

CRACKING AND FAILURE OF HYDROELECTRIC ENGINEERING STRUCTURES DUE TO THERMAL LOADS

BY

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in Partial Fulfillment of the Requirements
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DOCTOR OF PHILOSOPHY

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A Thesis/Practicum submitted to the Faculty of Graduate Studies of The University
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ABSTRACT

Many hydraulic structures exhibit significant cracking, which allows water intrusion into or through the structures. These cracks are the result of a variety of phenomena, including thermal gradient, ice loading, restrained concrete shrinkage and cycles of freezing and thawing. Conventional repair methods generally consist of local sealing of cracks and defective construction joints by cementitious or chemical grouting. In many cases, such repair is effective only for a few years and additional measures are required. The study of the mechanism of structural failures and repair effects is very meaningful for the rehabilitation of hydraulic structures.

This thesis is devoted to the analyses of dams with cracked construction joints. Such structural failure may result from the destructive combination of harsh weather conditions and ice loads. Existing methods for transient temperature analysis of dams are first discussed and suggestions are made on the suitable choice of the initial and boundary conditions for the design of new dams and rehabilitation of cracked dams. The motion of dams with defective construction joints is analyzed as a contact problem. A highly accurate interface stress evaluation method is presented for the development of an effective algorithm of crack or joint open and close displacement analysis. The results of field measurement and theoretical analyses of ice formation, horizontal and vertical loads on structures are summarized. Practical ice loading models are established in terms of ambient temperature and other environment conditions.

The dam motion, especially the joint open-close displacement is numerically simulated. The results are calibrated through field measurements. The influence of the thermal and ice loads on the dam motion are studied. The mechanism of construction joint failure is clearly captured through the interface stress analysis results. Important conclusions are drawn for both repair work and anchor strengthening strategies

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Chapter 1

Introduction

1.1 Objective

The structures of electrical power generating stations include dams, powerhouses and spillways. In the north territory, these structures might be subjected to seasonal surface temperature variation of more than 70°C . Thermally induced cyclic stresses and freeze-thaw process have significant influences on the degradation of the strength and stiffness properties of the structures. Many dams, which were designed with less considerations of the thermal loads, ice loads and other environmental impacts, are showing severe deteriorations. Concrete cracking allows more water penetration and even leakage that accelerate the deterioration process. Today, the rehabilitation of the cracked structures is a pressing issue facing the engineering community. The research on the mechanism for structural degradation, the concrete material damage and cracking are inevitable topics for rehabilitation studies.

The nonlinear computational aspect of concrete structure is one of the most delicate difficult topics in engineering mechanics. The computational problems of failure analysis transcend the usual framework of well-posed initial or boundary value problems. Complicated numerical processes are needed to capture the behavior of concrete structures. Various kinds of discontinuities may happen in the concrete masses. Special

finite element configurations have to be invoked in order to trace the formation of weak and strong discontinuities.

The motion analysis of cracked concrete structures with an embedded discontinuity is an important part of the safety assessment and repair work. Advanced repair plans can be made based on reasonable analyses. The numerical simulation of repair practice helps us to know the repair effect. Efficient and less-costly rehabilitation schemes can be made based on good theoretical analysis. This makes the studies of great engineering significance.

The objective of this doctoral thesis is to develop efficient numerical modeling procedures to investigate the behaviors of dam structures with discontinuities under the environmental impact. The structure failure mechanism will be studied based on the numerical procedures. Repair strategies for cracked concrete massive structures will be assessed.

1.2 Outline of the thesis

This doctoral thesis consists of 8 chapters. With the present content as introductory chapter 1, a literature survey is provided in chapter 2. Topics related to the studies are covered in three aspects as: a) thermal analysis of dam structures; b) stress analysis method for dam structures with failures; c) ice load analysis method.

