MCS, it is expected to develop a stable temperature cycle, rather than a single stable temperature as in larger MCS's. The temperatures below were determined using the methodology in ACI 207.1R ("Temperature variation with depth"). Typical distance from the RCC surface to the interior was determined to be 4.6 m (15 ft). From ACI 207.1R figure:

$$\frac{\text{Temp change through concrete}}{\text{Temp range at surface}} = 0.24$$

Temp range at surface = 24.8 - 7.3 = 17.5 deg C (31.5 deg F)

Temp change in concrete interior = (0.24)(17.5 deg C) = 4.2 deg C (7.6 deg F)

Temp range in concrete interior = 16.2 ± 4.2 deg C (61.1 ± 7.6 deg F)

T_{min} = minimum interior concrete temp. = 16.2 - 4.2 = 12 deg C (53.5 deg F)

(c) Determine long-term temperature change. This value is simply the peak RCC placement temperature less the stable minimum temperature. Assuming a May placement:

$\Delta T = T_p + T_{ad} - T_{min}$ = 41.1-11.9 = 29.2 deg C (53 deg F)

d. *Cracking analysis.*

(1) Step 5: Mass gradient cracking analysis.

(a) Determine mass gradient restraint conditions. Geometric restraint is conservatively set at K_R=1.0, since the structure has a low profile. Foundation restraint is set at K_f= 0.65, since the base is not rock but rather compacted structural backfill.

$K_R = 1.0 \qquad K_f = 0.65$

(b) Determine mass gradient thermal strain. The total induced strain in the mass RCC is the product of the long-term temperature change, the coefficient of thermal expansion and restraint factors:

Total induced strain = $(C_{th})(\Delta T)(K_R)(K_f)$
= (9.9 millionths/deg C)(29.2 deg F)(1.0)(0.65)
= 189 millionths

(c) Estimate mass gradient cracking. The strain that results in cracking of the structure is the total induced strain less the tensile strain capacity (ϵ_{sc}) of the material. The total crack width in the length of the structure is the cracking strain multiplied by the length of the structure. The estimated number of cracks are based on the assumed crack widths. Typical crack widths range from 0.002 to 5 mm(0.01 to 0.2 in.). The larger crack widths are typical of structures founded on flexible or yielding foundations. Since such a foundation exists here, a typical crack width of 4 mm (0.15 in.) was assumed:

Cracking strain = total induced strain - ϵ_{sc}
= 189 - 80 = 109 millionths

Total crack width = (weir length)(cracking strain) = (530 m)(1,000 mm/m)(109 millionths)
= 58 mm (2.3 in.)

Assumed crack widths = 4 mm (0.15 in.)

Estimated cracks = 58 mm/4 mm = 15 cracks

Estimated crack spacing = 530 m/15 cracks
= 35 m (116 ft)

Since contraction joints will be installed at 30-m (100-ft) spacing, additional cracking is not expected. Occasional center cracks can be expected where conditions and restraint factors vary from those assumed.

e. *Conclusions and recommendations*.

(1) Conclusions. Based on calculations similar to that shown above, on previous temperature analysis figures, and experience, the following conclusions were provided:

(a) May placement schedule. RCC placement temperatures should be 19.4 to 21.1 deg C (67 to

70 deg F) if aggregates are produced the preceding month. If aggregate processing is performed earlier, lower placement temperatures may result. Crack spacing in an unjointed structure is calculated to be 35 m (116 ft). The 30-m (100-ft) contraction joint interval easily accommodates this volume change with joint widths of approximately 3 mm (0.13 in.).

(b) June placement schedule. RCC placement temperatures should be 22.2 to 23.9 deg C (72 to 75 deg F) if aggregates are produced the preceding month. If aggregate processing is performed earlier, lower placement temperatures may result. Crack spacing in an unjointed structure is calculated to be 29 m (97 ft). The 30-m (100-ft) contraction joint interval just accommodates this volume change with joint widths of approximately 4 mm (0.15 in.).

(c) July and August placement schedules. RCC placement temperatures should be 23.9 to 26.7 deg C (75 to 80 deg F) if aggregates are produced the preceding month. If aggregate processing is performed earlier, lower placement temperatures may result. Crack spacing in an unjointed structure is calculated to be 26 m (87 ft). The 30-m (100-ft) contraction joint interval is not quite adequate to accommodate this volume change at a fixed joint width of 4 mm (0.15 in.). Joint widths will increase or additional cracking will occur.

(d) Since the anticipated period for RCC construction is during the late spring or summer months, the 29.4-deg C (85-deg F) placement temperature limitation specified could be a factor if unusually hot weather should occur. Under normal weather conditions, uncontrolled placing temperatures should range from 19.4 to 24.4 deg C (67 to 76 deg F) from May through August. In the event that abnormal weather causes average daily ambient temperature in excess of 29.4 deg C (85 deg F), RCC temperatures could exceed 29.4 deg C (85 deg F). Aggregate stockpile cooling and possible use of batch water chillers would be the most expedient solutions to this problem.

(e) The current joint spacing of 30 m (100 ft) is adequate for RCC placements during May and June. Later placements in July and August will result in occasional centerline cracking of monoliths, possibly in as many as three or four monoliths. Lesser cracking is very probable since material properties were conservatively estimated.

(f) Several material properties were applied conservatively. Small reductions of adiabatic temperature rise and coefficient of thermal expansion and small increases in tensile strain capacity could improve thermal cracking performance. If each of these properties were individually changed 10 percent, summer crack spacing would be around 30 m (100 ft). If these changes were cumulative, crack spacing would be over 40 m (130 ft).

(2) Recommendations.

(a) Maintain current 29.4-deg C (85-deg F) maximum placement temperature limitation. Consider allowing minor temperature violations so long as the time weighted average of the RCC placement temperature is maintained below 26.7 deg C (80 deg F).

(b) Maintain current contraction joint spacing of 30 m (100 ft). The current contraction joint configuration of 30-m (100-ft) joint intervals is sufficient to accommodate the total anticipated axial contractions due to cement induced temperature fluctuations during May and June placements. Some transverse cracking will occur during the July and August placement schedule, however the extent of cracking should not be of concern considering the upstream backfill and the frequency of use.

f. Field performance compared to predicted performance. During construction, RCC placement temperature was maintained at about 29.4 deg C (85 deg F), and transverse contraction joints were spaced at 30-m (100-ft) intervals. All the contraction joints opened properly during the first few months after construction, with no intermediate cracking. Crack widths varied from 1.5 to 6 mm (0.06 to 0.25 in.).

Table A2-2
Cache Creek Weir Placing Temperature Computation

Factor	May	Jun	Jul	Aug	Comments
		Temperature (deg C)			
Avg. annual temperature(deg C)	16.1	16.1	16.1	16.1	Base temperature, from NOAA data
Previous month temperature	15.0	18.9	22.6	24.8	From NOAA data
Added ambient temperature	-1.1	2.8	6.5	8.7	(0.67)(Annual temp. - prev. month temp.)
Aggregate subtotal temperature	15.4	18.0	20.5	21.9	Avg. annual temp. + added amb. temp.
Added processing temperature	+1.1	+1.1	+1.1	+1.1	Processing and crushing energy
Aggregate stockpile temperature	16.5	19.1	21.6	23.0	N/A
Current ambient temperature	18.9	22.6	24.8	23.9	From NOAA data
Added ambient temperature	+1.7	+2.3	+2.1	+0.6	(0.67)(Curr. Temp.-agg. stock. temp.)
Added mixer energy	+1.1	+1.1	+1.1	+1.1	N/A
Placement temperature	19.3	22.6	24.8	24.8	Agg. stockpile temp. + added effects
		Temperature (deg F)			
Avg. annual temperature (deg F)	61.1	61.1	61.1	61.1	Base temperature, from NOAA data
Previous month temperature	59.0	66.1	72.7	76.6	From NOAA data
Added ambient temperature	-1.4	3.3	7.8	10.4	(0.67)(Annual temp. - prev. month temp.)
Aggregate subtotal temperature	59.7	64.5	68.9	71.5	Avg. annual temp. + added amb. temp.
Added processing temperature	+2.0	+2.0	+2.0	+2.0	Processing and crushing energy
Aggregate stockpile temperature	61.7	66.5	70.9	73.5	N/A
Current ambient temperature	66.1	72.7	76.6	75.1	From NOAA data
Added ambient temperature	+3.0	+4.2	+3.8	+1.1	(0.67)(Curr. Temp.-Agg. Stock. Temp.)
Added mixer energy	+2.0	+2.0	+2.0	+2.0	N/A
Placement temperature	66.7	72.7	76.7	76.6	Agg. stockpile temp. + added effects

Table A2-3
Cache Creek Weir Thermal Analysis Summary

Parameter	Spring (May)	Late Spring (Jun)	Summer (Jul-Aug)
Temperature (deg C)			
Temperatures			
RCC placement temperature (deg C)	19.4	22.8	25.0
Adiabatic temperature rise (deg C)	22.2	22.2	22.2
Peak internal temperature (deg C) (Place temp. + adiabatic temp.)	41.7	45.0	47.2
Minimum temperature (deg C) (Based on annual temp. cycle)	12.2	12.2	12.2
Differential temperature (deg C) (Peak temp. - min. temp.)	29.4	32.8	35.0
Strain development			
Induced strain (millionths) (C_{th}=9.9 millionths/deg C, K_f=0.65, K_R=1.0)	189	211	225
Strain capacity (millionths)	80	80	80
Excess strain (millionths)	109	131	145
Crack distribution (length of weir = 530 m) (crack width = 4mm)			
Axis length contraction (mm)	51	76	76
Number of cracks (Contraction/crack width)	15	18	20
Avg. crack spacing (m) (Weir length/number of cracks)	35	29	26
Temperature (deg F)			
Temperatures			
RCC placement temperature (deg F)	67	73	77
Adiabatic temperature rise (deg F)	40	40	40
Peak internal temperature (deg F) (Place temp. + adiabatic temp.)	107	113	117
Minimum temperature (deg F) (Based on annual temp. cycle)	54	54	54
Differential temperature (deg F) (Peak temp. - min. temp.)	53	59	63
Strain development			
Induced strain (millionths) (C_{th}=5.5 millionths, K_f=0.65, K_R=1.0)	189	211	225
Strain capacity (millionths)	80	80	80
Excess strain (millionths)	109	131	145
Crack distribution (length of weir=1,740 ft.) (crack width=0.15 in.)			
Axis length contraction (in.)	2	3	3
Number of cracks (Contraction/crack width)	15	18	20
Avg. crack spacing (ft) (Weir length/number of cracks)	114	95	86

ANNEX 3: LEVEL 2 THERMAL STUDY MASS GRADIENT AND SURFACE GRADIENT ANALYSIS PROCEDURE AND EXAMPLES

A3-1. Procedure

a. General. This Annex summarizes typical steps in a Level 2 mass gradient and surface gradient thermal analysis of a mass concrete structure (MCS) and provides two examples of the procedure. Example 1 covers a simple one-dimensional (1-D) (strip model) finite element (FE) mass gradient and surface gradient thermal analysis. Example 2 presents a more complex two-dimensional (2-D) mass gradient and surface gradient thermal analysis. This procedure and the examples use FE methodology only because of the widespread availablility and use of this technology. Although other methods of conducting a Level 2 thermal analysis are available, these procedures are most commonly used.

b. Input properties and parameters. The level of data detail depends on the complexity of a Level 2 thermal analysis. Parametric analysis should be routinely conducted at this level, using a rational number and range of input properties and parameters to evaluate likely thermal problems.

(1) Step 1: Determine ambient conditions. Level 2 analyses may be based upon average monthly temperatures for a less complex analysis, or on average expected daily temperatures for each month for a complex analysis. Wind velocity data are generally needed for computing heat transfer coefficients. Extreme ambient temperature input conditions, such as cold fronts and sudden cold reservoir temperatures, can and should be considered when appropriate to identify possible problems.

(2) Step 2: Determine material properties. Thermal properties required for FE thermal analysis include thermal conductivity, specific heat, adiabatic temperature rise of the concrete mixture(s), and density of the concrete and foundation materials. Coefficient of thermal expansion is required for computing induced strain from temperature differences. Modulus of elasticity of concrete and foundation materials are required for determination of foundation restraint factors. Tensile strain capacity test results are important for cracking evaluation. When tensile strain capacity data are not available, the methodology presented in Annex 1 may be used to estimate probable tensile strain capacity performance of the concrete. Creep test results are necessary to determine the sustained modulus of elasticity (or an estimate of E_{sus} is made) if stress-based cracking analysis is used.

(3) Step 3: Determine construction parameters. Construction parameters must be compiled which include information about concrete placement temperature, structure geometry, lift height, construction start dates, concrete placement rates, and surface treatment such as formwork and insulation that are possible during construction of the MCS. To determine concrete placement temperature, a first approximation is to assume that concrete placement temperatures directly parallel the mean daily ambient temperature curve for the project site. Actual placement temperature data from other projects can be used for prediction, modified by ambient temperature data differences between the different sites. The temperature of the aggregate stockpiles may change more slowly than does the ambient temperature in the spring and fall. Hence, placement temperatures during spring months may lag several degrees below mean daily air temperatures, while placement temperatures in the fall may lag several degrees above mean daily air temperatures.

c. Temperature analysis

(1) Step 4: Prepare temperature model. Various temperature analysis methods suitable for Level 2 thermal analysis are discussed in Appendix A. Either step-by-step integration methods or FE models may be used for Level 2 temperature analysis or mass and surface gradients. If step-by-step integration methods are used, the computation or numerical model should be programmed into a personal computer spreadsheet. The decision on whether to use FE 1-D strip models or 2-D section analysis is gen-

erally based on complexity of the structure, complexity of the construction conditions, and on the stage of project design. Often 1-D strip models are used first for parametric analyses to identify concerns for more detailed 2-D analysis.

(2) Compute temperature histories. Once computed, temperature data should be tabulated as temperature-time histories and temperature distributions to obtain good visual representations of temperature distribution in the structure. ETL 1110-2-536 has examples of temperature distribution plots. Appropriate locations can then be selected for temperature distribution histories at which mass gradient and surface gradient analysis will be conducted.

(a) Step 5: Mass gradient temperature analysis. Temperature-time histories, showing the change in temperature with time at specific locations after placing, are generally used to calculate temperature differences for mass gradient cracking analysis. Temperature differences for mass gradient cracking analysis are generally computed as the difference between the peak concrete temperatures and the final stable temperatures that the cooling concrete will eventually reach.

(b) Step 6: Surface gradient temperature analysis. The objective of surface gradient temperature analysis is to determine at desired critical locations the variation of surface temperatures with depth and with time. This can be performed effectively with 1-D strip models or with 2-D analysis. Thinner sections may require temperature distributions entirely across the structure, while large sections often only require temperature to be evaluated to some depth where temperature changes are relatively slow. Ideally, temperature distribution histories are generated for a single lift, tabulated from one surface to the other (or a stable interior) with each distribution representing temperatures for a specific time after placement.

d. Cracking analysis.

(1) Step 7: Mass gradient cracking analysis. The mass gradient temperature differences are used with C_{th} and restraint factors (K_f and K_R) to evaluate mass gradient cracking potential, using Equation A-4 in Appendix A. Computed mass gradient strains are compared against tensile strain capacity to evaluate cracking potential. For a stress-based mass gradient cracking analysis, the sustained modulus of elasticity corresponding to the time frame of the analysis is used to convert strains calculated by Equation A-4 to stresses. The use of the sustained modulus allows for the relief of temperature-induced stress due to creep. These stresses are compared to the tensile strength of the concrete at the appropriate age to determine where and when cracking may occur.

(2) Step 8: Surface gradient cracking analysis. Surface gradient cracking analysis is based on higher temperature differences in the surface concrete compared to the more slowly cooling interior which creates areas of tension in the surface to some depth, H. Tensile strain is calculated based on C_{th}, the temperature difference at some depth of interest, and the degree of restraint based on H.

(a) Temperature differences are calculated using as a basis the temperature when the concrete first begins hardening, rather than a peak temperature as used in mass gradient computations. These temperature differences, with time and depth, allow determination of tensile and compression zones near the concrete surfaces. The point at which tension and compression zones balance is considered a stress-strain free boundary (located at H from the surface) used to compute restraint for surface gradient analysis. This point is generally calculated by evaluating temperature differences at depth with respect to temperature differences at the surface.

(b) Reference or initial temperatures for a surface gradient analysis are defined as the temperatures in the structure at the time when the concrete begins to harden and material properties begin to develop. Generally, this time is established at concrete ages of 0.25, 0.5, or 1.0 day. This age is dependent upon the rate at which the concrete achieves final set, the rate of subsequent cement hydration, and the properties of the mixture. For very lean concrete mixtures at normal temperature, a baseline time of 1.0 days may be reasonable. Mixtures that gain strength more rapidly at early ages may be better approximated by an earlier reference time of 0.25 or 0.33 days (6 or 8 hours).

(c) Internal restraint factors, K_R, are computed using Equation A-5 or A-6 in Appendix A, depending upon the ratio of *L/H*, where L is the horizontal distance between joints or ends of the structure, and *H* is the depth of the tension block. Induced tensile strains are computed at each analysis time from Equation A-8 in Appendix A using the coefficient of thermal expansion, the temperature differences between the surface and interior concrete, and the computed internal restraint factors. These strains are compared with slow load tensile strain capacity (selected or tested to correspond to the time that strains are generated) to determine cracking potential.

(d) Stress-based surface gradient cracking analysis is often handled in a slightly different way, particularly in the way creep is accounted for in the analysis. Commonly, incremental temperature differences at different depths and times are computed. These incremental temperature differences are converted to incremental stresses, including creep effects, using the C_{th}, E_{sus}, and K_R. The incremental stresses generated during each time period are summed to determine the cumulative tensile stress in the surface concrete at various depths. These stresses are compared to the tensile strength of the concrete at the appropriate age to determine cracking potential.

e. *Conclusions and recommendations*. These typically include expected maximum temperatures for starting placement in different seasons, expected transverse and longitudinal cracking without temperature or other controls, recommended concrete placement temperature limitations, anticipated concrete precooling measures, need for adjustment in concrete geometry, properties, joint spacing, and the sensitivity of the thermal analysis to changes in parameters. Typical temperature control measures evaluated might include reduced lift heights, use of insulated forms, and reduction in mix cement content. The potential for thermal shock may be addressed. In addition, recommendations for further or more advanced thermal analysis should be provided and justified.

A3-2. Example 1: One-Dimensional Mass Gradient and Surface Gradient Thermal Analysis

a. *General.* An example of a 1-D mass gradient and a surface gradient analysis in a Level 2 thermal study of an MCS is presented below. This example is based on preliminary 1-D analyses performed during feasibility studies on a proposed large flood control RCC gravity dam on the American River in California. This dam was planned to be 146 m (480 ft) high, 792 m (2,600 ft) long, with a downstream face slope of 0.7H:1.0V.

(1) The 1-D analysis was used as a screening tool only, to provide preliminary evaluation of several concerns and to develop information for more detailed analyses. These studies were conducted to ascertain the general extent of thermal cracking (cracking due to mass thermal gradients and surface thermal gradients), for guidance in selecting an appropriate joint spacing to accommodate transverse thermal cracking, to evaluate the possibility of longitudinal cracking in the structure, and for early planning and cost-estimating purposes. Figure A3-1 illustrates the 1-D strip models employed in this analysis and the overall dam proportions.

(2) FE analysis in this study was used only to determine temperature history for the various schedule alternatives, using the Fortran program "THERM." Stresses were determined by manual computational methods, based on temperature change computed by the FE temperature analysis, the coefficient of thermal expansion, the sustained modulus of elasticity, and the degree of restraint. To account for stress relief due to creep and because the mass concrete modulus of elasticity is very low at early ages, the analysis is segmented into several time spans, 1 to 3 days, 3 to 7 days, and 7 to 28 days. This allows use of changing material properties (modulus and creep) to be used for each time span, as well as changing *h* and *H* dimensions of the surface gradient tension block with time. Consequently, temperature changes were determined for each time span.