In chapter 3, the methods of transient temperature field analysis are discussed. The importance of the thermal frequency concept is emphasized for the choice of initial condition and the determination of convective boundary conditions. Numerical processes have been validated by correlation with the model of practical dam and field recorded data.

A new method for stress evaluation in finite element methods is proposed in chapter 4. High quality interface stresses or nodal stresses can be yielded by the new method. The analysis of dam structures with failure construction joint involves contact stress analysis. Based on the new stress evaluation method, an algorithm specially designed for the analysis of cracked dam structure analysis is developed in chapter 5. The structure response to thermal load is studied first as one part of the overall studies of the environment influence on the structures.

Chapter 6 is devoted to the ice load analysis. Models are introduced to simulate the forebay ice sheet formation, horizontal ice load and vertical ice load on the dam structures. The combined effects of thermal load and ice load are investigated. The stress level on the construction joint is analyzed to study the joint failure mechanism. The open displacement of a cracked joint is numerically predicted over a period of several years.

The repair strategy for cracked dam structures is assessed in chapter 7. The scheme for installation of anchors is numerically tested to find the anchor stress level and the dam behavior after anchor installation.

The thesis ends with the conclusion chapter 8, in which the research work and analysis results are summarized.

Chapter 2

Literature Survey

2.1 Introduction

Thermal stress analysis is a very important aspect of the safety and behavior study of dam structure. Transient temperature field analysis, thermal stress computation and the structure response to environmental factors are among the most interesting topics. Although the basic theory and computation methods have been established for many years, varieties of studies are still being carried on to treat many special engineering problems. Leger and Tinawi (1994) have listed a comprehensive collection of literature related to many aspects of dam analyses. For the present studies, the main concern lies in thermal and ice load analysis, and the behavior of dams under the action of these loads.

2.2 Thermal Analyses of Concrete Dams

Accurate transient thermal analysis of dam structures is a substantial requirement for design, rehabilitation and other structure analysis purpose. Heat transfer processes in dam structures are mainly controlled by the classic Fourier's heat transfer law. The main concern of thermal analyses of concrete dams is the integration of the heat transfer equation and the treatment of boundary and initial conditions.

2.2.1 Computational Procedures for Transient Temperature Field Analysis

Many methods have been developed to analyze the temperature field in dam structures. Finite difference method was used to obtain the temperature field in early time. Now, the finite element method (FEM) is the widely accepted numerical method for carrying out this task. The methodology in assessing seasonal temperature variation in gravity dams by finite element analysis has been described by Leger et al. (1993) and Armstrong (1993). The matrix form of the control equation about temperature T can be generally written as:

$$M\dot{T} + KT = Q \quad (2.1)$$

M is the heat capacity matrix, mainly depending on the density and specific heat capacity of dam material. K is the thermal conductivity matrix, which generally consists of two parts: heat conductivity and boundary heat exchange. Q is the heat source term, which may include convective heat transfer, surface solar radiation, hydration heat as well as other external heat supplies. To account the various boundary heat exchange phenomena and heat of hydration, nonlinearity and incremental solution procedures need be involved. For practical engineering problems, simplifications have to be made to deal with these complicated situations to obtain linearized equations in many cases. Direct integration of the heat transfer equations is well accepted as an efficient method to follow the transient variations of the temperature field inside a dam. The period of interest, in which the field temperature is required for a dam, is usually several years after the creation of the structures. The half day and day average schemes are often used (Venturelli and Leger

1992, Leger et al. 1993 and Leger et al. 1995) in practice. More computational problems come from the treatment of boundary conditions and initial conditions.

2.2.2 Boundary Conditions for Heat transfer

The boundary of a dam consists of dam-air boundary, dam-water boundary and dam-foundation boundary. The basic information required to perform a thermal analysis includes 1) weather data describing the air temperature, solar radiation, wind speed and snow cover; 2) water temperature and ice cover; 3) foundation temperature; and finally 4) thermal properties of the materials and the hydration characteristics of cementing materials. Suitable computational models have been established to treat these factors (Venturelli et al. 1992, Leger et al. 1993, and Armstrong 1993).