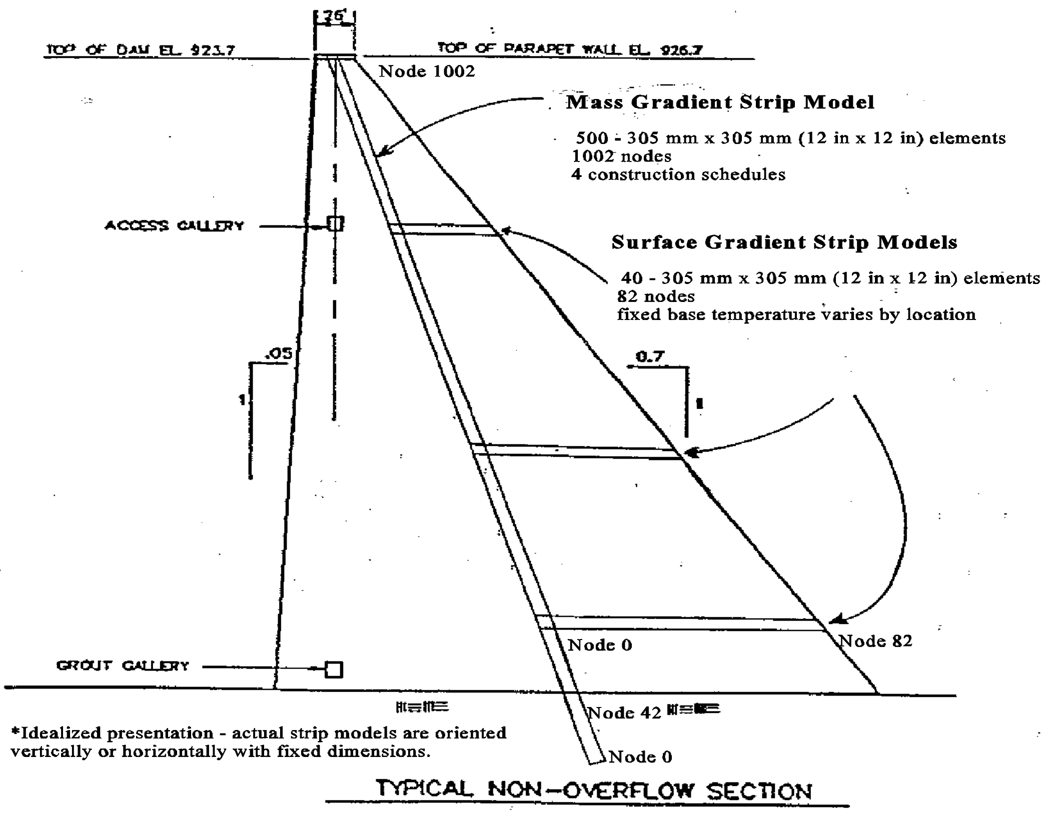

Figure A3-1. FE strip models

b. Input properties and parameters. At this early stage in the planning process, many of the details of the structure, materials performance, and placement constraints have not been determined and can only be approximated. It was decided that it would be prudent to make a reasonable estimate of those unknown parameters, and limit the study to evaluating the effects of variations of only a few items. In this study, those items subject to variations are certain material properties and the placing schedule.

(1) Step 1: Determine ambient conditions. Ambient air temperature data were produced from National Oceanic and Atmospheric Administration (NOAA) local climatological data. From these data, seven series of daily air temperature curves (shown in Figure A3-2) were developed, each representing the daily temperature cycle for one or more months. No data were available on how temperatures vary during each day. The curves are an estimate of the daily profile as it varies for each month throughout the year. No means of incorporating heat from solar gain was included in this analysis.

(2) Step 2: Determine material properties. Table A3-1 summarizes the applicable thermal and elastic properties of the materials considered for use in the structure. Most of the properties for the RCC and the foundation rock were estimated, or were the product of laboratory testing. Approximated values used for the modulus of elasticity, tensile strength, and creep rate are shown on Figure A3-3. Three materials were utilized for the analysis of the

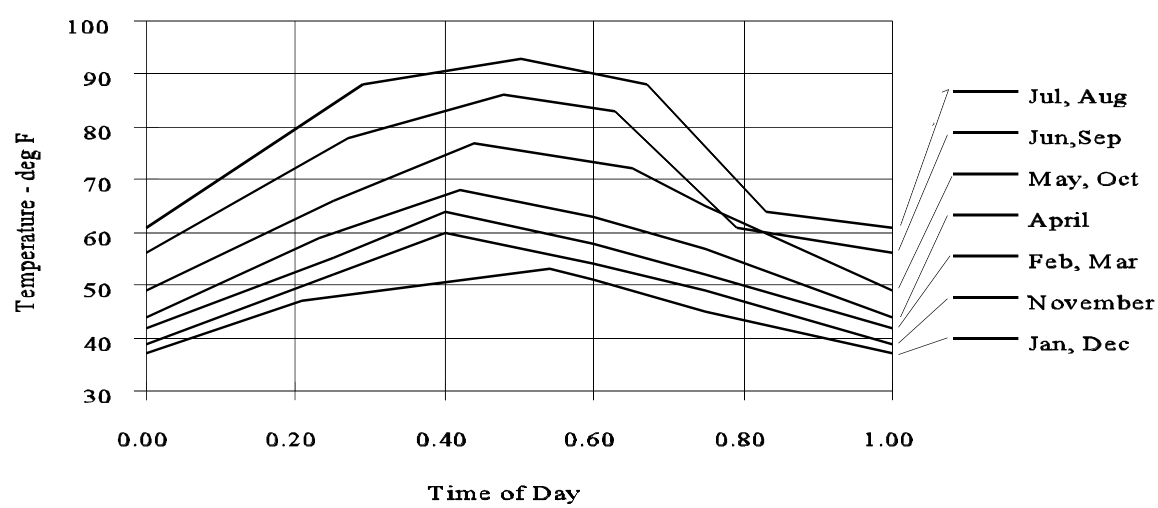

Figure A3-2. Daily ambient temperature cycles

Table A3-1
The RCC Material Properties for Mixtures

Property	Units	Damsite Alluvium	Damsite Amphibolite
Coefficient of thermal expansion (C_{th})[1]	millionths/deg C (millionths/deg F)	7.2 (4.00)	6.9 (3.86)
Thermal conductivity (K)	W/m-K (Btu/ft-hr-deg F)	2.42 (1.4)	2.77 (1.6)
Diffusivity (h^2)	m^2/hr (ft^2/hr)	0.038 (0.041)	0.0039 (0.042)
Specific heat ©	kJ/kg-K (Btu/lb-deg F)	0.92 (0.22)	0.92 (0.22)
Cement content[1]	kg/m^2 (lb/cy)	107 (180)	107 (180)
Flyash content[1]	kg/m^2 (lb/cy)	53 (90)	53 (90)
Adiabatic temperature rise (ΔT_{ad})	deg C (deg F)	15 (27)	15 (27)
Density[1]	kg/m^3 (lb/ft^3)	2,483 (155)	2,643 (165)
Tensile strain cap. (ϵ_{tc}) @ 7-90 day	millionths	100	100

[1] From test results

foundation and the dam construction. The foundation rock was assumed to provide thermal behavior similar to the amphibolite aggregate. The first 200 lifts of the dam use an RCC mixture with damsite alluvium aggregates. The remaining 280 lifts utilize an RCC mixture with amphibolite (metamorphosed sandstone) aggregate from the damsite.

(3) Step 3: Determine construction parameters.

(a) Construction start dates. To evaluate the effects of different construction start dates, the placement of concrete was evaluated during four time intervals. The initiation of RCC placements was set at 1 January, 1 April, 1 July, and 1 October

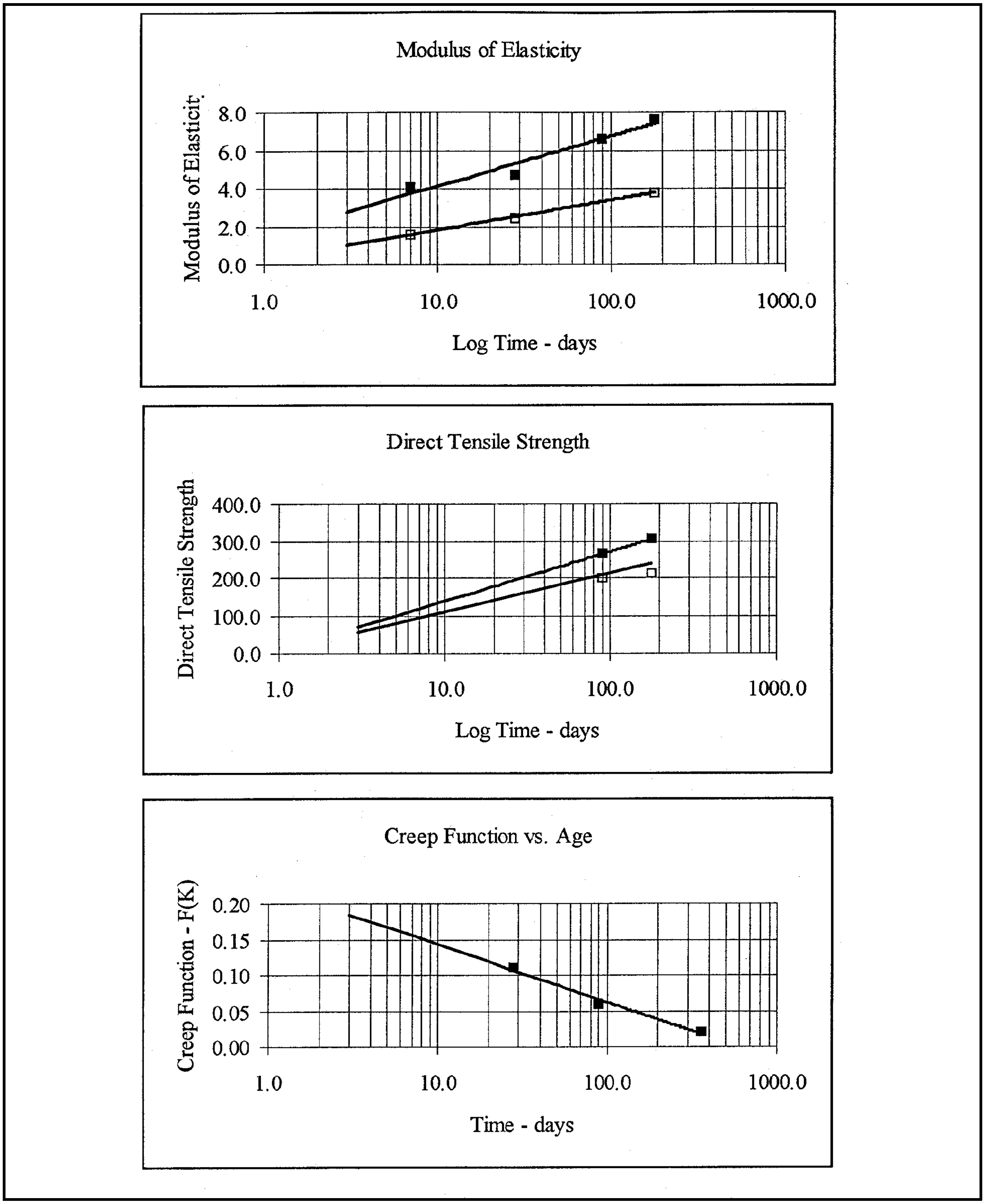

Figure A3-3. Estimated elastic and creep properties

of each year for the mass gradient analysis. For the surface gradient analysis, a 1 January start date was assumed.

(b) Concrete placing temperature. The temperature of the concrete aggregates has the greatest influence on the initial temperature of the fresh RCC. Because of the low volume of mix water, and the minor temperature differential of the water compared to the aggregate, the water temperature has a much less significant effect on overall temperature. Figure A3-4 provides the basis for the placing temperatures used in this study. Since aggregate production will be done concurrently by with RCC placement and regional temperatures tend to be moderate, stockpile temperatures should closely parallel the average monthly ambient temperatures. Some heat is added because of screening, crushing, and transportation activities, as shown in the figure, based on experience.

(c) Placement Assumptions. The RCC structure will be composed of two RCC mixtures, as previously described. The RCC placement will be in a 610-mm (24-in.) lift operation. The FE model is dimensioned having elements 305 mm (12 in.) in height. This allows future evaluations of 305-mm (12-in.) placing schemes, if desired. The RCC placement was assumed to occur on a schedule of 6 days per week, 20 hours per day, for the duration of the placement.

c. *Temperature analysis*.

(1) Step 4: Prepare temperature model (FE).

(a) The Fortran FE program "THERM", developed originally by Wilson (Wilson 1968), was used on a PC for the temperature analysis in this example. An Excel spreadsheet was used for development of an input file for THERM. Output nodal temperatures were imported into Excel spreadsheets for further analysis of cracking and graphical output. The FE grid, termed the mesh, provides more realistic results as it more accurately simulates the geometry of the structure. Since 1-D models (strip models) were used for the mass gradient analysis, heat only flowed vertically in or out of the model. Lateral heat flow in the upstream or downstream direction was not modeled. It is anticipated that actual heat dissipation in the dam over the long term will be at a more rapid rate than the model predicts. Since RCC construction is the continuous placement of relatively thin lifts, it is best modeled with elements of a height equivalent to the lift height or less. Unfortunately, since the American River Dam is a very massive structure, a mesh that provides ample detail would be monumental. A mesh of this magnitude is not necessary for the extent of evaluations to be done at this stage. Consequently, it was determined that a reasonable determination of internal temperatures could be done using strip models. A strip model is simply a vertical or horizontal "strip" of elements, usually only one element wide. Heat flows through the ends of the strip, but no heat flows from the sides. The model is located where necessary to simulate the thermal activity at that location. While the effects of many factors cannot be easily modeled using this method, generalized behavior can be determined.

(b) The primary mesh for mass gradient analysis, shown in Figure A3-1, is composed of 500 elements and 1,002 nodes. It simulates a strip through a cross section of the dam originating 6 m (20 ft) in the foundation rock. Elements 1 to 20 form the rock foundation with the bottom row of nodes set at a fixed temperature of 115.5 deg C (60 deg F), the mean annual air temperature for the area. An arbitrary time of 30 days is allowed to elapse prior to concrete placement to allow the rock temperatures to stabilize.

(c) The RCC at about dam midheight was evaluated for a surface temperature gradient. The surface gradient strip model spans from the exposed surface along a single lift to a point inside the structure where temperatures are assumed to not be influenced by ambient conditions. A small FE model was generated of approximately 82 nodes and 40 elements. Temperature histories of these nodes were then determined. The exterior surface of the surface gradient strip model was assumed to be fully exposed, with no insulation, using a heat transfer coefficient of 28.45 W/m^2-K (5.011 Btu/ft^2-hr-deg F).

Month	Mean Temp	Mean Annual	Diff	2/3 Diff	Sub Total	Crush Add	Stock Temp	Mixing Add	Trans Add	Final Temp
	degC (degF)	degC (degF)	degC (degF)	degC (degF)	degC (degF)	degC (degF)	degC (degF)	degC (degF)	degC (degF)	degC (degF)
Jan	7.1 (44.8)	15.5 (60.0)	-8.4 (-15.2)	-5.6 (-10.1)	9.9 (49.9)	1.1 (2.0)	11.1 (51.9)	1.1 (2.0)	-0.6 (-1.0)	11.7 (53)
Feb	9.2 (48.6)	15.5 (60.0)	-6.3 (-11.4)	-4.2 (-7.6)	11.3 (52.4)	1.1 (2.0)	12.4 (54.4)	1.1 (2.0)	0	13.3 (56)
Mar	10.5 (50.9)	15.5 (60.0)	-5.1 (-9.1)	-3.4 (-6.1)	12.2 (53.9)	1.1 (2.0)	13.3 (55.9)	1.1 (2.0)	0.6 (1.0)	15.0 (59)
Apr	13.2 (55.8)	15.5 (60.0)	-2.3 (-4.2)	-1.6 (-2.8)	14.0 (57.2)	1.1 (2.0)	15.1 (59.2)	1.1 (2.0)	0.6 (1.0)	16.7 (62)
May	17.0 (62.6)	15.5 (60.0)	1.4 (2.6)	0.9 (11.7)	16.5 (61.7)	1.1 (2.0)	17.6 (63.7)	1.1 (2.0)	1.1 (2.0)	20.0 (68)
Jun	21.4 (70.5)	15.5 (60.0)	5.8 (10.5)	3.9 (7.0)	19.4 (67.0)	1.1 (2.0)	20.6 (69.0)	1.1 (2.0)	1.1 (2.0)	22.8 (73)
Jul	25.1 (77.2)	15.5 (60.0)	9.6 (17.2)	6.4 (11.5)	21.9 (71.5)	1.1 (2.0)	23.1 (73.5)	1.1 (2.0)	1.7 (3.0)	25.6 (78)
Aug	24.5 (76.1)	15.5 (60.0)	8.9 (16.1)	5.9 (10.7)	21.5 (70.7)	1.1 (2.0)	22.6 (72.7)	1.1 (2.0)	1.7 (3.0)	25.6 (78)
Sep	22.1 (71.8)	15.5 (60.0)	6.5 (11.8)	4.4 (7.9)	19.9 (67.9)	1.1 (2.0)	21.1 (69.9)	1.1 (2.0)	1.1 (2.0)	23.3 (74)
Oct	17.4 (63.4)	15.5 (60.0)	1.9 (3.4)	1.3 (2.3)	16.8 (62.3)	1.1 (2.0)	17.9 (64.3)	1.1 (2.0)	0.6 (1.0)	19.4 (67)
Nov	11.5 (52.7)	15.5 (60.0)	-4.1 (-7.3)	-2.7 (-4.9)	12.8 (55.1)	1.1 (2.0)	13.9 (57.1)	1.1 (2.0)	0	15.0 (59)
Dec	7.7 (45.9)	15.5 (60.0)	-7.8 (-14.1)	-5.2 (-9.4)	10.3 (50.6)	1.1 (2.0)	11.4 (52.6)	1.1 (2.0)	-0.6 (-1.0)	12.2 (54)

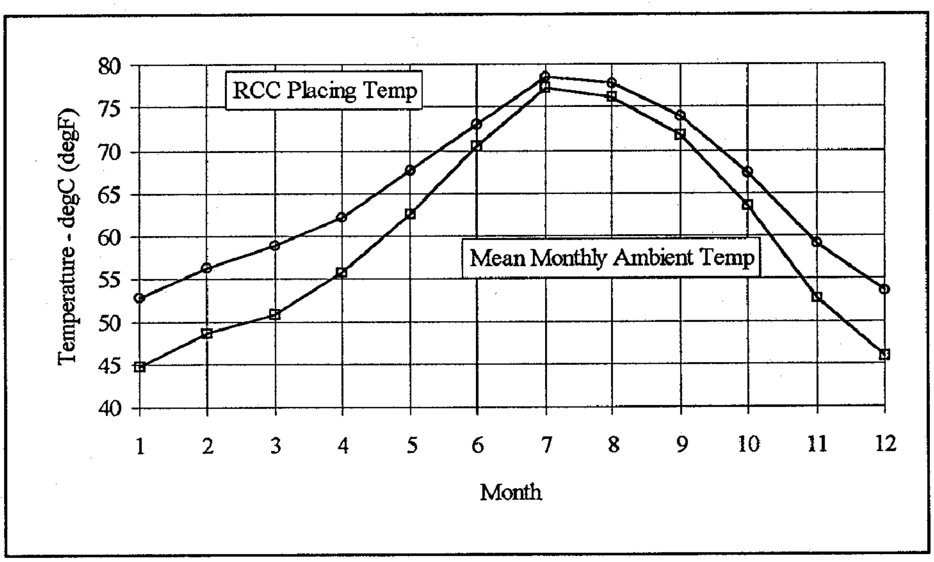

Figure A3-4. RCC placing temperature

(2) Compute temperature histories.

(a) Step 5: Mass gradient temperature analysis. Graphical representations for each of the four cases analyzed (one for each season) are shown in Figures A3-5 through A3-12. The first graph in each set is a time-history of nodal temperatures for selected nodes in the structure. This graph is useful to determine the time when certain zones in the structure reach certain temperatures. The second graph displays the maximum and minimum temperature experienced by each node. Note that these maximums and minimums occur at different times. The minimum temperatures of adjacent nodes fluctuate approximately 4 deg C (8 deg F) because of ambient temperature fluctuations. This graph is useful in determining the maximum temperature differentials, as well as determining the critical zones.

(b) Step 6: Surface gradient temperature analysis. Graphical representation of the single start date case analyzed is shown in Figure A3-13, and is comprised of families of curves representing temperature change with time for different depths from the exterior surface of the MCS. Figure A3-14 shows these temperatures converted to a family of curves of time versus distance from the surface on the x-axis. This conversion is done to ease the subsequent cracking analysis computations.

d. Cracking analysis. It is assumed for the purposes of this study that the initial (baseline) temperatures of the hardened RCC are those temperatures when the RCC is 24 hours old. Any subsequent change in temperature from this base forms the temperature gradient. For surface gradient analysis, the shallowest interior nodes where

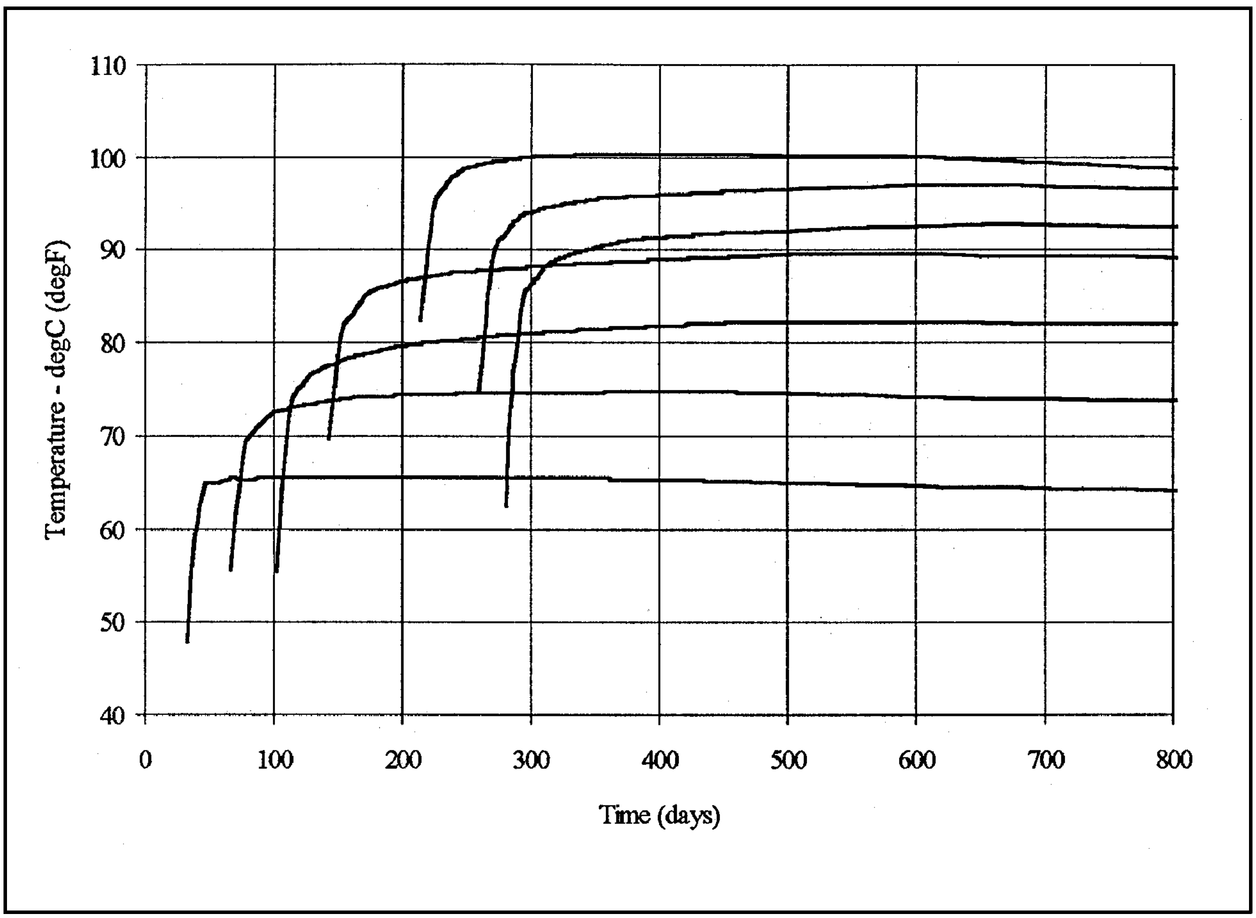

Figure A3-5. Mass gradient temperature histories for 1 January start

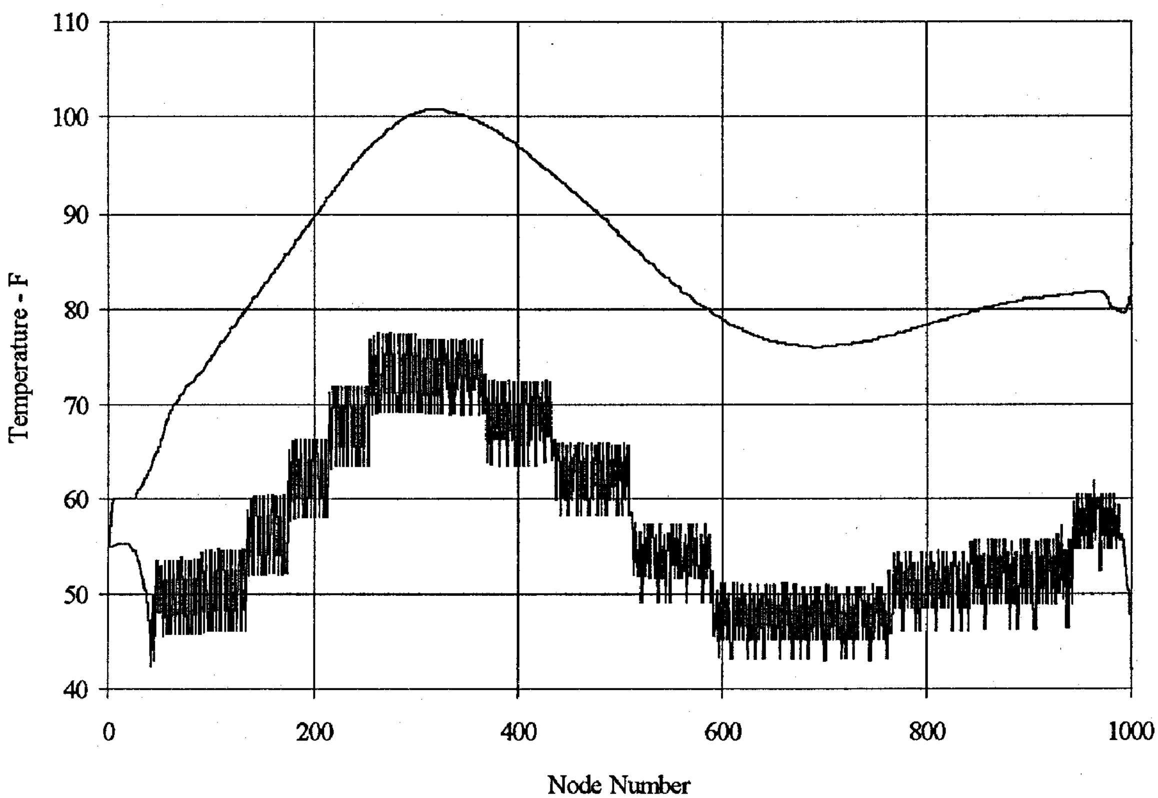

Figure A3-6. Mass gradient peak temperatures for 1 January start

temperatures do not change are assumed to be the location of the stress and strain-free surface. The distance from the surface to the location under consideration is used to calculate restraint factors *(K_R)* for both surface and mass gradient analysis.

(1) Step 7: Mass gradient cracking analysis. Several general statements can be made regarding the data. At locations low in the structure near the foundation, restraint conditions are the greatest. Consequently, allowable temperature differentials are at a minimum there. Progressing up and away from the foundation, restraint decreases, allowing a greater temperature differential before the onset of cracking. The graphs (Figures A3-6, 8, 10, and 12) in each of the analysis sets represent sections for the full height of the structure. However, the data can be applied to dam sections founded at higher elevations (*e.g.,* the abutments) by merely moving the y-axis to the right to a point corresponding to the appropriate foundation elevation. In this manner, the performance of the entire structure can be evaluated. In general, no cracking is expected if peak temperatures, low in the structure, do not exceed 29.4 deg C (85 deg F); because long-term cooling of the structure to 15.5 deg C (60 deg F) results in a 13.9-deg C (25-deg F) differential. Where nodal temperatures approach 37.8 deg C (100 deg F), they can be expected to remain above 29.4 deg C (85 deg F) for at least 5 years, and final cooling of the interior to 15.5 deg C (60 deg F) may take 15 to 20 years.

(a) Placement start on 1 January (Figures A3-5 and 6). Peak temperatures of 29.4 to 37.8 deg C (85 to 100 deg F) are realized in the part of the structure represented by nodes 200 to 500. This peak occurs during the month of July, after

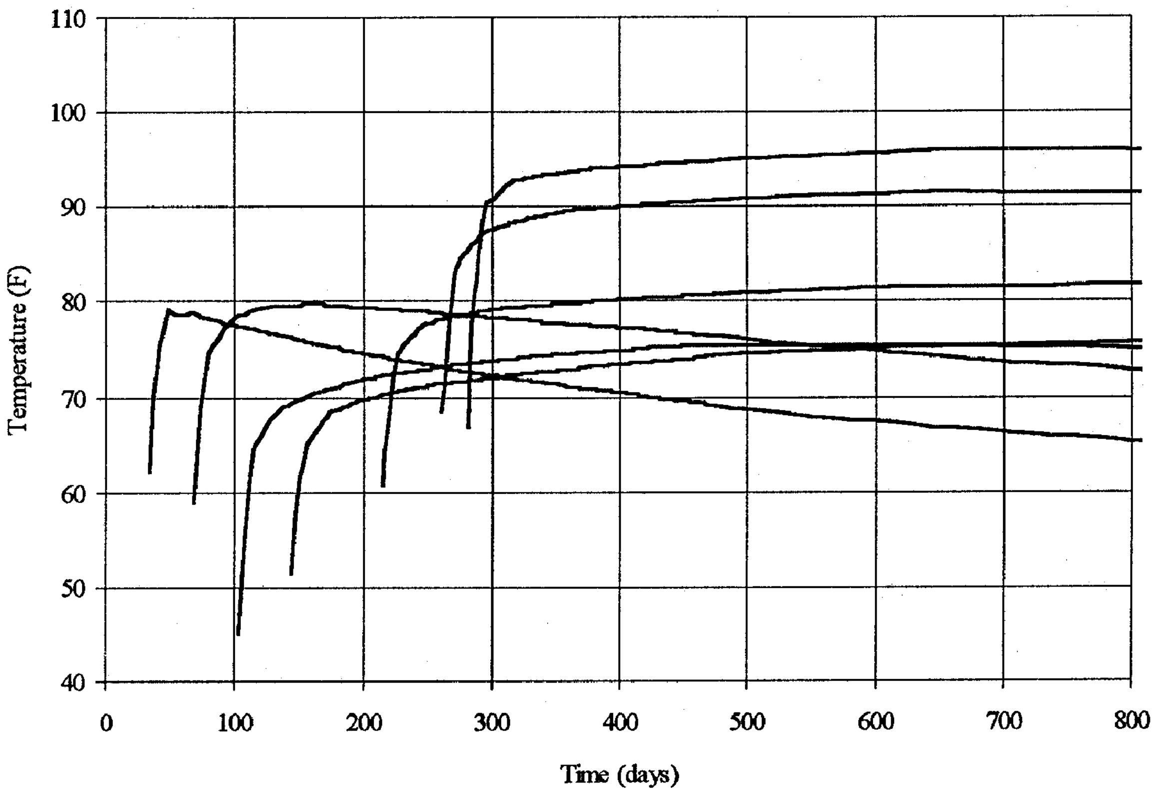

Figure A3-7. Mass gradient temperature histories for 1 October start

approximately 200 days of placement. Initial placements for the large monoliths are performed during the cool part of the year (winter and early spring), resulting in crack-free performance. Higher in the structure, where peak temperatures exceed 29.4 deg C (85 deg F), cracking does not occur because foundation restraint is reduced. The placements generating peak temperatures and resultant strains that may initiate cracking are those placements on the abutments between elevation 90 and 240 for a January start. This can be seen on Figure A3-6. Nodes 200 to 500 exceed 29.4 deg C (85 deg F). These nodes are located 27 to 73 m (90 to 240 ft) above the deepest foundation elevation.

(b) Placement start on 1 October (Figures A3-7 and 8). Peak temperatures of 29.4 to 37.8 deg C (85 to 100 deg F) are realized in the part of the structure represented by nodes 300 to 900. This peak occurs during the month of July, after approximately 300 days of placement. Initial placements for the large monoliths are performed during the cooler part of the year (fall, winter, and early spring), and peak temperatures never reach the critical level of 29.4 deg C (85 deg F). However, higher in the structure, where temperatures do exceed 29.4 deg C (85 deg F), cracking does not occur because foundation restraint is reduced. For an October start, the placements generating peak temperatures and resultant strains that may initiate cracking are those placements on the abutments at elevations 43 to 134 m (140 to 440 ft) from the lowest foundation elevation.

(c) Placement start on 1 July (Figures A3-9 and 10). Peak temperatures of 29.4 to 37.8 deg C (85 to 100 deg F) are realized in the part of the structure represented by nodes 50 to 200 and 500 to 1000.

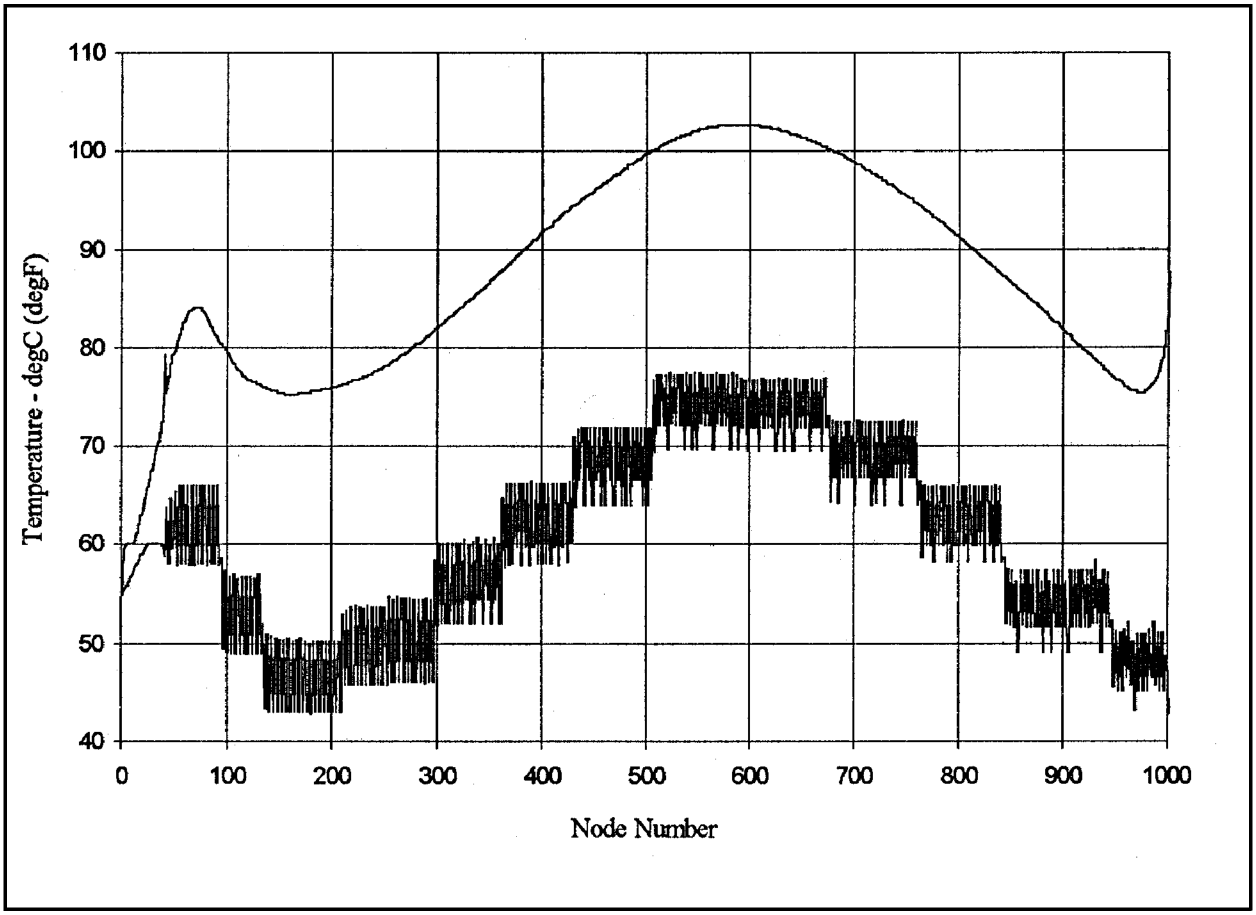

Figure A3-8. Mass gradient peak temperatures for 1 October start

This peak occurs after approximately 100 days of placement (during the month of July) for the early placements; and 1 year later for the upper dam placements. Initial placements for the large monoliths are performed during the warmest part of the year (the summer and early fall months), and peak temperatures exceed the critical level of 29.4 deg C (85 deg F). However, higher in the structure, where temperatures do exceed 29.4 deg C (85 deg F), cracking does not occur because foundation restraint is reduced. For a July start, the additional placements generating peak temperatures and resultant strains that may initiate cracking are those placements on the abutments at elevations 73 to 146 m (240 to 480 ft) above the lowest foundation elevation.

(d) Placement start on 1 April (Figures A3-11 and 12). Peak temperatures of 29.4 to 37.8 deg C (85 to 100 deg F) are realized in the part of the structure represented by nodes 100 to 400 and 800 to 1000. This peak occurs during the month of July, after approximately 100 days of placement for the early placements; and 1 year later for the upper dam placements. Initial placements for the large monoliths are performed during the moderate part of the year (the spring), avoiding cracking. Higher in the structure, where temperatures exceed 29.4 deg C (85 deg F), cracking does not occur because foundation restraint is reduced. Additional placements generating peak temperatures and resultant strains that may initiate cracking are those placements on the abutments from an elevation 12 to 49 m (40 to 160 ft) above the lowest foundation elevation and placements near the top of the dam.

(e) Mass gradient cracking analysis results. The following table summarizes, for each placing

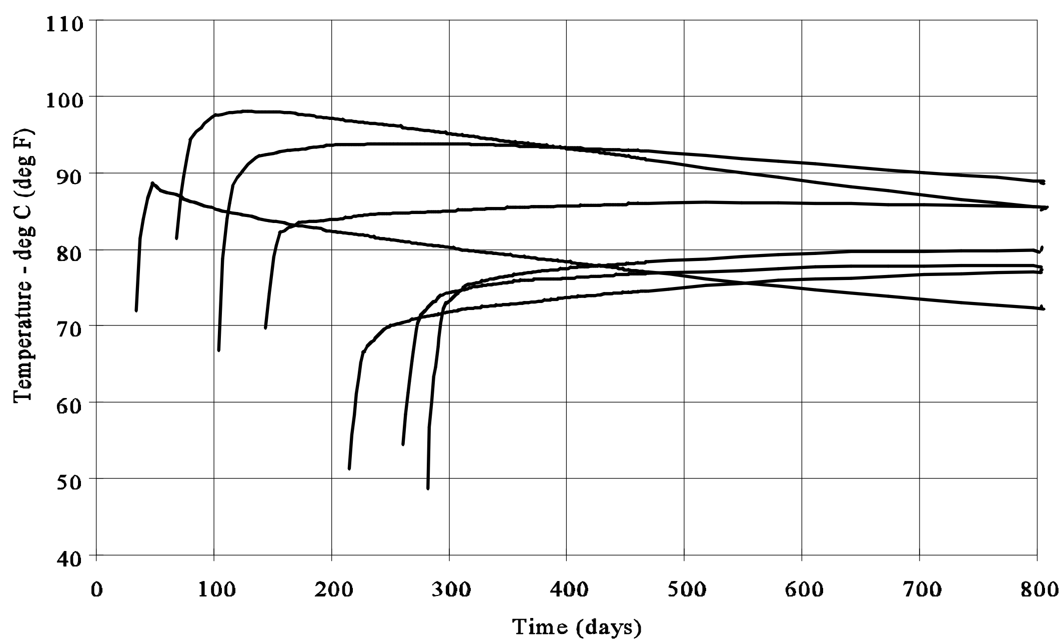

Figure A3-9. Mass gradient temperature histories for 1 July start

schedule evaluated, the nodes and the node locations where mass gradient thermal cracking is expected. The "Height Above Foundation" refers to those abutment foundation locations at elevations above the lowermost foundation elevation. For example, a January-start schedule results in probable cracking of nodes 200 to 400, and foundation elevations located 27 to 73 m (90 to 240 ft) above the lowest foundation elevation.