Although in some cases the foundation is taken as part of the dam structure, an adiabatic condition is more often assumed at the dam-foundation interface. Because concrete is a fair insulator, this kind of simplification does not influence significantly the thermal response away from the interface (Leger et. al, 1993). The dam-water boundary is usually treated as a temperature prescribed boundary. There is a small difference between the temperature of water and dam along the dam-water interface, no matter whether water flows or not (Venturelli et al. 1992). This means that the temperature gradient is very small on the interface. Therefore, it is assumed that the water temperature is the same as the temperature of concrete.

The air-dam boundary is always a difficult problem to deal with. A large difference exists between the temperature of the air and concrete. Heat exchanges

convectively along this boundary. The amount of heat exchanged with the air can be determined by Newton's law of cooling:

$$Q = -h_c(T_a - T) \quad (2.2)$$

in which h_c is the convective coefficient; T_a is the air temperature; T is the temperature of the surface. Difficulties rise from the determination of the convective coefficient. Because of the air flow and snow cover, the convective coefficient may be environment dependent. A larger convective coefficient will make the surface temperature more close to the air temperature.

Besides heat exchange with the air, solar radiation is also an important factor to influence the dam surface temperature. The amount of solar radiation is measured by the Stefan-Boltzmann law as:

$$Q = -C_s e(T_a^4 - T^4), \quad (2.3)$$

where e is the emissivity of the surface, C_s is the Stefan-Boltzmann constant. A quasi-linear form is often used (Elbdry and Ghali 1983, Mirambell and Aguado 1989):

$$Q = -h_r(T_a - T) \quad (2.4)$$

where the radiation coefficient h_r is defined as:

$$h_r = C_s e(T_a^2 + T^2)(T_a + T) \quad (2.5)$$

This quasi-linear form is similar to the Newton's Law of cooling. Therefore, a new convective coefficient can be obtained by combining h_c and h_r . In some cases, an average value can be used for h_r (Dilger et al. 1983) to linearize the problem.

2.2.3 Heat of Hydration

During the casting of massive concrete, the hydration of cement is accompanied by liberation of heat. Concrete is a good insulator, and in a large mass such as a dam, the heat generated in setting and hardening is not readily dissipated. Due to hydration of the cement in concrete, a temperature rise will take place in the concrete after placement. The peak temperature is generally reached between 7 to 20 days, depending on the thickness of section, the ambient temperature, the continued heat of hydration as well as other factors. The cooling and contraction of the hardened concrete may cause high residual stresses and serious cracks. Finite element modeling algorithms were developed for incrementally constructed concrete structures by Machida et al. (1987), Ishikawa (1991) and Armstrong (1993). A high temperature can be maintained for several weeks. The temperature will drop slowly to a stable state temperature over a period of several years. After the concrete construction phase, the heat of hydration can be neglected for long-term analysis of structure thermal response.

2.2.4 Initial Condition of the Transient Temperature Analysis

The initial condition is necessary for the transient temperature field analysis of a dam structure. Usually the initial data is not available unless the time history of dam is known. The common method for prescribing the initial temperature field is to assume a certain type of steady temperature solution as the initial condition (Venturelli et al 1992, Leger et al. 1995). Boundary temperature variations of certain period are repeated in time. When a repetitive temperature time history of a 2 to 3 year simulation is observed, the transient heat transfer analysis is considered to be convergent. Only the results after

convergence is achieved seem reliable for practical use. A finite period of time is required beyond which errors introduced by the assumed initial field temperature to drop to a negligible level.