Uncontrolled RCC placing temperatures will result in peak temperatures of 37.8 deg C (100 deg F) and ultimate temperature differentials of 22.2 deg C (40 deg F). The maximum temperature differential calculated from tensile strain capacity and the coefficient of thermal expansions is 13.9 deg C (25 deg F) for the near term, increasing to near 16.7 deg C (30 deg F) for cooling periods of 15 years. Fall and winter placements result in cool placing temperatures, with peak temperatures for those placements of less than 29.4 deg C (85 deg F). Spring and summer placements result in peak temperatures exceeding 29.4 deg C (85 deg F), making cracking very probable. Cracking is generally induced at the foundation, where full restraint occurs and progresses up until restraint conditions lessen to the point where the driving force behind the crack is reduced. Since the force to propagate an existing crack is less than the force necessary to initiate the crack, it seems appropriate to assume that existing cracks may propagate further. The values shown in Table A3-2 do not include this extra crack height. Longitudinal cracking of the RCC in the large sections is not expected to be a problem when placement is done during the cool periods of the year. If these placements are done during the hot periods of the year, longitudinal

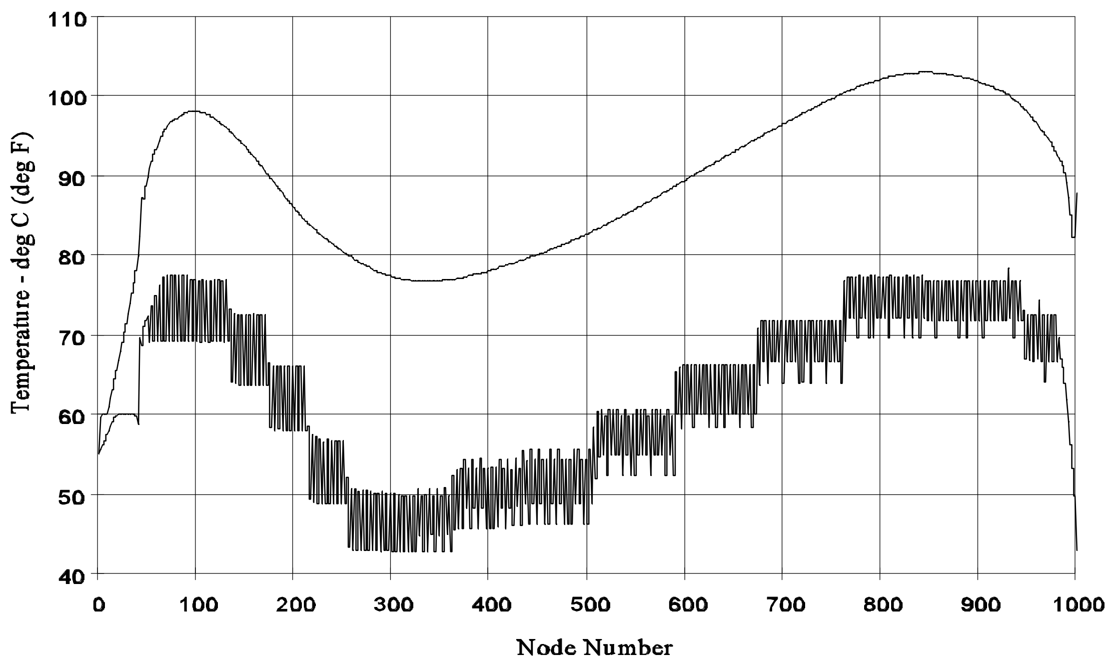

Figure A3-10. Mass gradient peak temperatures for 1 July start

cracking may occur. As construction progresses, placement of smaller RCC sections (those placements founded on rock at higher elevations) during hot periods is unavoidable. Longitudinal cracking of RCC placed against higher elevation foundation areas during these periods may occur. The conditions that may initiate longitudinal cracking may also initiate transverse cracking. The occurrence of transverse cracks can be reduced by installing transverse joints, thereby reducing the restraint.

(2) Step 8: Surface gradient cracking analysis. Surface gradient analysis was performed for several concrete placement start times, including the 1 January start time shown in this example. The effects of transverse joints at three different spacings were evaluated, including 30 m (100 ft), 61 m (200 ft), and 91 m (300 ft). The amphibolite aggregate RCC mixture was used in the evaluation. The procedure described here allows for consideration of changing concrete properties with age, such as *E* and creep, as well as changing *h* and *H* dimensions of the surface gradient tension block with time.

(a) Figure A3-13 presents the temperature data as a time-history plot for the conditions that should create the greatest surface gradient. Replotting the same data, based on nodal locations, yields Figure A3-14. Note that each curve represents the temperature cross section of the structure for a specific time. Each curve extends into the structure until the temperature becomes constant. Temperature differentials at specific locations are selected from Figure A3-14 and listed in Figure A3-15 (for 91-m (300-ft) joint spacing. Two basic assumptions are made in this analysis. First, temperatures of the RCC, at an age of 24 hours, are the baseline temperatures against which temperature change is determined. Second, the stress-strain free surface is assumed to be the depth at which the temperature

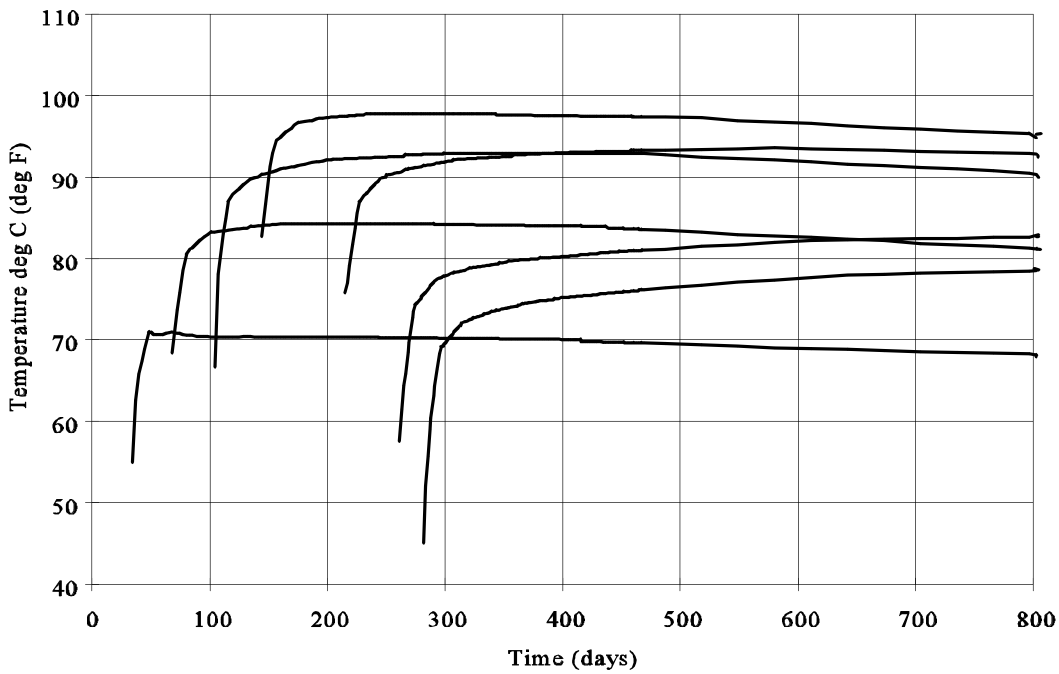

Figure A3-11. Mass gradient temperature histories for 1 April start

change, measured from the baseline temperature, approaches 0. Figure A3-15 shows the temperature deviations (*dT*) from the baseline temperature, as well as the depth at which the temperature gradient approaches 0. The Sum *dT* temperature differences are included on Figure A3-15 as a starting point for calculating induced stresses. "Induced *dT*," or the individual increments of temperature gradient induced with each age period, is calculated from the "Sum *dT*'s." Sustained modulus of elasticity (E_{sus}) is determined in Figure A3-15 for each age increment. To calculate incremental stress generated by temperature gradients:

$$\text{Incremental Stress} = (Ind\ dT)(C_{th})(E_{sus})$$

To determine K_R, Equation A-5 (Appendix A) is used, requiring calculation of *H*, *L*, and *h*. *H* is the distance from the exterior surface to the stress and strain-free surface at each incremental time period and is determined from the Temperature Differential Table in Figure A3-15 (note *H* for each age increment is the same). *L* is the joint spacing. *h* is the distance from the surface to the depth of interest (near surface, 0.6 , 1.5, 3, and 6 m (2, 5, 10, and 20 ft) in the figures), and *h/H* is the proportion of *H* from the surface to the depth of interest. *h/H* largely determines the amount of restraint at any location. K_R is calculated from Equation A-5 (Appendix A) for $L/H \geq 2.5$. The "Adj Stress" is calculated by:

$$\text{Adj Stress} = (K_R)(\text{Incremental Stress})$$

Cumulative stresses are then summed by superposition of stress from each age interval. Crack development is judged by whether the cumulative stress exceeds the tensile strength.

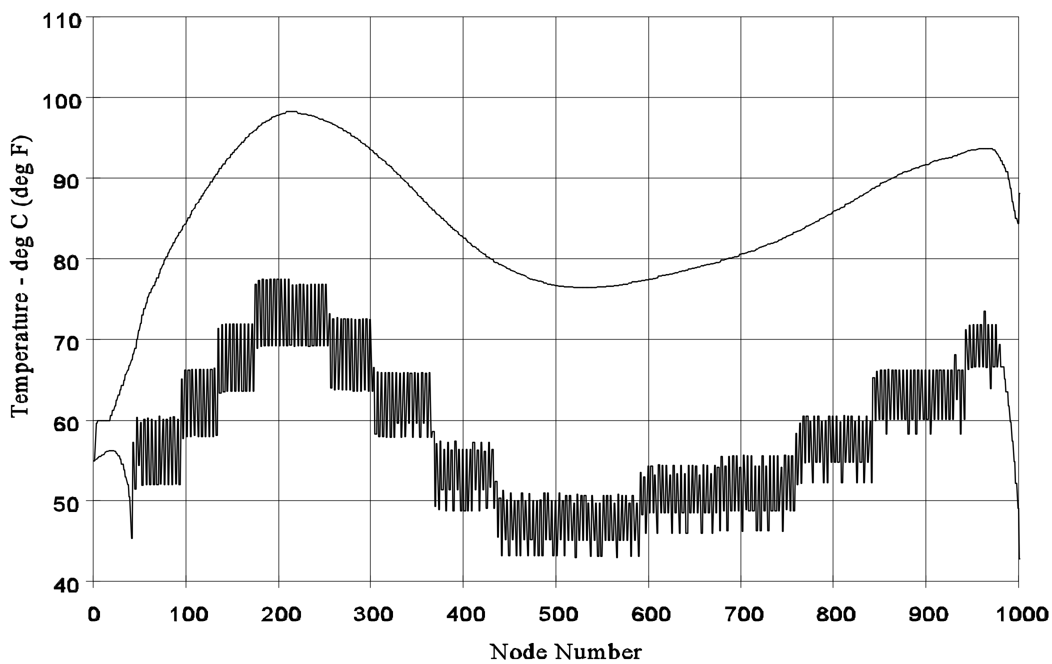

Figure A3-12. Mass gradient peak temperatures for 1 April start

From Figure A3-15 and similar computations for 30-and 61-m (100- and 200-ft) joint spacings, the computations indicate that surface cracking is not likely for a 30-m (100-ft) joint spacing. Surface cracking may increase to a depth of 0.6 m (2 ft) for joint spacings up to 61 m (200 ft) and up to 1.5 m (5 ft) for joint spacings of 91 m (300 ft). The full extent of surface cracking is controlled by the formation of the initial surface cracks. For example, at a joint spacing of 91 m (300 ft), the surface may crack at the midpoint. The analysis shows that this crack may propagate to a depth of 1.5 m (5 ft) after several weeks to months. However, the occurrence of this crack forms a new joint pattern at a spacing of 46 m (150 ft). While the depth of cracking may not be sufficient to change the restraint conditions (*L/H*), it may be enough to relieve induced stresses and stabilize the crack growth to depths of 0.6 m (2 ft). A joint spacing of 61 m (200 ft) may be an optimum spacing for this project based on the occurrence of surface cracking. Evaluation of the combined effects of surface gradient strains with mass gradient strains was not pursued, since the surface gradient strain contribution is not considered to be significant to the overall cracking performance of the structure using joint spacings of 30 and 61 m (100 and 200 ft).

e. Conclusions and recommendations. The maximum temperature differential under full restraint conditions (K_R = 1.0) that will not result in cracking of the RCC is 13.9 deg C (25 deg F). Since the final temperature of the RCC will be 15.5 deg C (60 deg F) (the average annual temperature), a crack-free peak RCC temperature is 29.4 deg C (85 deg F). This allowable differential of 13.9 deg C (25 deg F) increases as the distance of the RCC placements from the foundation

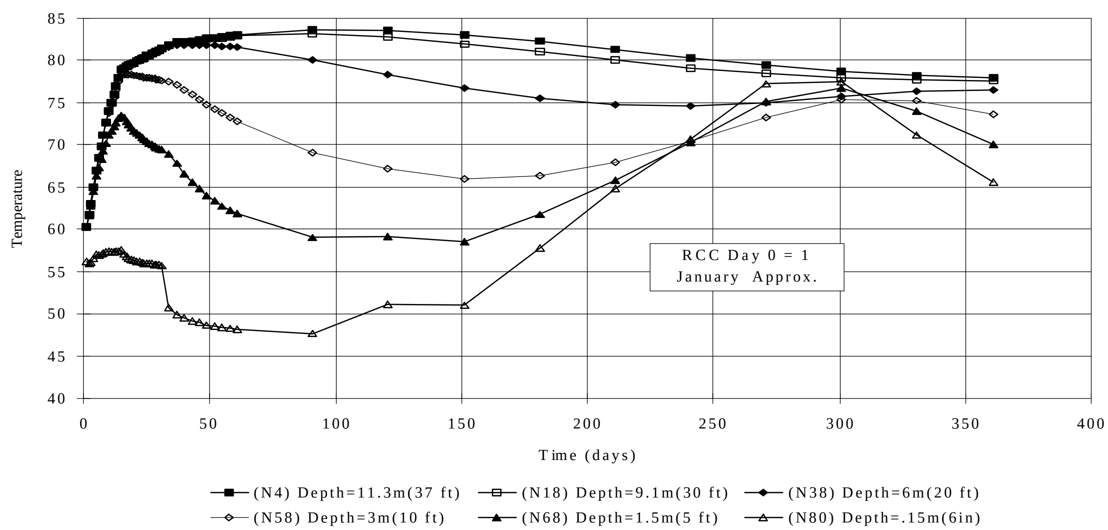

Figure A3-13. Temperature history for selected nodes from surface gradient model

increases. After evaluating several placing schedules, it was apparent that the most beneficial conditions occurred when the RCC placement of the lower third of the dam commenced in the fall of the year and was completed during late spring. This means that, for the larger dam sections, the upper two-thirds would then be placed during a hotter time period. The reduction in foundation restraint at this height in the structure, however, more than offset the effects of the higher temperatures.

Surface gradients were evaluated for several transverse joint intervals. Because the site is located in a relatively temperate area, where cold temperatures are rare, stresses from surface gradients were of little consequence for joint spacings up to 61 m (200 ft). Greater joint spacings increase the depth of surface cracking.

For contraction joints set at a spacing of approximately 61 m (200 ft), transverse cracking of the structure may occur in the lower 6 to 12 m (20 to 40 ft) of the structure. Similarly, longitudinal cracking may occur in the lower 6 to 12 m (20 to 40 ft) of the structure for sections of the dam having an upstream-downstream dimension greater than 61 m (200 ft). Since the occurrence of a longitudinal crack could create serious stability concerns, more rigorous analyses coupling the effects of other simultaneous loadings are necessary to better evaluate the extent of cracking.

An alternate rock source, a nearby quarried limestone aggregate, provides an RCC with a very low coefficient of thermal expansion of 4.5 millionths/deg C (2.5 millionths/deg F). The net effect of using this aggregate instead of the damsite amphibolite is to raise the allowable maximum peak temperature from 29.4 to 37.8 deg C (85 to 100 deg F). It appears that if this aggregate is used, no further control of aggregate temperatures may be necessary. Without this aggregate, measures are necessary to control placing temperatures so that peak temperatures do not exceed 29.4 deg C (85 deg F). This requires a 15.5-deg C (60-deg F) placing temperature for certain placements. This placing temperature could be raised to 23.9 deg C (75 deg F), if the limestone aggregate was used.

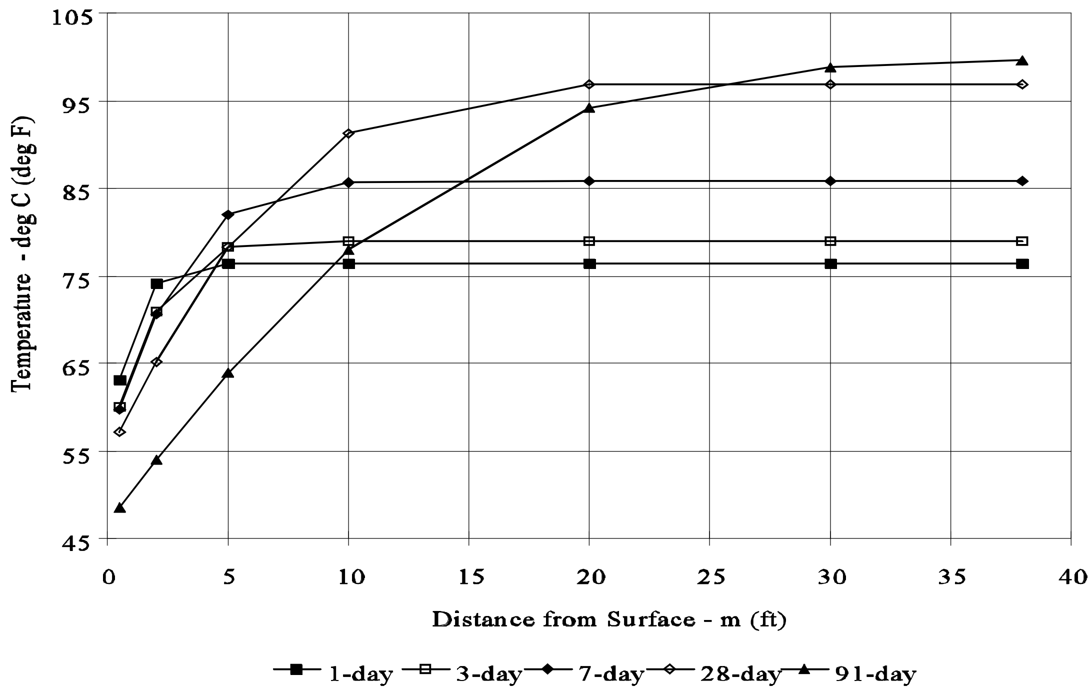

Figure A3-14. Surface gradient temperature distribution

Completion of RCC placements up to a minimum elevation during a fall and winter time period should be required in the construction contract. Otherwise, if these low elevation placements are placed during the spring and summer period, the RCC placing temperature should be specified not to exceed 26.7 to 29.4 deg C (80 to 85 deg F). This will require the use of additional cooling measures. Stockpile sprinkling, water chilling, and possible shading may be sufficient to achieve these temperatures.

The scope of this study was of a limited nature: to identify the potential extent of thermal cracking in the structure. Only generalized conclusions are possible. For a structure of this height, volume, and seismic loadings, a more rigorous study should be performed during design of the structure.

Full-section modeling, incorporating foundation properties, restraint conditions, and early-age material properties (time- and temperature-dependent properties) should be done. The structure should be analyzed in sections to ascertain the strain development that may lead to longitudinal cracking and in elevation to ascertain strain development that may lead to transverse cracking. The results of these studies should guide the designer as to whether a three-dimensional (3-D) model is necessary. It is presumed that a 3-D analysis will indicate better cracking performance of the structure than a two-dimensional (2-D) model would indicate. This analysis should quantify the effects of several load conditions in addition to the thermal loads. It may be that the combined action of these factors will initiate cracking.

Table A3-2
Summary of Locations of Mass Gradient Thermal Cracks

Schedule	Peak Temp deg C (deg F)	Critical Nodes	Height Above Foundation, m (ft)
Jan	37.8 (100)	200-400	27 - 73 (90-240)
Oct	37.8 (100)	300-900	43 - 134 (140-440)
July	37.8 (100)	50-200 and 500-1000	73 - 146 (240-480)
April	37.8 (100)	100-400 and 800-1000	12 - 49 (40-160) and near top of dam

A3-3. Example 2: Two-Dimensional Mass Gradient and Surface Gradient Thermal Analysis

a. *General.* An example of each step in the performance of a relatively complex mass gradient and a surface gradient analysis in a Level 2 thermal study of an MCS is presented. This example is based on 2-D analyses performed during design studies for locks and dam facilities on the Monongahela River in Pennsylvania. These studies were conducted to maximize lift heights and determine optimum placement temperatures, to expedite construction and minimize costs. Although numerous lock monolith configurations exist in the project, the most massive section was selected for analysis. Conclusions and recommendations from this analysis could be applied to the other project monoliths. Figure A3-16 shows a cross section representation of the geometry of a river wall monolith with nominal 3-m (10-ft) lifts used in this example analysis. Two-dimensional FE analysis was used to determine temperature histories and temperature distribution during and following construction. FE analysis was not applied for cracking analysis. Cracking analysis was performed using a strain-based criteria similar to procedures described in ACI 207.2R. Slow-load tensile strain capacity test results (which include creep effects) were used to determine the extent of cracking. Analysis was performed on 15 combinations of several parameters, including three lift heights, two maximum concrete placement temperatures, three construction start times, two lift placement rates, and insulated forms for fall placement.

b. *Input properties and parameters.*

(1) Step 1: Determine ambient conditions. These data were gathered from local records. Ambient temperature data are shown in Figure A3-17.

(2) Step 2: Determine material properties. Table A3-3 contains thermal properties used in the example thermal analysis. Adiabatic temperature rise is shown in Figure A3-18. This adiabatic temperature rise is characteristic of the heat generation of an exterior concrete in a mass concrete structure and is not characteristic of interior mass concrete. The foundation material is assumed to be limestone of moderate strength. Table A3-4 contains mechanical properties used in the example thermal analysis modulus of elasticity of concrete and foundation materials are required for determination of foundation restraint factors. Slow-load tensile strain capacity values were developed using Annex 1 methodology for use in mass and surface gradient cracking analysis as discussed later in this annex.

(3) Step 3: Determine construction parameters. Figure A3-17 shows the concrete placement temperatures used in the example thermal analysis. Maximum placement temperature during the summer is 15.5 deg C (60 deg F), and minimum placement temperature during the winter is 4.4 deg C (40 deg F), based on previous specification experience. Placement temperatures are expected to

Sustained Modulus of Elasticity (x 10E6 psi)

Age (T)		E2	E1	Eave	F(k)	Esus
1	3	1.20	2.70	1.95	0.20	1.72
3	7	2.70	4.00	3.35	0.17	2.42
7	28	4.00	5.20	4.60	0.14	2.31
28	90	5.20	6.70	5.95	0.11	2.51

Data from Figure B3

1/Esus=(1/((E1+E2)/2)+F(k)*ln(T2-T1)/2
Esus = Sustained Modulus
F(k) = Creep Function

Temperature Differential (degF)

Surface	.5-ft	2-ft	5-ft	10-ft	20-ft	30-ft
3-day						
dT	3	3	-2	-3	-3	-3
Sum-d	6	6	1	0	0	0
7-day						
dT	3	3	-6	-10	-10	-10
Sum-d	13	13	4	0	0	0
28-day						
dT	6	9	-2	-14	-20	-20
Sum-d	26	29	18	6	0	0
91-day						
dT	15	20	12	-2	-18	-22
Sum-d	37	42	34	20	4	0

Data from surface geadient temperature distribution Figure
dT = temperature differential between baseline and specific time curve
Sum-dT = dT adjusted to the difference from the constant interior temperature

Induced Stress at each Depth for 300-foot Joint Spacing

Age days	Sum dT	Ind dT	Incre. Stress	H	L/H	h/H	KR	Adj Stress	Age Range	Cumm Stress	Tensile Str.	Crack
		Cth =	3.86		Length btw Joints=			300 feet				
Near Surface (0.5 ft)												
0-3	6	6	40	10	30	1.00	0.90	36	0-3	36	70	no
3-7	13	7	65	10	30	1.00	0.90	59	0-7	95	140	no
7-28	26	13	116	20	15	1.00	0.81	94	0-28	189	200	no
28-90	37	11	106	30	10	1.00	0.73	77	0-90	266	260	yes
2-Ft												
0-3	6	6	40	10	30	0.80	0.92	37	0-3	37	70	no
3-7	13	7	65	10	30	0.80	0.92	60	0-7	97	140	no
7-28	29	16	142	20	15	0.90	0.83	118	0-28	215	200	yes
28-90	42	13	126	30	10	0.93	0.74	93	0-90	309	260	yes
5-Ft												
0-3	1	1	7	10	30	0.50	0.95	6	0-3	6	70	no
3-7	4	3	28	10	30	0.50	0.95	27	0-7	33	140	no
7-28	18	14	125	20	15	0.75	0.86	107	0-28	140	200	no
28-90	34	16	155	30	10	0.83	0.77	119	0-90	258	260	yes
10-Ft												
0-3	0	0	0	10	30	0.00	1.00	0	0-3	0	70	no
3-7	0	0	0	10	30	0.00	1.00	0	0-7	0	140	no
7-28	6	6	53	20	15	0.50	0.90	48	0-28	48	200	no
28-90	20	14	135	30	10	0.67	0.81	109	0-90	158	260	no
20-Ft												
0-3	0	0	0	10	30	-1.00	1.11	0	0-3	0	70	no
3-7	0	0	0	10	30	-1.00	1.11	0	0-7	0	140	no
7-28	0	0	0	20	15	0.00	1.00	0	0-28	0	200	no
28-90	4	4	39	30	10	0.33	0.90	35	0-90	35	260	no

KR calculated from earlier Figure. Tensile strength (psi) from earlierFigure.
Sum dT transferred from temp. diff. Table. Ind dT = incremental induced temp. gradient
Incre. stress = (Ind dT)(Cth)(Esus)
H = dist. from face to zero temp. diff. region (from temp. diff. table) L = joint spacing
h = depth of interest (0.5, 2, 6, 10, and 20 ft)
Adj stress = (KR)(Incre. stress)

Figure A3-15. Surface gradient cracking analysis

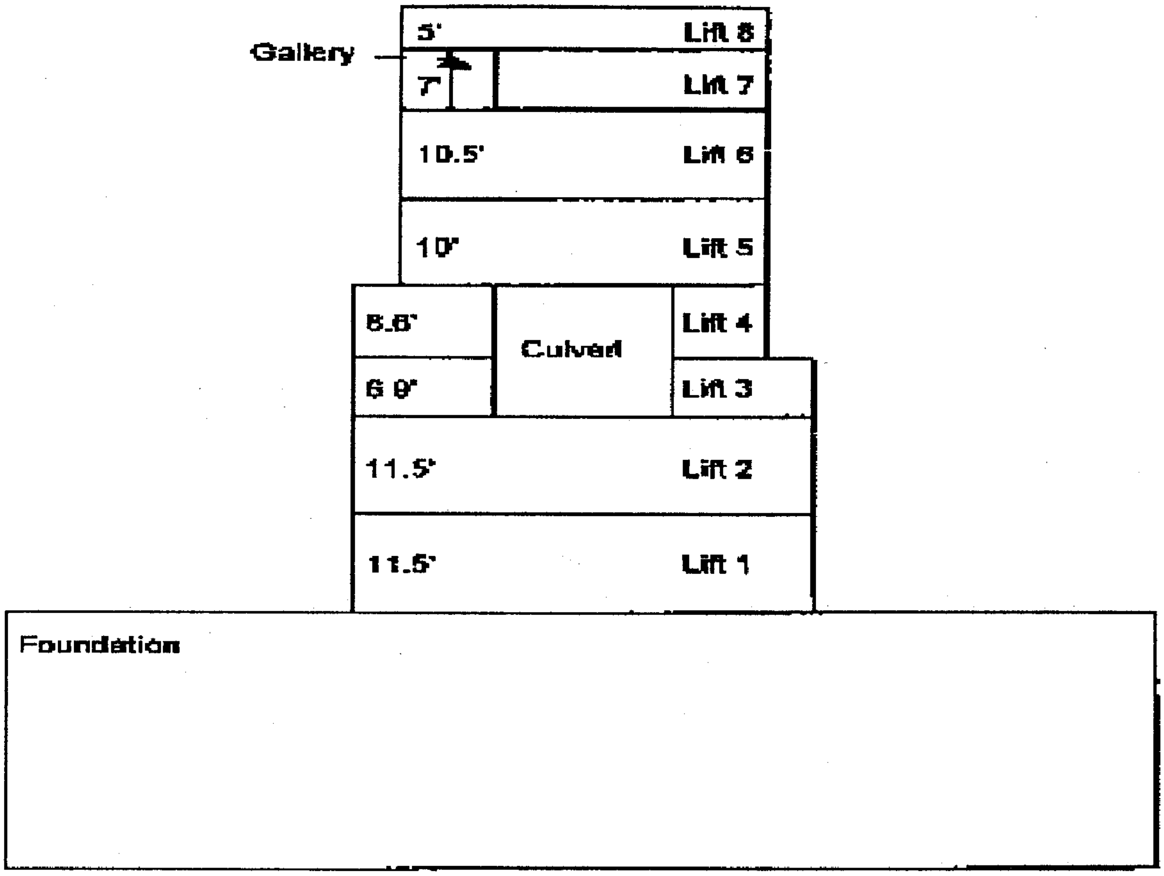

Figure A3-16. Lock wall section used in example

follow mean daily temperatures, except during summer and winter, when temperature controls are typically imposed. Placement temperatures lag mean daily ambient temperatures in the fall by 2.8 deg C (5 deg F), until the 4.4-deg C (40-deg F) minimum placement temperature permitted is reached. Other construction parameters assumed are a nominal lift height of 3 m (10 ft), a construction start date of 1 July, a concrete placement rate of 5 days/lift, with plywood forms removed 2 days after placement, and no insulation.

c. Temperature Analysis.

(1) Step 4: Prepare temperature model. The ABAQUS FE program was used in this example. Details regarding the use of ABAQUS and various ABAQUS and general FE program setup considerations in thermal analyses can be found in ETL 1110-2-365. Figure A3-19 shows the FE model used for the example. These analyses were performed on the Cray at the U.S. Army Engineer Waterways Experiment Station (WES). A time-step of 0.25 days was used to compute

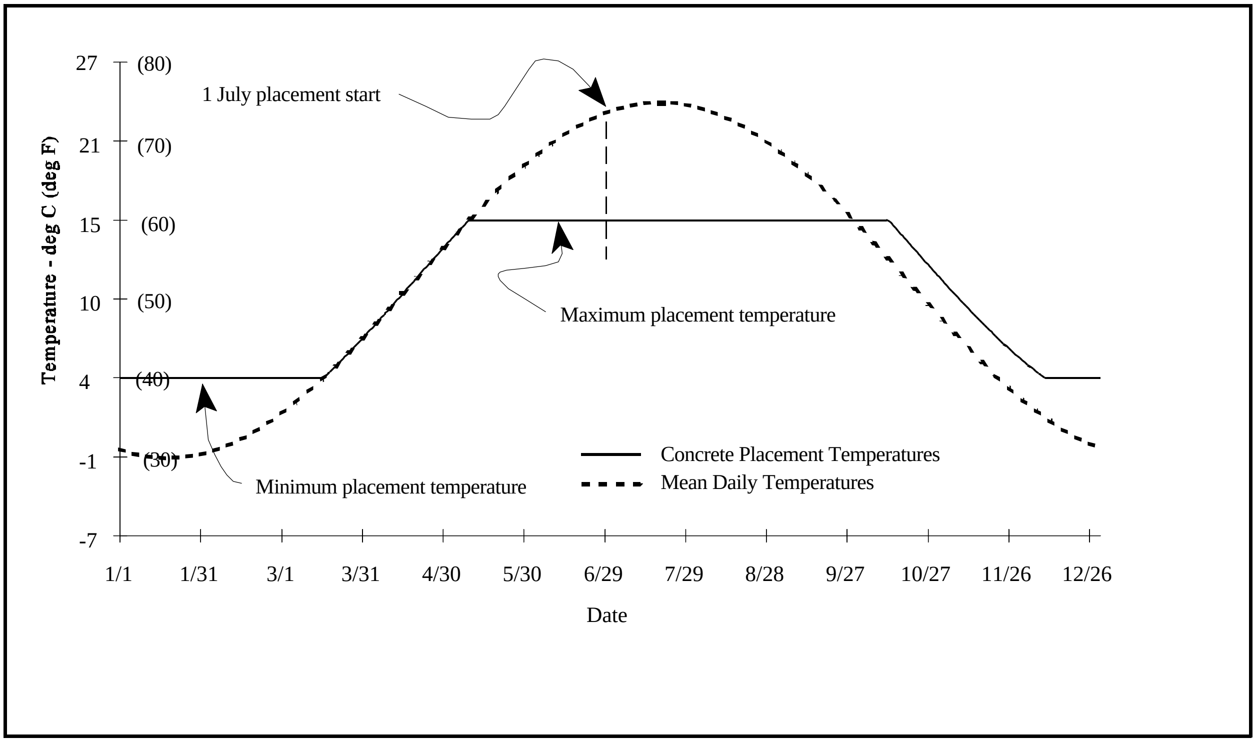

Figure A3-17. Mean daily ambient temperatures and concrete placement temperatures

Table A3-3
Concrete and Foundation Thermal Properties

Material	Thermal Conductivity W/m-K (Btu/hr-ft-deg F) (Btu/day-in-deg F)	Specific Heat kJ/kg-K (Btu/lb-deg F)	Coefficient of Thermal Expansion millionths/ deg C (millionths/deg F)
Limestone foundation	0.86 (0.500)(1.000)	0.96 (0.230)	9.90 (5.50)
Exterior concrete mixture	1.75 (1.012)(2.025)	0.98 (0.235)	10.46 (5.81)

temperature changes, primarily to capture temperature changes during the first 2 days after placement.

(a) Surface heat transfer coefficients computations. Equations A-2 and A-3 from Appendix A were used for computing the surface heat transfer coefficient. Table A3-5 shows surface heat transfer coefficients computed for various surface treatments at several time periods during the year. The heat transfer coefficients used in this example were those computed for wind only or for wind and plywood forms.

(b) Compute temperature histories. Figure A3-16 shows locations of mass gradient and surface gradient analysis in the structure used in the example. A July 1 start date was assumed for placement of the first lift of mass concrete.

(2) Step 5: Mass gradient temperature analysis. Figure A3-20 shows temperature histories at the locations of mass gradient analysis in the example.

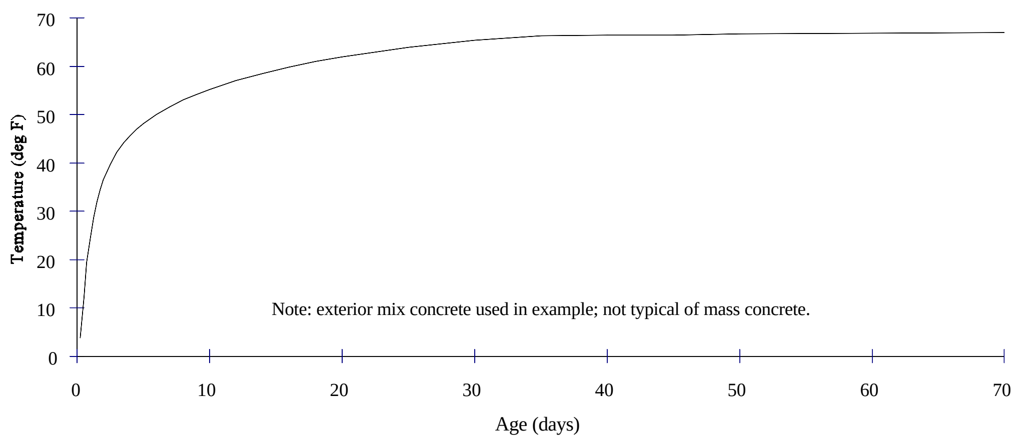

Figure A3-18. Adiabatic temperature rise for Level 2 thermal analysis 2-D example

Table A3-4
Concrete and Foundation Mechanical Properties

Material	Density	Compressive Strength	Modulus of Elasticity
	kg/m^3 (lb/ft^3)	Mpa (psi)	GPa (x 10^6 psi)
Limestone	2,563 (160)	103.4 (15,000)	48.26 (7.00)
Exterior concrete @ 1 day	2,243 (140)	3.93 (570)	12.41 (1.80)
Exterior concrete @ 3 days	same	7.65 (1,110)	20.20 (2.93)
Exterior concrete @ 7 days	same	11.24 (1,630)	23.44 (3.40)
Exterior concrete @ 28 days	same	22.48 (3,260)	33.65 (4.88)
Exterior concrete @ 90 days	same	31.10 (4,510)	35.51 (5.15)

(3) Step 6: Surface gradient temperature analysis. Surface gradient cracking in the example was analyzed at nominal ages of 0.5, 1, 2, 3, 5, 7, 14, 28, 60, 90, 120, 150, and 180 days after placement in lift 6 for this example. Table A3-6 and Figure A3-21 show the surface gradient temperature distributions across lift 6 in the upper portion of the mass concrete structure, determined from FE temperature analysis. Placement time for this lift was 25 days after placement of lift 1.

(a) Calculate surface gradient strains. To calculate surface gradient strains requires determination of the depth from the surface of effective interior restraint. This is performed by evaluating the magnitude of temperature change in the interior versus the surface concrete, thereby defining a surface "tension block" described in Appendix A and earlier in this annex. The following steps illustrate a procedure for determining the distance from the surface where tensile and compressive forces balance, thereby determining the distance from the surface to the point of zero strain, defining the tension block depth. A series of manipulations of temperature history results are used to define the depth, "*H*," of the tension block, where temperature

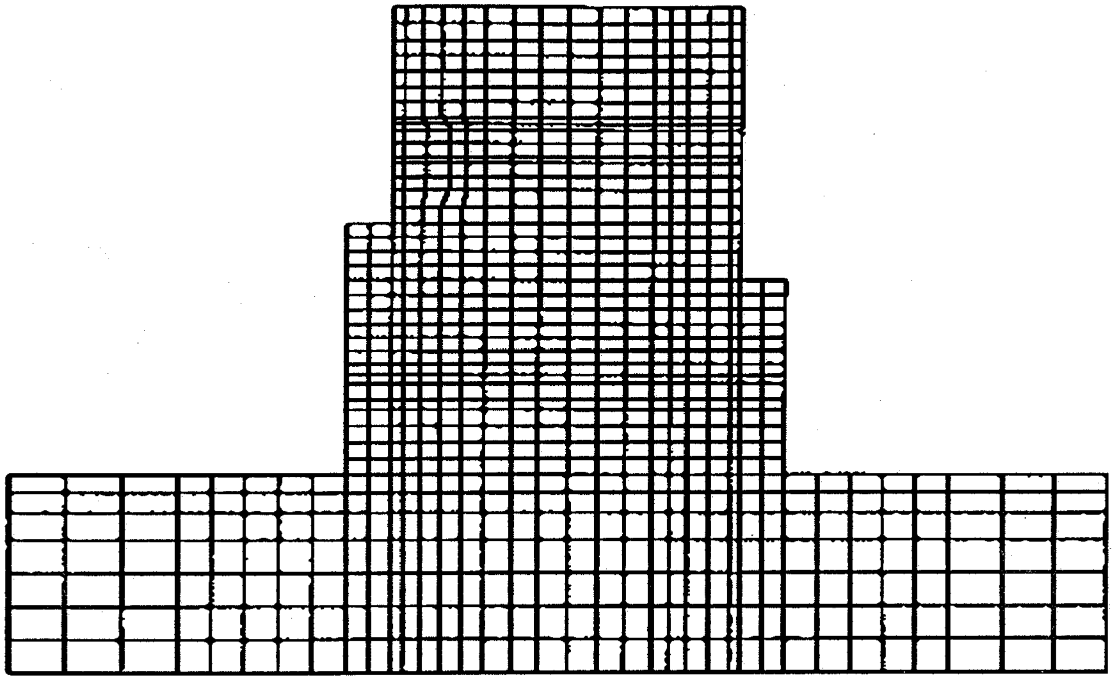

Figure A3-19. Finite element model of lock wall example

Table A3-5
Summary of Surface Heat Transfer Coefficients For FE Thermal Analyses

	Wind Velocity	Surface Heat Transfer Coefficient - *h* W/m^2-K (Btu/day-in^2-deg F)			
Time Span Months	km/h (mi/hr)	Wind Velocity Only	Wind Velocity & Plywood	Wind Velocity & Insulation	Air, Plywood, & Insulation
Nov. - Apr.	16 (10)	25.72 (0.7548)	4.913 (0.1442)	1.345 (0.03949)	1.101 (0.03233)
May - June	13 (8)	22.01 (0.6460)	4.763 (0.1398)	1.333 (0.03914)	1.094 (0.03210)
July - Sept.	11 (7)	19.71 (0.5785)	4.644 (0.1363)	1.324 (0.03887)	1.087 (0.03191)
Oct.	13 (8)	21.88 (0.6423)	4.756 (0.1396)	1.333 (0.03913)	1.093 (0.03209)

changes causing tension and compression are balanced.

(b) Determine reference temperatures. In the example, the reference time was established as 0.5 days after placement of lift 6 (25.5 days after concrete placement start at lift 1). Because the concrete attained a 1-day modulus of elasticity of 12.4 Gpa (1.8×10^6 psi), it was assumed that elastic strains were sustainable in this concrete at an age of 0.5 days.

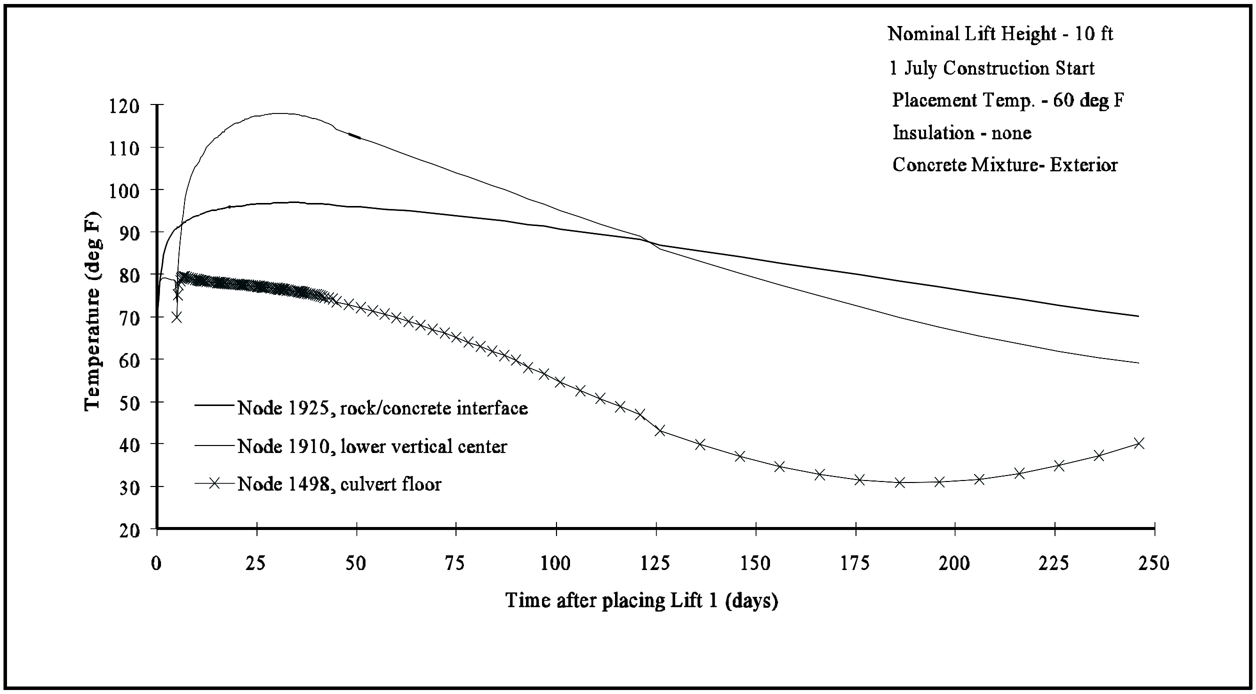

Figure A3-20. Typical temperature histories at locations of mass gradient analysis

(c) Determine temperature change or differences relative to the reference temperatures. Table A3-7 shows distributions of temperature difference at all analysis times relative to the reference temperatures at 0.5 days age of lift 6 (25.5 days after lift 1). These are developed by subtracting all of the temperatures in Table A3-6 from the respective 0.5-day temperatures at the same horizontal coordinates.