2.2.5 Thermal Load on Structures

The thermal stress inside a massive concrete structure is mainly caused by the restrained volume changes of the concrete. The restraint can be either external like the structure constraint or an internal one that is due to the non-uniform deformation. Generally, the thermal stresses σ are related to the structure strain ϵ and temperature field T by

$$\{\sigma\} = [D] \{ \epsilon - \alpha(T - T_0) \mathbf{I} \} \quad (2.6)$$

where $[D]$ is the stiffness matrix; α is the heat expansion coefficient; T_0 is a reference temperature and $\mathbf{I}$ is a constant vector. Therefore, the magnitude of the thermal stresses not only depends on the temperature field, but also on the material properties of the concrete. In large temperature variation range, the concrete properties may change as functions of temperature themselves. In another respect, the cracks and the creep deformation of the concrete can also make the concrete properties change. An effective stiffness, defined as

$$[D_e] = \frac{1}{(1 + \phi)} [D] \quad (2.7)$$

may be used to accommodate the elastic modulus reduction (Ghali and Favre 1986, Machida and Uehhara 1987). The relaxation coefficient ϕ can be determined by the aging and creep deformation accumulation of concrete. Nonlinearity and creep analysis need be

coupled with the thermal stress analyses in this case. To avoid these difficulties, the lower and upper bounds of the effective stiffness were used, respectively, to study the influence of the variation of material properties for a long-term thermal stress analysis (Venturelli et al 1992).

2.3 Numerical Simulation of Dam Structures

Numerical simulation methods have been extensively applied to durability, safety and rehabilitation analyses of hydroelectric structures. The numerical methods have been developed to deal with many complicated processes, such as concrete cracking, structure deterioration and damage and response to thermal, ice and earthquake loading (Leger et al. 1994). The uncertainty of the environmental influences and the nonlinearity of concrete raise many difficult problems for computational mechanics. Sophisticated numerical simulation methods are generally developed for the purposes of prediction of the structure behavior, interpretation of observed phenomena, study of the damage mechanism and efficient rehabilitation strategies.

2.3.1 Dam Cracking Analyses

Cracking is a common structure failure form. Cracks appear in an early stage of massive concrete casting because of the thermal stresses caused by hydration heat, and the cracks continuously initiate and propagate under the action of service load, thermal load, ice load and earthquake load. The methods of linear elastic fracture mechanics (LEFM) were applied early to the dam crack analyses by Kaplan (1961). The Irwin's stress intensity factor and Griffith energy balance criteria were used to predict the crack

stability and propagation. In recent decades, the discrete crack analysis model of LEFM has been applied to many investigations of the static fracture response (Ayari 1988, Linsbauer et al. 1989, Ingraffea 1990, Gioia et al. 1992) and dynamic response (Mlakar 1987, Chopra 1988, Ayari and Saouma 1990, Bhattacharjee and Leger 1993). Cracks are simulated as geometrical discontinuities in the discrete crack model. The LEFM criteria were used to predict the growth of the crack. In the early time of finite element analyses, after the crack is introduced in the structure, the crack is constrained to follow a predefined path along the element edges. The mesh dependence and the requirement of continuous change in nodal connectivity and remesh cause many difficulties in computation. Ingraffea and Saouma (1985) successfully eliminated one of the major drawbacks by using interactive computer graphics and automatic remeshing. The energy conservation criteria were proposed by (Bazant and Oh 1983) to make the method less mesh dependent. Ayari (1988) simulated the discrete crack growth in gravity dams by the interactive graphics method. A LEFM criterion was used to determine the crack extension process. It was found that the traditional approach, in which the tensile resistance of material is taken as the criterion for a crack initiation, predicts a lower structural response than the fracture mechanics does. The discrete fracture method was proposed as an alternative to the traditional method for dam crack analyses

The stress-strain diagram of concrete is found to have a descending branch in the post-peak region. Several investigations have been carried out to incorporate the nonlinear fracture process zone with the concrete softening, development of small size crack group and size effect. This led to the development of several nonlinear fracture

mechanics models. Hillerborg (1976) proposed the fictitious crack model for predicting the influence of a fracture zone on the crack propagation. In his model, the crack is composed of two parts, a physical crack across which no stress can be transmitted and a fictitious crack in front of the physical crack. Within the fictitious crack, the stress has its peak equal to the tensile strength of the concrete at one tip and decreases to zero at the physical crack tip. As there is a stress to be overcome for the crack to open, energy is absorbed. Fracture energy criterion is used to predict the crack growth (Brincker and Dahl 1989, Dahlblom and Ottosen 1990, Hillerborg 1991). The smeared crack model was introduced by Rashid (1968). In this approach, the crack zone corresponds to the material degradation in the continuum finite element domain. The stiffness of the finite element is modified along the crack path. Various constitutive relations and criteria are adopted to determine the element stiffness. This model has been used in many dam crack analyses (Mays et al. 1989, Ingraffea 1990, Fauchet et al. 1991, Beshara 1993, Bhattacharjee and Leger 1993b, Leger and Bhattacharjee et al. 1994). Some current issues in computational mechanics of concrete were discussed by de Borst et.al (1994), and the performances of various crack models were compared. Many recent research works can also be found in (Mang 1994).

The initiation of microcracks is usually studied by a damage mechanics method. Damage evaluation models are constructed based on many microscopic material failure mechanisms. Progressive damage and cracking of concrete in dams was studied by Cervera et al (1990) and (1992). Combined plasticity and damage models were proposed by Yazdani and Schreyer (1990) and Abu-Lebdeh, and Voyiadjis (1993) and Voyiadjis, G.Z. & Abu-Lebdeh

(1993). The applications of damage mechanics theory to the concrete analysis were summarized by Mazars et al (1989) and Bourdarot et al (1994).

2.3.2 Numerical Modeling of Concrete Joints

When construction joints or long cracks are encountered, special numerical processes are needed to simulate the crack opening and closing displacements. The opening and closing displacement analysis of the crack surface is a type of contact problem between two solid bodies. Extensive research has been carried out for solving a variety of simple to complex elastic boundary value problems in the contact analyses. The contact surface may be initially open or closed, and with or without friction. Therefore, the primary difficulty of this sort of problem comes from satisfying geometric compatible condition of the contacting surface. The main approaches fall into two categories: Lagrange Multiplier methods (Hughes et al 1976) and Penalty methods (Francavilla and Zienkiewicz 1975). The advantage of second method is that the interface displacement condition can be easily imposed and the algorithms can be made independent of interface material constitutions. Contact conditions can be imposed directly from kinematic considerations by imposing geometric compatibility of the contacting surface during the incremental loading process (Sacheva and Ramakrishnan 1981, Bathe and Chaudary 1985, and Ayari and Saouma 1991). Usually pairs of contactor nodes are laid out along the potential contact surface. The compatibility of displacement is imposed by means of the relative normal and tangential nodal displacements.

Although effort was made to include the crack closure situation into the smeared crack model, interface element method prevails for the modeling of various geometrical

discontinuities in different kind of structure problems. Many interface elements or 'gap' elements have been developed based on the penalty functions where the element stiffness is actually the penalty numbers (Stadter and Weiss 1979, Beer 1985, Chandrasekaran 1987 and Gens 1988). Interface elements can be divided into two elementary classes. The first class contains the continuous line or plane interface elements (Alonso 1985, Gens 1988, Hohberg 1989), whereas the second class contains the nodal or point interface elements (Arya and Hegemier 1982, Mehlhorn et al. 1985).

For the interface element, usually it is supposed to have a pseudo-thickness or very small thickness. The stiffness of the element actually depends on the penalty parameter. The main difficulty associated with the penalty method is the proper choice of the penalty parameter to prevent the ill conditioning of the global stiffness matrix. To avoid this, Wilson (1975) suggested the relative degree of freedom for the contact element. The penalty parameters are associated with the contact boundary compliance. The numerical integration of the interface element is another important issue. It is found that the choice of the integration strategy has a significant influence on the element's performance. The eigenvalue method was used to analyze the element behavior with different integration strategies by Gens (1988) and Schellekens and de Borst (1993).