(d) Determine temperature differences relative to surface temperature differences, or "normalized" temperature differences. Table A3-8 and Figure A3-22 show temperature differences normalized relative to the surface temperature differences. These normalized temperature differences were developed by subtracting the surface temperature differences (along coordinates 4.0 and 36.0) in Table A3-7 from the corresponding interior temperature differences at the same time intervals in Table A3-7, producing the Table A3-8 normalized temperature differences.

(e) Determine offset balance temperatures. To balance tension and compression zones, a balance temperature, T_0, is determined such that the areas of the normalized temperature distribution above and below T_0 are equal. Table A3-9 and Figure A3-23 show balanced, normalized temperature differences.

(f) The depth of T_0 defines the depth of "*H*" of the tension block. A formula for the sums of individual areas between temperature points of the normalized temperature difference distribution across a section above and below T_0 was used for the determination of *H*. These calculations were solved by extensive computer spreadsheet analysis, resulting in tension block "*H*" values.

d. Cracking analysis.

(1) Step 7: Mass gradient cracking analysis. Mass gradient thermal strains are computed from Equation A-4 in Appendix A. Table A3-10 summarizes the computations.

(a) Foundation restraint factor (K_f). Foundation restraint, based upon relative differences in the

Table A3-6
Temperature Distributions in Lift 6 for Surface Gradient Analysis

Degrees C

Horizontal Coordinate (m)	Age of Concrete in Lift 6 placed 25 days after Lift 1 (days)												
	0.5	1	2	3	5	7	14	29	59	91	121	151	181
	Elapsed Time (T) after Placement of Lift 1 (days)												
	25.5	26	27	28	30	32	39	54	84	116	146	176	206
1.2	23.3	27.8	30.5	26.6	25.8	25.4	24.4	22.3	17.0	10.1	4.1	0.1	-0.9
1.4	23.0	28.5	32.4	30.3	29.2	28.5	26.8	24.1	18.2	11.2	5.1	0.9	-0.4
1.5	22.6	29.2	34.2	34.1	32.7	31.6	29.2	25.8	19.4	12.4	6.2	1.8	0.1
1.8	22.4	29.2	35.4	37.2	37.2	36.2	33.3	28.9	21.6	14.5	8.1	3.3	1.0
2.1	22.4	29.2	35.6	38.3	39.7	39.3	36.5	31.7	23.7	16.3	9.8	4.7	1.9
2.4	22.4	29.2	35.7	38.7	41.0	41.2	39.0	34.0	25.5	18.0	11.4	6.0	2.8
2.7	22.4	29.2	35.7	38.8	41.5	42.3	40.8	36.0	27.2	19.6	12.9	7.3	3.7
3.0	22.4	29.2	35.7	38.8	41.7	42.7	41.9	37.5	28.6	20.8	14.0	8.2	4.4
3.2	22.4	29.2	35.7	38.8	41.8	43.0	42.7	38.8	29.9	22.0	15.1	9.2	5.1
3.5	22.4	29.2	35.7	38.8	41.9	43.1	43.4	40.0	31.2	23.2	16.2	10.2	5.8
3.7	22.4	29.2	35.7	38.8	41.9	43.2	43.9	41.1	32.4	24.3	17.3	11.1	6.6
4.1	22.4	29.2	35.7	38.8	41.9	43.3	44.5	42.6	34.3	26.0	18.9	12.5	7.7
4.5	22.4	29.2	35.7	38.8	41.9	43.3	44.9	44.0	36.0	27.6	20.4	13.9	8.7
4.9	22.4	29.2	35.7	38.8	41.9	43.3	45.2	45.0	37.5	29.1	21.7	15.1	9.7
5.3	22.4	29.2	35.7	38.8	41.9	43.3	45.4	45.9	38.8	30.3	22.9	16.1	10.5
5.7	22.4	29.2	35.7	38.8	41.9	43.3	45.4	46.5	39.9	31.4	23.8	17.0	11.2
6.1	22.4	29.2	35.7	38.8	41.9	43.3	45.5	47.0	40.6	32.1	24.5	17.6	11.7
6.5	22.4	29.2	35.7	38.8	41.9	43.3	45.5	47.2	41.0	32.5	24.9	17.9	12.0
6.9	22.4	29.2	35.7	38.8	41.9	43.3	45.5	47.1	41.0	32.5	24.9	17.9	12.0
7.3	22.4	29.2	35.7	38.8	41.9	43.3	45.4	46.9	40.7	32.2	24.6	17.7	11.8
7.7	22.4	29.2	35.7	38.8	41.9	43.3	45.3	46.4	39.9	31.5	24.0	17.1	11.3
8.1	22.4	29.2	35.7	38.8	41.9	43.3	45.1	45.7	38.8	30.5	23.0	16.2	10.6
8.5	22.4	29.2	35.7	38.8	41.9	43.2	44.7	44.5	37.3	29.1	21.7	15.1	9.7
8.7	22.4	29.2	35.7	38.8	41.9	43.2	44.3	43.6	36.2	28.0	20.7	14.2	9.0
9.0	22.4	29.2	35.7	38.8	41.8	43.0	43.7	42.4	34.8	26.7	19.5	13.1	8.2
9.2	22.4	29.2	35.7	38.8	41.7	42.8	42.9	41.0	33.3	25.3	18.2	12.0	7.3
9.4	22.4	29.2	35.7	38.8	41.5	42.3	41.7	39.3	31.6	23.8	16.8	10.7	6.4
9.8	22.4	29.2	35.7	38.7	41.0	41.2	39.7	36.8	29.3	21.6	14.8	9.0	5.1
10.1	22.4	29.2	35.6	38.3	39.7	39.3	37.1	34.0	26.7	19.2	12.5	7.1	3.7
10.4	22.4	29.2	35.4	37.2	37.2	36.2	33.7	30.6	23.8	16.5	10.0	5.0	2.3
10.7	22.6	29.2	34.2	34.1	32.7	31.6	29.4	26.8	20.7	13.6	7.3	2.7	0.8
10.8	22.9	28.9	32.8	31.0	29.5	28.7	27.0	24.7	19.0	12.0	5.8	1.5	0.0
11.0	23.3	27.8	30.5	26.6	25.8	25.4	24.4	22.6	17.2	10.3	4.3	0.3	-0.7

Degrees F

Horizontal Coordinate (ft)	Age of Concrete in Lift 6 placed 25 days after Lift 1 (days)												
	0.5	1	2	3	5	7	14	29	59	91	121	151	181
	Elapsed Time (T) after Placement of Lift 1 (days)												
	25.5	26	27	28	30	32	39	54	84	116	146	176	206
4.00	73.9	82.1	87.0	79.8	78.5	77.7	75.9	72.2	62.5	50.2	39.4	32.2	30.4
4.50	73.3	83.4	90.3	86.6	84.6	83.3	80.2	75.4	64.7	52.2	41.3	33.7	31.3
5.00	72.7	84.6	93.6	93.3	90.8	88.8	84.6	78.5	66.9	54.3	43.2	35.2	32.1
6.00	72.3	84.6	95.7	99.0	99.0	97.2	91.9	84.1	70.9	58.0	46.6	37.9	33.8
7.00	72.3	84.5	96.2	101.0	103.5	102.8	97.7	89.0	74.6	61.4	49.7	40.5	35.4
8.00	72.3	84.5	96.3	101.6	105.8	106.2	102.1	93.2	77.9	64.4	52.5	42.8	37.0
9.00	72.3	84.5	96.3	101.8	106.7	108.1	105.4	96.8	81.0	67.3	55.1	45.1	38.6
9.81	72.3	84.5	96.3	101.9	107.1	108.9	107.4	99.5	83.5	69.5	57.2	46.8	39.9
10.63	72.3	84.5	96.3	101.9	107.3	109.4	108.9	101.9	85.8	71.6	59.2	48.6	41.2
11.44	72.3	84.5	96.3	101.9	107.4	109.7	110.1	104.0	88.1	73.7	61.1	50.3	42.5
12.25	72.3	84.5	96.3	101.9	107.4	109.8	111.0	106.0	90.3	75.8	63.1	52.0	43.8
13.50	72.3	84.5	96.3	101.9	107.4	109.9	112.1	108.8	93.7	78.9	66.0	54.5	45.8
14.75	72.3	84.5	96.3	101.9	107.4	109.9	112.8	111.1	96.7	81.8	68.7	57.0	47.7
16.00	72.3	84.5	96.3	101.9	107.4	109.9	113.3	113.1	99.5	84.4	71.1	59.1	49.4
17.25	72.3	84.5	96.3	101.9	107.4	109.9	113.6	114.6	101.8	86.6	73.2	61.0	50.9
18.58	72.3	84.5	96.3	101.9	107.4	109.9	113.8	115.8	103.8	88.5	74.9	62.6	52.2
19.92	72.3	84.5	96.3	101.9	107.4	109.9	113.9	116.5	105.1	89.8	76.1	63.6	53.1
21.25	72.3	84.5	96.3	101.9	107.4	109.9	113.9	116.9	105.8	90.4	76.8	64.2	53.5
22.58	72.3	84.5	96.3	101.9	107.4	109.9	113.9	116.8	105.8	90.5	76.8	64.3	53.6
23.88	72.3	84.5	96.3	101.9	107.4	109.9	113.8	116.4	105.2	89.9	76.3	63.8	53.2
25.17	72.3	84.5	96.3	101.9	107.4	109.9	113.6	115.6	103.9	88.7	75.2	62.8	52.3
26.46	72.3	84.5	96.3	101.9	107.4	109.9	113.3	114.2	101.9	86.9	73.4	61.2	51.1
27.75	72.3	84.5	96.3	101.9	107.4	109.8	112.5	112.1	99.2	84.3	71.1	59.1	49.4
28.56	72.3	84.5	96.3	101.9	107.4	109.7	111.8	110.4	97.1	82.4	69.3	57.5	48.2
29.38	72.3	84.5	96.3	101.9	107.3	109.4	110.7	108.3	94.7	80.1	67.2	55.7	46.8
30.19	72.3	84.5	96.3	101.9	107.1	109.0	109.1	105.7	92.0	77.6	64.8	53.6	45.2
31.00	72.3	84.5	96.3	101.8	106.8	108.1	107.0	102.7	89.0	74.8	62.2	51.3	43.5
32.00	72.3	84.5	96.3	101.6	105.8	106.2	103.5	98.3	84.8	70.9	58.6	48.2	41.2
33.00	72.3	84.5	96.2	101.0	103.5	102.8	98.8	93.1	80.1	66.6	54.5	44.7	38.7
34.00	72.3	84.6	95.7	99.0	99.0	97.2	92.6	87.1	74.9	61.8	50.0	41.0	36.1
35.00	72.7	84.6	93.6	93.3	90.8	88.8	85.0	80.2	69.2	56.4	45.1	36.9	33.4
35.50	73.3	84.0	91.0	87.8	85.2	83.6	80.7	76.5	66.2	53.6	42.5	34.8	32.1
36.00	73.9	82.1	87.0	79.8	78.5	77.7	76.0	72.6	63.0	50.6	39.7	32.5	30.7

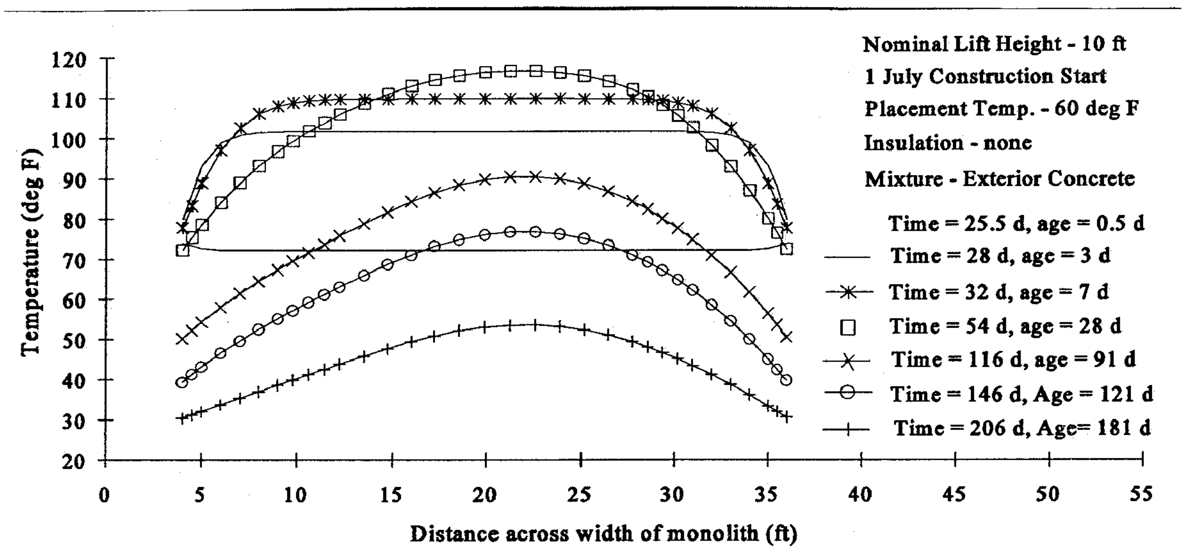

Figure A3-21. Temperature distributions across lift 6 used in surface gradient analysis

stiffness of the foundation material and the concrete, is computed from Equation A-7 in Appendix A as shown below.

$$K_f = \frac{1}{1 + \frac{A_g E_c}{A_f E_f}} = 0.64$$

where

A_g = gross area of concrete cross section (relative value) = 1

A_f = 2.5 (area of foundation or zone restraining contraction of concrete, generally as a plane surface at contact, recommended maximum value is 2.5)

E_f = modulus of elasticity of foundation = 48.3 Gpa (7.0×10^6 psi)

E_c = modulus of elasticity of mass concrete (mean value during cooling period) = 34.5 Gpa (5.0×10^6 psi)

(b) Structure restraint factor (K_R). Structure restraint factors are computed at distances, *h*, along the vertical centerline of the structure at h = 3.5 m (11.5 ft) and at $h = H = 7.0$ m (23 ft) at the base of the culvert. The length, *L*, of the structure is assumed to be 13.4 m (44 ft) in the axial direction. Note that the mass gradient analysis shown below assumes that the foundation restraint is applied by the foundation material adjacent to the concrete. Therefore, the foundation temperatures used in the analysis are taken at the foundation-concrete interface rather than at the location of constant foundation temperature at a depths of 6.1 m (20 ft) or more.

Using Equation A-6 (Appendix A) for *L/H* less than 2.5

$$K_R = K_f \left(\frac{\frac{L}{H} - 1}{\frac{L}{H} + 10} \right)^{h/H} = 0.28$$

Table A3-7
Temperature Differences Referenced to Temperature at 0.5 Days

Degrees C

Horizontal Coordinate (m)	Age of Concrete in Lift 6 placed 25 days after Lift 1 (days)												
	0.5	1	2	3	5	7	14	29	59	91	121	151	181
	Elapsed Time (T) after Placement of Lift 1 (days)												
	25.5	26	27	28	30	32	39	54	84	116	146	176	206
1.2	0.0	4.6	7.2	3.3	2.5	2.1	1.1	-1.0	-6.3	-13.2	-19.2	-23.2	-24.2
1.4	0.0	5.6	9.4	7.4	6.3	5.5	3.9	1.1	-4.8	-11.7	-17.8	-22.0	-23.4
1.5	0.0	6.6	11.6	11.4	10.1	9.0	6.6	3.2	-3.2	-10.2	-16.4	-20.8	-22.6
1.8	0.0	6.8	13.0	14.8	14.8	13.8	10.9	6.5	-0.8	-7.9	-14.3	-19.1	-21.4
2.1	0.0	6.8	13.3	16.0	17.4	17.0	14.1	9.3	1.3	-6.1	-12.5	-17.7	-20.5
2.4	0.0	6.8	13.3	16.3	18.6	18.9	16.6	11.6	3.1	-4.4	-11.0	-16.4	-19.6
2.7	0.0	6.8	13.3	16.4	19.2	19.9	18.4	13.7	4.9	-2.8	-9.5	-15.1	-18.7
3.0	0.0	6.8	13.3	16.4	19.4	20.4	19.5	15.1	6.2	-1.5	-8.4	-14.1	-18.0
3.2	0.0	6.8	13.3	16.5	19.5	20.6	20.4	16.4	7.5	-0.4	-7.3	-13.2	-17.2
3.5	0.0	6.8	13.3	16.5	19.5	20.8	21.0	17.6	8.8	0.8	-6.2	-12.2	-16.5
3.7	0.0	6.8	13.3	16.5	19.5	20.9	21.5	18.8	10.0	2.0	-5.1	-11.3	-15.8
4.1	0.0	6.8	13.3	16.5	19.5	20.9	22.1	20.3	11.9	3.7	-3.5	-9.8	-14.7
4.5	0.0	6.8	13.3	16.5	19.5	20.9	22.6	21.6	13.6	5.3	-2.0	-8.5	-13.6
4.9	0.0	6.8	13.3	16.5	19.5	20.9	22.8	22.7	15.1	6.7	-0.6	-7.3	-12.7
5.3	0.0	6.8	13.3	16.5	19.5	20.9	23.0	23.5	16.4	8.0	0.5	-6.3	-11.9
5.7	0.0	6.8	13.3	16.5	19.5	20.9	23.1	24.2	17.5	9.0	1.5	-5.4	-11.2
6.1	0.0	6.8	13.3	16.5	19.5	20.9	23.1	24.6	18.2	9.7	2.2	-4.8	-10.7
6.5	0.0	6.8	13.3	16.5	19.5	20.9	23.1	24.8	18.6	10.1	2.5	-4.5	-10.4
6.9	0.0	6.8	13.3	16.5	19.5	20.9	23.1	24.8	18.6	10.1	2.5	-4.4	-10.4
7.3	0.0	6.8	13.3	16.5	19.5	20.9	23.1	24.6	18.3	9.8	2.3	-4.7	-10.6
7.7	0.0	6.8	13.3	16.5	19.5	20.9	23.0	24.1	17.6	9.1	1.6	-5.3	-11.1
8.1	0.0	6.8	13.3	16.5	19.5	20.9	22.8	23.3	16.5	8.1	0.7	-6.1	-11.8
8.5	0.0	6.8	13.3	16.5	19.5	20.9	22.4	22.2	15.0	6.7	-0.6	-7.3	-12.7
8.7	0.0	6.8	13.3	16.5	19.5	20.8	22.0	21.2	13.8	5.6	-1.7	-8.2	-13.4
9.0	0.0	6.8	13.3	16.5	19.5	20.7	21.3	20.0	12.5	4.4	-2.8	-9.2	-14.2
9.2	0.0	6.8	13.3	16.4	19.4	20.4	20.5	18.6	11.0	3.0	-4.1	-10.4	-15.0
9.4	0.0	6.8	13.3	16.4	19.2	19.9	19.3	16.9	9.3	1.4	-5.6	-11.6	-16.0
9.8	0.0	6.8	13.3	16.3	18.6	18.9	17.4	14.5	7.0	-0.7	-7.6	-13.4	-17.3
10.1	0.0	6.8	13.3	16.0	17.4	16.9	14.7	11.6	4.3	-3.2	-9.9	-15.3	-18.6
10.4	0.0	6.8	13.0	14.8	14.8	13.8	11.3	8.2	1.4	-5.9	-12.4	-17.4	-20.1
10.7	0.0	6.6	11.6	11.4	10.1	9.0	6.8	4.2	-1.9	-9.0	-15.3	-19.9	-21.8
10.8	0.0	6.0	9.9	8.1	6.6	5.7	4.1	1.8	-3.9	-10.9	-17.1	-21.4	-22.9
11.0	0.0	4.6	7.2	3.3	2.5	2.1	1.2	-0.7	-6.0	-12.9	-19.0	-23.0	-24.0