Interface element method was applied to model the soil-reinforcement interaction by Gens (1988), the intermediate layer between rock and concrete in arch dam by Alonso(1985) and Hohberg (1989), rock joint by Goodman (1968) and concrete masonry mortar joints by Arya and Hegemier (1982). The structural analysis of jointed dams can

be found in Hohberg 1988, Singhal and Nuss 1990, Ahmadi and Razavi 1992 and Lei et al. 1992. More works are listed in Leger et al. (1994).

2.4 Ice Load on Structures

The action of ice on dam structures in the northern region is one of the most important environmental factors affecting their design and safety assessment. Extensive research has been done in the past on the physical properties of ice, ice formation processes and the interaction of the ice sheet and the dam structure. Two international forums: the Ice Symposiums of the International Association for Hydraulic Research (IAHR) and the Conferences of Port and Ocean Engineering under Arctic Condition (POAC) have served to highlight major developments in ice studies. Literature surveys have been done by Michel (1981), Bengtsson (1981) and Gerard (1983). Large number of research articles are also listed in (Cammaert and Muggeridge 1988) and (Léger and Tinawi 1994). Among other research, the ice formation and ice load on dam structures have the most concern for the present study.

2.4.1 Ice Formation

The prediction of the thickness of ice layer over the forebay of a dam is the purpose of the ice formation study. Formation of an ice sheet results because of a net heat loss from the surface of water to the ambient air. Cooling of water depends on heat transfer by conduction, convection, evaporation and radiation, and also on hydraulic factors such as flow, and on meteorological conditions. The ice cover formation in rivers and lakes was discussed by Michel (1972, 1978) and Ashton (1986). In calm water of lakes, ice crystals will nucleate right at the water surface in the form of ice needles. The

individual needles will grow in the supercooled top water layer and form a continuous ice plate at the surface. Snowfall in calm water close to freezing point also originates the initial ice plate. The surface layer of the ice cover will grow underneath. The initial growth of the ice sheet is of columnar ice, and then snow ice. The first snowmelt may infiltrate to the solid ice cover and refreeze to form more snow ice. Variety of circumstance factors can influence the ice thickness growth. It makes the theoretical prediction of the thickness of ice sheet a very difficult task. However, it is found that the growth is more or less depends on the freezing-degree-days (FDD) (Gerard 1983). The “freezing-degree-days” method is discussed in detail by Bengtsson (1981) and Ashton (1986). A field measurement of freezing-degree-days is reported by Schmidlin (1995). This method is used to predict the ice growth in rivers as well (Desmond et al 1981). For engineering problems, a reasonable estimation of ice thickness can be obtained by this method.

2.4.2 Ice Properties

The mechanical properties of ice are among the most interesting topics of ice studies. Ice loads on structures are intimately related to the physical properties of ice. Extensive laboratory experiments have been done to get the elastic modulus and strength of various types of ice. Comprehensive reviews of the mechanical properties of ice have been made by Michel (1978), Ashton (1986), and Cammaert and Muggeridge (1988). Experimental results show a considerable scatter in the values of the elastic modulus, tensile and compressive strength. Ice forming mechanism strongly influences the ice properties. Beside, the mechanical properties generally depend on the ice temperature and

loading speed and direction. Data concerning the physical characteristics of ice such as its crushing strength, its modulus of elasticity, and the effects of plastic flow are inadequate and approximate. Comparisons of ice strength data can be found in (Sinha, 1981) and (Ashton 1986). The adhesion strength of ice to a variety of structural materials was studied by Saeki (1981). This strength is quite important when shear forces on structures are taken into account.

2.4.3 Ice load

Ice loading or ice pressure on structures might be the main concern of the research work on ice. Generally ice sheet exerts horizontal and vertical force on structures. The horizontal force is usually considered as one of the major loads for dams in a cold region. The horizontal force is mainly produced by thermal expansion in the ice sheet and by wind drag. Furthermore, the thrust exerted by expanding ice depends on the thickness of the sheet, the rate of temperature rise in the ice, fluctuations in the water surface, character of the reservoir shores. The rate of temperature rise in the ice is a function of rate of the air temperature rise and the amount of snow cover on the ice. Lateral restraint of the ice sheet depends on the character of the reservoir shores. In view of all of these variables, we are faced with a difficult task in estimating the amount of ice pressure acting against a structure. However, several aids are available to help in the estimation. Rose (1947) collected and put into a usable form all the known data on ice pressures. Theoretically, the thrust of the order 7.5 to 30 t/m is possible under North American climatic conditions. A figure of 15 t/m was commonly used in Canada. The designer may

elect to include or disregard the effects of lateral restraint and solar energy (Bureau of Reclamation, 1977).

The United States Bureau of Reclamation has conducted field and laboratory tests on ice temperatures and pressures. These tests have been summarized by Monfore (1954). In the field tests at Eleven Mile Canyon Reservoir, it was found that most of the expansion in the ice sheet took place in the upper portions and, therefore, the thrust was largely concentrated there. In the computations for thrust exerted by an 18-inch-thick sheet, it was found that no change in temperature occurred at the bottom of the ice; hence, zero pressure was assumed there. It was also demonstrated that a 5-inch snow covering had a remarkable insulating effect in preventing temperature rise and thus minimizing the ice thrust. Similar observation is also reported by Xu (1981) for dams in China. A criteria for dam by the Japanese National Committee on Large Dams (1970) provides a guide to pressures likely to occur under conditions of rising temperature neglecting the effects of solar radiation (Thomas, 1976).

Field measurements of ice pressure were also reported by Cox (1984), Walter (1992), Lewis (1993), Comfort (1994) and Ko et al. (1994). On Canaan Street Lake, Canaan NH, the field measurement in March 1983 shows that the normal stress perpendicular to and parallel to shore can be -250 to 160 and -320 to 160 KPa, respectively (Cox 1984). The peak ice pressure recorded by Comfort et al (1994) in Paugan Dam was 69 KPa. A pressure of 248 KPa was recorded at Tarryall reservoir (Ko et al. 1994). These measurements indicate that the ice pressure can be quite different from place to place and can be as high as 320 KPa. Models and empirical formula were

developed by these authors to relate the ice pressure to the air and ice temperature, snow accumulation, and environment events. Because of the strong environmental dependence, the models are quite different. However, the ice temperature is considered as the direct influence in all the models. The ice rheology model (Michel 1981, Cox 1984) can likely be taken as a base for the development of suitable ice pressure calculation model.

The vertical load of ice layer is mainly caused by water fluctuation. The mechanism is easy to understand, but the load is difficult to be measured in the field. Fewer measurements were reported than for the horizontal pressure. A shear stress in the range of -100 to 100 KPa in Canaan Street Lake was reported by Cox (1984). The uplifting force of the ice layer was theoretically studied by Billfalk (1981) and Christensen (1987). The ice layer is modeled by a thin plate on an elastic foundation. The ultimate force was calculated as the force immediately prior to the formation of a crack close to the wall. The force after the ice sheet interlocked at the crack was also estimated by the thin plate model. The ice sheet can be partially flooded after the appearance of the root crack. The influence of the flooding on the ice load was studied by Kerr (1985). It was found that the flooding may substantially reduce the ice vertical load on structure.