Degrees F

Horizontal Coordinate (ft)	Age of Concrete in Lift 6 placed 25 days after Lift 1 (days)												
	0.5	1	2	3	5	7	14	29	59	91	121	151	181
	Elapsed Time (T) after Placement of Lift 1 (days)												
	25.5	26	27	28	30	32	39	54	84	116	146	176	206
4.00	0.0	8.2	13.0	5.9	4.5	3.8	2.0	-1.7	-11.4	-23.8	-34.6	-41.7	-43.5
4.50	0.0	10.1	16.9	13.2	11.3	10.0	6.9	2.0	-8.6	-21.1	-32.0	-39.6	-42.0
5.00	0.0	11.9	20.9	20.6	18.1	16.1	11.9	5.8	-5.8	-18.4	-29.5	-37.5	-40.6
6.00	0.0	12.3	23.4	26.7	26.6	24.8	19.5	11.8	-1.4	-14.3	-25.7	-34.4	-38.6
7.00	0.0	12.3	23.9	28.7	31.2	30.5	25.4	16.7	2.3	-10.9	-22.6	-31.8	-36.9
8.00	0.0	12.3	24.0	29.4	33.5	33.9	29.9	20.9	5.7	-7.8	-19.7	-29.4	-35.2
9.00	0.0	12.3	24.0	29.6	34.5	35.8	33.2	24.6	8.8	-5.0	-17.1	-27.2	-33.6
9.81	0.0	12.3	24.0	29.6	34.9	36.7	35.2	27.2	11.2	-2.8	-15.1	-25.4	-32.3
10.63	0.0	12.3	24.0	29.6	35.0	37.1	36.7	29.6	13.6	-0.6	-13.1	-23.7	-31.0
11.44	0.0	12.3	24.0	29.6	35.1	37.4	37.9	31.8	15.8	1.4	-11.1	-22.0	-29.7
12.25	0.0	12.3	24.0	29.6	35.1	37.5	38.8	33.8	18.1	3.5	-9.2	-20.3	-28.4
13.50	0.0	12.3	24.0	29.6	35.2	37.6	39.8	36.5	21.4	6.6	-6.3	-17.7	-26.4
14.75	0.0	12.3	24.0	29.6	35.2	37.7	40.6	38.9	24.5	9.5	-3.6	-15.3	-24.5
16.00	0.0	12.3	24.0	29.6	35.2	37.7	41.1	40.8	27.2	12.1	-1.2	-13.1	-22.8
17.25	0.0	12.3	24.0	29.6	35.2	37.7	41.4	42.4	29.6	14.3	0.9	-11.3	-21.3
18.58	0.0	12.3	24.0	29.6	35.2	37.7	41.6	43.5	31.5	16.2	2.7	-9.7	-20.1
19.92	0.0	12.3	24.0	29.6	35.2	37.7	41.6	44.3	32.8	17.5	3.9	-8.6	-19.2
21.25	0.0	12.3	24.0	29.6	35.2	37.7	41.6	44.6	33.5	18.2	4.5	-8.0	-18.7
22.58	0.0	12.3	24.0	29.6	35.2	37.7	41.6	44.6	33.6	18.3	4.6	-8.0	-18.7
23.88	0.0	12.3	24.0	29.6	35.2	37.7	41.6	44.2	32.9	17.7	4.1	-8.5	-19.1
25.17	0.0	12.3	24.0	29.6	35.2	37.7	41.4	43.4	31.6	16.5	2.9	-9.5	-19.9
26.46	0.0	12.3	24.0	29.6	35.2	37.6	41.0	42.0	29.7	14.6	1.2	-11.0	-21.2
27.75	0.0	12.3	24.0	29.6	35.1	37.6	40.3	39.9	27.0	12.1	-1.2	-13.1	-22.8
28.56	0.0	12.3	24.0	29.6	35.1	37.4	39.5	38.2	24.9	10.1	-3.0	-14.7	-24.1
29.38	0.0	12.3	24.0	29.6	35.0	37.2	38.4	36.0	22.5	7.9	-5.1	-16.6	-25.5
30.19	0.0	12.3	24.0	29.6	34.9	36.7	36.9	33.5	19.7	5.4	-7.4	-18.6	-27.1
31.00	0.0	12.3	24.0	29.6	34.5	35.9	34.8	30.5	16.7	2.6	-10.0	-20.9	-28.8
32.00	0.0	12.3	24.0	29.4	33.5	34.0	31.3	26.1	12.5	-1.3	-13.7	-24.1	-31.1
33.00	0.0	12.3	23.9	28.7	31.3	30.5	26.5	20.8	7.8	-5.7	-17.7	-27.5	-33.5
34.00	0.0	12.3	23.4	26.7	26.6	24.8	20.3	14.8	2.6	-10.6	-22.3	-31.3	-36.2
35.00	0.0	11.9	20.9	20.6	18.1	16.1	12.3	7.5	-3.5	-16.3	-27.6	-35.8	-39.3
35.50	0.0	10.8	17.8	14.5	11.9	10.3	7.4	3.2	-7.1	-19.7	-30.8	-38.5	-41.2
36.00	0.0	8.2	13.0	5.9	4.5	3.8	2.1	-1.3	-10.9	-23.3	-34.2	-41.4	-43.2

Table A3-8
Temperature Differences Normalized in Reference to Surface Temperature Differences For Surface Gradient Analysis

Degrees C

Horizontal Coordinate (m)	Age of Concrete in Lift 6 placed 25 days after Lift 1 (days)												
	0.5	1	2	3	5	7	14	29	59	91	121	151	181
	Elapsed Time (T) after Placement of Lift 1 (days)												
	25.5	26	27	28	30	32	39	54	84	116	146	176	206
1.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.4	0.0	1.0	2.2	4.1	3.8	3.4	2.8	2.1	1.6	1.5	1.4	1.2	0.8
1.5	0.0	2.1	4.3	8.2	7.6	6.9	5.5	4.2	3.1	3.0	2.8	2.3	1.6
1.8	0.0	2.3	5.7	11.6	12.3	11.7	9.8	7.5	5.6	5.3	4.9	4.0	2.7
2.1	0.0	2.3	6.0	12.7	14.8	14.9	13.0	10.2	7.6	7.1	6.7	5.5	3.7
2.4	0.0	2.3	6.1	13.1	16.1	16.8	15.5	12.6	9.5	8.8	8.2	6.8	4.6
2.7	0.0	2.3	6.1	13.2	16.6	17.8	17.3	14.6	11.2	10.4	9.7	8.1	5.5
3.0	0.0	2.3	6.1	13.2	16.8	18.3	18.4	16.1	12.6	11.6	10.8	9.0	6.2
3.2	0.0	2.3	6.1	13.2	16.9	18.5	19.3	17.4	13.9	12.8	11.9	10.0	6.9
3.5	0.0	2.3	6.1	13.2	17.0	18.7	19.9	18.6	15.1	14.0	13.0	11.0	7.6
3.7	0.0	2.3	6.1	13.2	17.0	18.8	20.4	19.7	16.4	15.1	14.1	11.9	8.4
4.1	0.0	2.3	6.1	13.2	17.0	18.8	21.0	21.2	18.2	16.9	15.7	13.3	9.5
4.5	0.0	2.3	6.1	13.2	17.0	18.8	21.5	22.6	19.9	18.5	17.2	14.7	10.5
4.9	0.0	2.3	6.1	13.2	17.0	18.8	21.7	23.6	21.5	19.9	18.6	15.9	11.5
5.3	0.0	2.3	6.1	13.2	17.0	18.8	21.9	24.5	22.8	21.2	19.7	16.9	12.3
5.7	0.0	2.3	6.1	13.2	17.0	18.8	22.0	25.1	23.8	22.2	20.7	17.8	13.0
6.1	0.0	2.3	6.1	13.2	17.0	18.8	22.0	25.5	24.6	22.9	21.4	18.4	13.5
6.5	0.0	2.3	6.1	13.2	17.0	18.8	22.0	25.7	24.9	23.3	21.7	18.7	13.7
6.9	0.0	2.3	6.1	13.2	17.0	18.8	22.0	25.7	25.0	23.3	21.7	18.7	13.8
7.3	0.0	2.3	6.1	13.2	17.0	18.8	22.0	25.5	24.6	23.0	21.5	18.5	13.5
7.7	0.0	2.3	6.1	13.2	17.0	18.8	21.9	25.0	23.9	22.3	20.8	17.9	13.1
8.1	0.0	2.3	6.1	13.2	17.0	18.8	21.7	24.3	22.8	21.3	19.9	17.0	12.4
8.5	0.0	2.3	6.1	13.2	17.0	18.8	21.3	23.1	21.3	19.9	18.6	15.9	11.5
8.7	0.0	2.3	6.1	13.2	17.0	18.7	20.9	22.1	20.1	18.8	17.5	15.0	10.8
9.0	0.0	2.3	6.1	13.2	16.9	18.6	20.2	21.0	18.8	17.6	16.4	13.9	10.0
9.2	0.0	2.3	6.1	13.2	16.8	18.3	19.4	19.6	17.3	16.2	15.1	12.8	9.1
9.4	0.0	2.3	6.1	13.2	16.6	17.8	18.2	17.9	15.6	14.6	13.6	11.5	8.2
9.8	0.0	2.3	6.1	13.1	16.1	16.8	16.3	15.4	13.3	12.5	11.6	9.8	6.9
10.1	0.0	2.3	6.0	12.7	14.8	14.9	13.6	12.5	10.7	10.0	9.3	7.9	5.5
10.4	0.0	2.3	5.7	11.6	12.3	11.7	10.2	9.1	7.8	7.3	6.8	5.7	4.1
10.7	0.0	2.1	4.3	8.2	7.5	6.9	5.7	5.1	4.4	4.2	3.9	3.3	2.3
10.8	0.0	1.4	2.6	4.8	4.1	3.6	3.0	2.7	2.4	2.3	2.1	1.8	1.3
11.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.2	0.3	0.2	0.2	0.2	0.1

Degrees F

Horizontal Coordinate (ft)	Age of Concrete in Lift 6 placed 25 days after Lift 1 (days)												
	0.5	1	2	3	5	7	14	29	59	91	121	151	181
	Elapsed Time (T) after Placement of Lift 1 (days)												
	25.5	26	27	28	30	32	39	54	84	116	146	176	206
4.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4.50	0.0	1.9	3.9	7.4	6.8	6.2	5.0	3.8	2.8	2.7	2.5	2.1	1.4
5.00	0.0	3.7	7.8	14.7	13.6	12.4	9.9	7.5	5.6	5.4	5.0	4.2	2.9
6.00	0.0	4.1	10.3	20.8	22.1	21.1	17.6	13.5	10.0	9.5	8.8	7.3	4.9
7.00	0.0	4.1	10.8	22.8	26.7	26.7	23.5	18.4	13.7	12.9	12.0	9.9	6.6
8.00	0.0	4.1	11.0	23.5	29.0	30.2	27.9	22.7	17.1	15.9	14.8	12.3	8.2
9.00	0.0	4.1	11.0	23.7	30.0	32.1	31.2	26.3	20.2	18.8	17.5	14.5	9.8
9.81	0.0	4.1	11.0	23.7	30.3	32.9	33.2	28.9	22.6	21.0	19.5	16.3	11.1
10.63	0.0	4.1	11.0	23.7	30.5	33.4	34.7	31.3	24.9	23.1	21.5	18.0	12.4
11.44	0.0	4.1	11.0	23.7	30.6	33.6	35.9	33.5	27.2	25.2	23.5	19.7	13.7
12.25	0.0	4.1	11.0	23.7	30.6	33.8	36.8	35.5	29.5	27.3	25.4	21.4	15.0
13.50	0.0	4.1	11.0	23.8	30.6	33.9	37.9	38.2	32.8	30.4	28.3	24.0	17.0
14.75	0.0	4.1	11.0	23.8	30.6	33.9	38.6	40.6	35.9	33.3	31.0	26.4	18.9
16.00	0.0	4.1	11.0	23.8	30.6	33.9	39.1	42.5	38.6	35.9	33.4	28.6	20.7
17.25	0.0	4.1	11.0	23.8	30.6	33.9	39.4	44.1	41.0	38.1	35.5	30.4	22.1
18.58	0.0	4.1	11.0	23.8	30.6	33.9	39.6	45.2	42.9	40.0	37.2	32.0	23.4
19.92	0.0	4.1	11.0	23.8	30.6	33.9	39.6	46.0	44.2	41.3	38.4	33.1	24.3
21.25	0.0	4.1	11.0	23.8	30.6	33.9	39.7	46.3	44.9	41.9	39.1	33.7	24.7
22.58	0.0	4.1	11.0	23.8	30.6	33.9	39.6	46.3	44.9	42.0	39.1	33.7	24.8
23.88	0.0	4.1	11.0	23.8	30.6	33.9	39.6	45.9	44.3	41.4	38.6	33.2	24.4
25.17	0.0	4.1	11.0	23.8	30.6	33.9	39.4	45.1	43.0	40.2	37.5	32.2	23.6
26.46	0.0	4.1	11.0	23.8	30.6	33.9	39.0	43.7	41.1	38.4	35.8	30.7	22.3
27.75	0.0	4.1	11.0	23.7	30.6	33.8	38.3	41.6	38.4	35.8	33.4	28.6	20.7
28.56	0.0	4.1	11.0	23.7	30.6	33.6	37.5	39.9	36.3	33.9	31.6	26.9	19.4
29.38	0.0	4.1	11.0	23.7	30.5	33.4	36.4	37.8	33.9	31.6	29.5	25.1	18.0
30.19	0.0	4.1	11.0	23.7	30.3	32.9	34.9	35.2	31.1	29.1	27.1	23.0	16.4
31.00	0.0	4.1	11.0	23.7	30.0	32.1	32.8	32.2	28.1	26.3	24.5	20.7	14.7
32.00	0.0	4.1	11.0	23.5	29.0	30.2	29.3	27.8	23.9	22.4	20.9	17.6	12.4
33.00	0.0	4.1	10.9	22.8	26.7	26.7	24.5	22.6	19.2	18.1	16.8	14.2	9.9
34.00	0.0	4.1	10.3	20.8	22.1	21.1	18.3	16.5	14.0	13.2	12.3	10.3	7.3
35.00	0.0	3.7	7.8	14.7	13.6	12.4	10.3	9.2	7.9	7.5	7.0	5.9	4.2
35.50	0.0	2.6	4.7	8.6	7.4	6.5	5.4	4.9	4.3	4.1	3.7	3.2	2.3
36.00	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.4	0.5	0.4	0.4	0.3	0.2

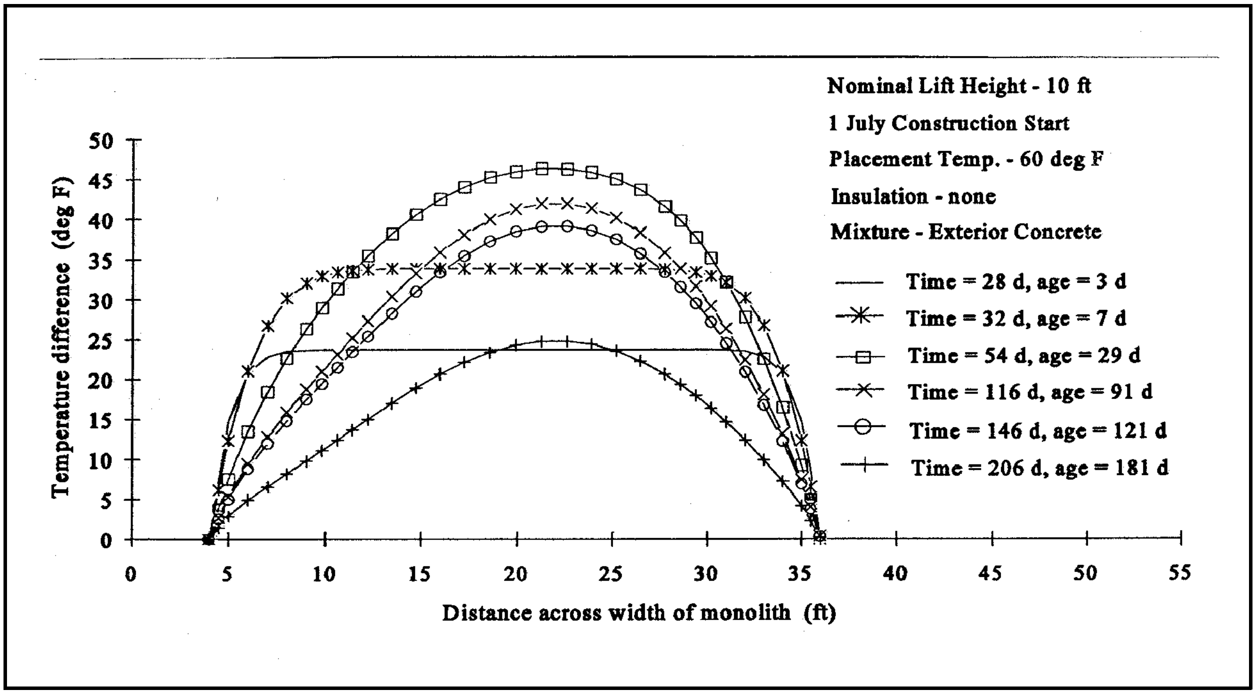

Figure A3-22. Temperature differences in lift 6 for surface gradient analysis

where

L/H = 13.4 m/7.0 m [44 ft / 23 ft] = 1.9

h/H = 3.5 m/ 7.0 m [11.5 ft / 23 ft] = 0.5

(c) Calculate tensile strains.

$\epsilon = (C_{th})(dT)(K_R)$ = 41 millionths

where

C_{th} = 10.5 millionths/deg C
(5.81 millionths/deg F)

dT = 13.9 deg C (25 deg F)

K_R = 0.28

(d) Estimate cracking. TSC information is shown in Table A3-11 for various ages. Comparison of mass gradient tensile strains with the slow-load TSC for equivalent time periods indicates no anticipated cracking under the given conditions.

(2) Step 8: Surface gradient cracking analysis. Table A3-11 presents the surface gradient cracking calculations. The upper portion of the table shows the determination of restraint factors based on time and location. The lower portion shows calculation of strains using Equation A-8 from Appendix A, and comparison of calculate strains with slow-load TSC values for the appropriate time period. Figure A3-24 compares the development of tensile strains at the lock wall surface and concrete TSC with time.

(a) Internal restraint factor (K_R). Internal restraint factors are based on the depth of the tension block, "H." "H" is determined from Table A3-9 by observing the depth where temperatures change from negative to positive, which shows where effective strains are balanced between tension and compression. These depths are shown in Table A3-11 as the tension block width. K_R is calculated based on Equation A-5, as shown in the table.

Table A3-9
Balanced or Effective Temperature Differences to Determine "*H*" and Surface Gradients Strains

Degrees C

Horizontal Coordinate (m)	Age of Concrete in Lift 6 placed 25 days after Lift 1 (days)												
	0.5	1	2	3	5	7	14	29	59	91	121	151	181
	Elapsed Time (T) after Placement of Lift 1 (days)												
	25.5	26	27	28	30	32	39	54	84	116	146	176	206
1.2	0.0	-2.2	-5.8	-12.3	-15.4	-16.6	-17.9	-18.5	-16.6	-15.5	-14.4	-12.3	-8.8
1.4	0.0	-1.2	-3.6	-8.2	-11.6	-13.1	-15.1	-16.4	-15.1	-14.0	-13.0	-11.1	-8.0
1.5	0.0	-0.1	-1.4	-4.2	-7.8	-9.7	-12.4	-14.3	-13.5	-12.5	-11.7	-10.0	-7.2
1.8	0.0	0.1	0.0	-0.8	-3.1	-4.9	-8.1	-11.0	-11.1	-10.2	-9.5	-8.3	-6.1
2.1	0.0	0.1	0.2	0.3	-0.5	-1.7	-4.8	-8.3	-9.0	-8.4	-7.8	-6.8	-5.2
2.4	0.0	0.1	0.3	0.7	0.7	0.2	-2.4	-5.9	-7.1	-6.7	-6.2	-5.5	-4.3
2.7	0.0	0.1	0.3	0.8	1.3	1.2	-0.5	-3.9	-5.4	-5.1	-4.7	-4.2	-3.4
3.0	0.0	0.1	0.3	0.8	1.5	1.7	0.6	-2.4	-4.1	-3.9	-3.6	-3.3	-2.7
3.2	0.0	0.1	0.3	0.9	1.6	2.0	1.4	-1.1	-2.8	-2.7	-2.5	-2.3	-1.9
3.5	0.0	0.1	0.3	0.9	1.6	2.1	2.1	0.1	-1.5	-1.5	-1.4	-1.3	-1.2
3.7	0.0	0.1	0.3	0.9	1.6	2.2	2.6	1.2	-0.2	-0.4	-0.3	-0.4	-0.5
4.1	0.0	0.1	0.3	0.9	1.6	2.3	3.2	2.7	1.6	1.4	1.3	1.0	0.6
4.5	0.0	0.1	0.3	0.9	1.6	2.3	3.6	4.0	3.3	3.0	2.8	2.4	1.7
4.9	0.0	0.1	0.3	0.9	1.6	2.3	3.9	5.1	4.8	4.4	4.1	3.6	2.6
5.3	0.0	0.1	0.3	0.9	1.6	2.3	4.0	6.0	6.1	5.7	5.3	4.6	3.5
5.7	0.0	0.1	0.3	0.9	1.6	2.3	4.1	6.6	7.2	6.7	6.2	5.5	4.2
6.1	0.0	0.1	0.3	0.9	1.6	2.3	4.2	7.0	7.9	7.4	6.9	6.1	4.6
6.5	0.0	0.1	0.3	0.9	1.6	2.3	4.2	7.2	8.3	7.8	7.3	6.4	4.9
6.9	0.0	0.1	0.3	0.9	1.6	2.3	4.2	7.2	8.4	7.8	7.3	6.4	4.9
7.3	0.0	0.1	0.3	0.9	1.6	2.3	4.1	7.0	8.0	7.5	7.0	6.2	4.7
7.7	0.0	0.1	0.3	0.9	1.6	2.3	4.0	6.5	7.3	6.8	6.4	5.6	4.3
8.1	0.0	0.1	0.3	0.9	1.6	2.3	3.8	5.8	6.2	5.8	5.4	4.7	3.6
8.5	0.0	0.1	0.3	0.9	1.6	2.2	3.4	4.6	4.7	4.4	4.1	3.6	2.6
8.7	0.0	0.1	0.3	0.9	1.6	2.1	3.0	3.6	3.5	3.3	3.1	2.7	1.9
9.0	0.0	0.1	0.3	0.9	1.6	2.0	2.4	2.5	2.2	2.1	1.9	1.6	1.1
9.2	0.0	0.1	0.3	0.8	1.5	1.7	1.5	1.0	0.7	0.7	0.6	0.5	0.3
9.4	0.0	0.1	0.3	0.8	1.3	1.3	0.4	-0.6	-1.0	-0.9	-0.8	-0.8	-0.7
9.8	0.0	0.1	0.3	0.7	0.7	0.2	-1.6	-3.1	-3.3	-3.0	-2.8	-2.5	-1.9
10.1	0.0	0.1	0.3	0.4	-0.5	-1.7	-4.2	-6.0	-5.9	-5.5	-5.1	-4.4	-3.3
10.4	0.0	0.1	0.0	-0.8	-3.1	-4.9	-7.7	-9.4	-8.9	-8.2	-7.6	-6.6	-4.8
10.7	0.0	-0.1	-1.4	-4.2	-7.8	-9.7	-12.1	-13.4	-12.2	-11.3	-10.6	-9.0	-6.5
10.8	0.0	-0.8	-3.1	-7.5	-11.3	-12.9	-14.9	-15.8	-14.2	-13.2	-12.4	-10.5	-7.6
11.0	0.0	-2.2	-5.8	-12.3	-15.4	-16.6	-17.8	-18.3	-16.3	-15.3	-14.2	-12.1	-8.7

Negative temperature differences produce tensile strain

Degrees F

Horizontal Coordinate (ft)	Age of Concrete in Lift 6 placed 25 days after Lift 1 (days)												
	0.5	1	2	3	5	7	14	29	59	91	121	151	181
	Elapsed Time (T) after Placement of Lift 1 (days)												
	25.5	26	27	28	30	32	39	54	84	116	146	176	206
4.00	0.0	-3.9	-10.4	-22.2	-27.7	-29.8	-32.2	-33.3	-29.9	-27.9	-26.0	-22.1	-15.9
4.50	0.0	-2.1	-6.5	-14.8	-20.9	-23.6	-27.2	-29.6	-27.1	-25.2	-23.5	-20.1	-14.5
5.00	0.0	-0.2	-2.6	-7.5	-14.1	-17.5	-22.2	-25.8	-24.3	-22.5	-21.0	-18.0	-13.0
6.00	0.0	0.2	-0.1	-1.4	-5.6	-8.8	-14.6	-19.8	-19.9	-18.4	-17.2	-14.9	-11.0
7.00	0.0	0.1	0.4	0.6	-1.0	-3.1	-8.7	-14.9	-16.2	-15.0	-14.0	-12.3	-9.3
8.00	0.0	0.1	0.6	1.3	1.3	0.4	-4.2	-10.7	-12.9	-12.0	-11.2	-9.9	-7.7
9.00	0.0	0.1	0.6	1.5	2.3	2.2	-1.0	-7.0	-9.7	-9.1	-8.5	-7.6	-6.1
9.81	0.0	0.1	0.6	1.5	2.7	3.1	1.0	-4.4	-7.3	-6.9	-6.5	-5.9	-4.8
10.63	0.0	0.1	0.6	1.5	2.8	3.6	2.6	-2.0	-5.0	-4.8	-4.5	-4.1	-3.5
11.44	0.0	0.1	0.6	1.5	2.9	3.8	3.7	0.2	-2.7	-2.7	-2.5	-2.4	-2.2
12.25	0.0	0.1	0.6	1.5	2.9	4.0	4.7	2.2	-0.4	-0.6	-0.6	-0.7	-0.9
13.50	0.0	0.1	0.6	1.5	3.0	4.1	5.7	4.9	2.9	2.5	2.3	1.8	1.1
14.75	0.0	0.1	0.6	1.5	3.0	4.1	6.5	7.3	6.0	5.4	5.0	4.2	3.0
16.00	0.0	0.1	0.6	1.5	3.0	4.1	7.0	9.2	8.7	8.0	7.4	6.4	4.7
17.25	0.0	0.1	0.6	1.5	3.0	4.1	7.3	10.7	11.0	10.2	9.5	8.3	6.2
18.58	0.0	0.1	0.6	1.5	3.0	4.1	7.4	11.9	13.0	12.1	11.2	9.8	7.5
19.92	0.0	0.1	0.6	1.5	3.0	4.1	7.5	12.7	14.3	13.4	12.5	10.9	8.4
21.25	0.0	0.1	0.6	1.5	3.0	4.1	7.5	13.0	15.0	14.0	13.1	11.5	8.8
22.58	0.0	0.1	0.6	1.5	3.0	4.1	7.5	13.0	15.0	14.1	13.2	11.6	8.9
23.88	0.0	0.1	0.6	1.5	3.0	4.1	7.4	12.6	14.4	13.5	12.6	11.1	8.5
25.17	0.0	0.1	0.6	1.5	3.0	4.1	7.2	11.7	13.1	12.3	11.5	10.1	7.7
26.46	0.0	0.1	0.6	1.5	3.0	4.1	6.9	10.4	11.1	10.5	9.8	8.5	6.4
27.75	0.0	0.1	0.6	1.5	2.9	4.0	6.1	8.3	8.4	7.9	7.4	6.4	4.7
28.56	0.0	0.1	0.6	1.5	2.9	3.8	5.4	6.5	6.3	6.0	5.6	4.8	3.5
29.38	0.0	0.1	0.6	1.5	2.8	3.6	4.3	4.4	3.9	3.7	3.5	3.0	2.1
30.19	0.0	0.1	0.6	1.5	2.7	3.1	2.8	1.9	1.2	1.2	1.1	0.9	0.5
31.00	0.0	0.1	0.6	1.5	2.3	2.3	0.7	-1.1	-1.8	-1.6	-1.5	-1.4	-1.2
32.00	0.0	0.1	0.6	1.3	1.3	0.4	-2.9	-5.5	-6.0	-5.5	-5.1	-4.5	-3.5
33.00	0.0	0.1	0.5	0.6	-1.0	-3.1	-7.6	-10.8	-10.7	-9.8	-9.2	-8.0	-6.0
34.00	0.0	0.2	-0.1	-1.4	-5.6	-8.8	-13.8	-16.9	-15.9	-14.7	-13.7	-11.8	-8.6
35.00	0.0	-0.2	-2.6	-7.5	-14.1	-17.5	-21.8	-24.1	-22.0	-20.4	-19.0	-16.3	-11.7
35.50	0.0	-1.4	-5.7	-13.6	-20.3	-23.3	-26.7	-28.4	-25.6	-23.8	-22.2	-19.0	-13.7
36.00	0.0	-3.9	-10.4	-22.2	-27.7	-29.8	-32.1	-32.9	-29.4	-27.5	-25.6	-21.8	-15.7

Negative temperature differences produce tensile strain

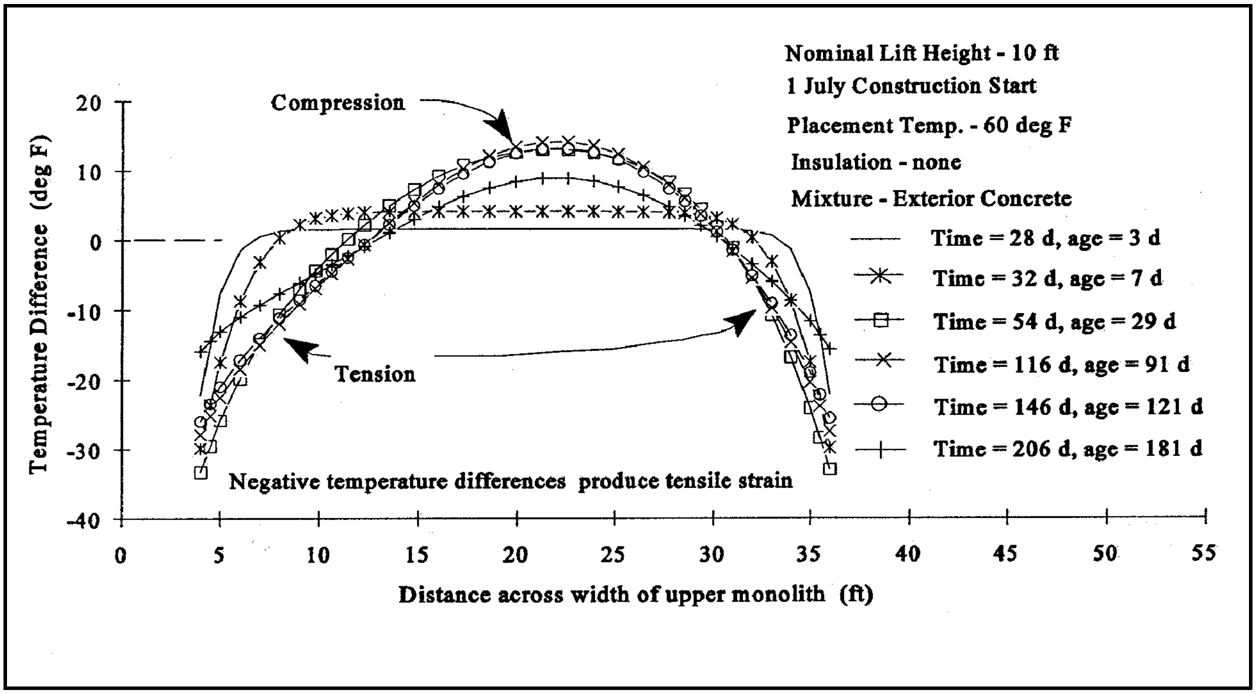

Figure A3-23. Balanced temperature difference distributions in lift 6 for surface gradient analysis

(b) Calculate tensile strains. Surface gradient tensile strains shown on Table A3-11, are based on the use of Equation A-8 (Appendix A), shown below:

$$\epsilon = (C_{th})(dT)(K_R) \qquad \text{(A-8)}$$

where

ϵ = induced tensile strain

C_{th} = coefficient of thermal expansion

dT = temperature difference with respect to interior temperature difference

K_R = internal restraint factor

dT is taken from the surface effective temperature differences in Table A3-9, at the exterior surfaces at each time period. These are shown on Table A3-11 for each lock wall face. For this example, only strains at the exterior surface are calculated and are hown on Table A3-11. Exterior surface strains are shown in this Table for K_R = 1.0, for comparison assuming the surface is completely restrained, and for various lengths (L = 11.0, 12.2, and 13.4 m)(L = 36, 40, and 44 ft) between vertical joints in the lock wall, where the surface restraint is less than 1.0. Strain variation with depth from the surface could be developed using corresponding K_R for interior locations.

(c) Estimate cracking. Comparison of strains with slow load TSC provides an estimation of where and when surface gradient cracking may develop, as shown in Table A3-11. The estimated depth of cracking could be evaluated using K_R at varying depths from the surface, and comparing with slow load TSC.

Table A3-10
Mass Gradient Cracking Aanalysis

1 July start, 15.5 deg C (60 deg F) placement temperature, no insulation, exterior mix

Analysis Location/ Node No.				Rock/Concrete Interface (Node 1925)			dT= dT(c)- dT(r)	Restraint Factor K_r	Thermal Strain	Slow Load TSC	Cracking yes/no
	T(max)	T(min)	dT©	T(max)	T(min)	dT(r)					
	deg C (deg F)	deg C (deg F)	deg C (deg F)	deg C (deg F)	deg C (deg F)	deg C (deg F)	deg C (deg F)	K_r = 0.64	millionths	millionths	
A / 1910	47.8 (118)	12.8 (55)	35.0 (63)	36.1 (97)	15.0 (59)	21.1 (38)	13.9 (25)	0.28	41	144	no
B / 1498	26.1 (79)	-0.6 (31)	26.7 (48)	33.3 (92)	25.5 (78)	7.8 (14)	18.9 (34)	0.08	16	144	no

Table A3-11
Surface Gradient Cracking Analysis

Example of Surface Gradient Analysis using Temperature Difference Distributions through Center of Lift 6												
3 m (10 ft) lifts, 1 July Start, 15.5 degC (60F) Placement Temperature, No Insulation, Cth = 10.5 millionths/degC (5.81 millionths/degF)												
Construction (days)		27	28	30	32	39	54	84	116	146	176	206
Concrete age (days)		2	3	5	7	14	29	59	91	121	151	181
Tension Block Width:			See Figure A3-9									
H(left)	m (ft)	0.6 (2.1)	0.8 (2.7)	1.0 (3.4)	1.2 (3.9)	1.6 (5.4)	2.2 (7.4)	2.6 (8.4)	2.6 (8.5)	2.6 (8.5)	2.6 (8.6)	2.7 (8.8)
H(right)	m (ft)	0.6 (2.1)	0.8 (2.7)	1.0 (3.4)	1.2 (3.9)	1.5 (4.8)	1.6 (5.3)	1.7 (5.5)	1.7 (5.5)	11.7 (5.5)	1.7 (5.5)	1.7 (5.6)
Monolith	Joint		RESTRAINT FACTORS KR AT SURFACES FOR L									
Analysis	Spacing											
Location	m (ft)		For L/H >= 2.5, Use equation Kr = [(L/H-2)/(L/H+1)]exp(h/H), where h=H at surface									
Left-side	11.0 (36)	0.83	0.79	0.74	0.71	0.61	0.49	0.43	0.43	0.43	0.42	0.41
Outer	12.2 (40)	0.85	0.81	0.76	0.73	0.64	0.53	0.48	0.47	0.47	0.47	0.46
Surface	13.4 (44)	0.86	0.83	0.78	0.76	0.67	0.57	0.52	0.51	0.51	0.51	0.50
Right-side	11.0 (36)	0.83	0.79	0.74	0.71	0.65	0.61	0.60	0.61	0.61	0.60	0.60
Outer	12.2 (40)	0.85	0.81	0.76	0.73	0.68	0.65	0.64	0.64	0.64	0.64	0.63
Surface	13.4 (44)	0.86	0.83	0.78	0.76	0.70	0.68	0.67	0.67	0.67	0.67	0.66
			EFFECTIVE TEMPERATURE DIFFERENCES AT SURFACE									
Eff. Temp. Diff. (Table A3-9)												
dT(left) (deg F)		-5.5 (-10)	-12.2 (-22)	-15.5 (-28)	-16.7 (-30)	-17.8 (-32)	-18.3 (-33)	-16.7 (-30)	-15.6 (-28)	-14.4 (-26)	-12.2 (-22)	-7.8 (-14)
dT(right) (deg F)		- 5.5 (-10)	-12.2 (-22)	-15.5 (-28)	-16.7 (-30)	-17.8 (-32)	18.3 (-33)	-16.1 (-29)	-15.0 (-27)	-14.4 (-26)	-12.2 (-22)	-7.8 (-14)
				SLOW LOAD TENSILE STRAIN CAPACITY								
concrete age (days)		2	3	5	7	14	28	61	90	125	155	185
slow load TSC (millionths)		86	95	104	108	116	124	134	140	144	146	149
Monolith	Joint			SURFACE TENSILE STRAIN CORRECTED FOR INTERNAL RESTRAINT (KR)								
Analysis	Spacing											
Location	m (ft)	(Assume cracking when tensile strains exceed slow-load tensile strain capacity (TSC) for respective age, indicated in bold)										
Left-side	11.0 (36)	50	**102**	**119**	**122**	114	95	75	69	64	54	38
Outer	12.2 (40)	51	**105**	**123**	**127**	**120**	103	83	77	71	60	42
Surface	13.4 (44)'	52	**107**	**126**	**131**	**126**	110	90	83	78	66	46
Right-side	11.0 (36)	50	**102**	**119**	**122**	**121**	119	105	98	91	78	55
Outer	12.2 (40)	51	**105**	**123**	**127**	**127**	**126**	111	104	97	82	59
Surface	13.4 (44)	52	**107**	**126**	**131**	**132**	**131**	116	108	101	86	61

e. Conclusions and recommendations. Some of the recommendations from this thermal study included the following:

(1) Maximum lift height = 1.5 m (5 ft).

(2) Maximum concrete placement temperature = 15.5 deg C (60 deg F) producing a 35.0 deg C (95 deg F) interior temperature.

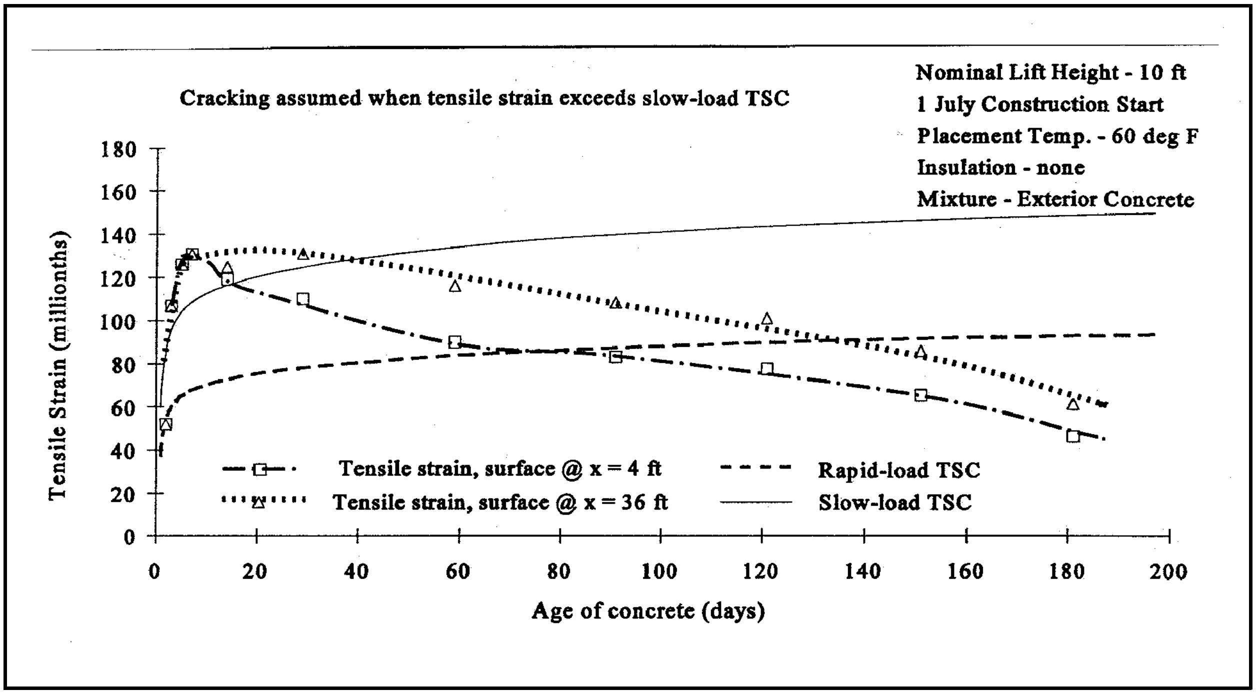

Figure A3-24. Evaluation of surface gradient cracking potential by comparing induced tensile strain with slow load tensile strain capacity

(3) Conduct additional mixture proportioning studies to further reduce the cement content.

(4) Insulate all exposed concrete surfaces placed between 15 October and 1 March.

(5) Remove insulation only when ambient temperatures are above mean daily temperatures, to aid thermal shock.

(6) Open culvert space to cool air slowly, to avoid thermal shock.

ANNEX 4: LIST OF REFERENCES

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