Chapter 3

Transient Heat Transfer Analysis of Concrete Dams

3.1 Introduction

Cyclic stresses and strains due to seasonal thermal variation play a significant role in crack initiations and the strength degradation of massive concrete structures, such as dams and other structures in electrical generating stations. In Canada, many structures are subjected to surface temperature variations exceeding 70 to 80 °C from winter to summer condition. Such severe conditions usually induce important thermal loads that jeopardize the intended durability of the structure.

Accurate transient thermal analysis of dam structures is an important requirement for design and rehabilitation purpose. The finite element method (FEM) is the widely accepted numerical method for carrying out this task. The methodology in assessing a seasonal temperature variation in gravity dams by finite element analysis has been described by Leger et al. (1993) and Armstrong (1993). The basic information required to perform a thermal analysis includes 1) weather data describing the air temperature, solar radiation, wind speed and snow cover; 2) water temperature and ice cover; 3) foundation temperature; and finally 4) thermal properties of materials and hydration characteristics of cementing materials. Suitable computational models have been established to treat these factors (Leger et al. 1993, Armstrong 1993, Venturelli and Leger et al. 1992). In all the methods, two major computational issues ought to be treated properly. One is the

initial condition of the dams, and the second is related to the numerical integration scheme when a long time behavior is required.

To predict the thermal stresses at any point in time during the service life of the dam, the thermal history of the structure can not be neglected. Strictly speaking, prediction of the stresses must begin with an estimation of the thermal history of the structure beginning at the time of construction and continuing to the time of interest. Factors that must be accounted for include the temperature distribution in the rock foundation material prior to concrete placement, dimensions and concrete types comprising individual lifts of the structure, the construction schedule, concrete thermal properties, heat of hydration characteristics of the concrete, placing temperatures and ambient conditions at the project site. A finite element modeling algorithm was developed by (Armstrong 1993) for incrementally constructed dams. Unfortunately, the data required for the thermal history is not available for most of dams. The common method for prescribing the initial temperature field is to assume a certain type of steady temperature solution as the initial condition (Venturelli and Leger 1992). Boundary temperature variations of a certain period are repeated in time. When a repetitive temperature time history of a 2 to 3 year simulation is observed, the transient heat transfer analysis is considered to be convergent. Only the results after convergence is achieved seem reliable for practical use. A finite period of time is required beyond which errors introduced by the assumed initial field temperature drop to a negligible level.

The heat exchange between air and concrete is the main reason for the temperature variation inside these structures. Besides the temperature difference between air and concrete, solar radiation, wind speed and snow cover are also responsible for this exchange. The treatment of this kind of boundary condition can be very difficult.

Generally, a convective boundary condition is used for the concrete exposed to air in practical analyses. The determination of a precise convective coefficient for this boundary condition is a complicated process and the sensitivity of a transient process to this coefficient has to be studied as well.

Direct integration of the heat transfer equations is well accepted as an efficient method to follow the transient variations of field temperature inside a dam. The period of interest, in which the field temperature is required for a dam, is usually several years after the creation of the structures. The choice of a time step length for numerical integration of the semi-discrete finite element equations is important because of numerical accuracy and computational efficiency. The half day and day average schemes are often used (Venturelli and Leger 1992, Leger et al. 1993 and Leger et al. 1995) in practice. The question whether week long or hour-long time steps should be used for efficient integration must be answered based on some deliberate consideration.

A dam, like any other structure, has its own eigen-characteristics that command its heat response. In this paper the composition of the transient temperature solution is first analyzed and the influence of initial field temperature, which decreases exponentially in time, is subsequently evaluated. The time period for this influence to decay to a negligible level can be determined based on the heat-transfer frequencies of the structure. The sensitivity of the structure to convective coefficients is also studied by means of the structure frequencies. Besides, the first order heat-transfer frequency is also used in the choice of the time step length for accurate and stable numerical integration.

3.2 Transient Heat Transfer Processes

For an arbitrary two-dimensional body as shown in Figure 3.1, the governing equation for transient heat transfer is:

$$\rho c \frac{\partial T}{\partial t} - k \left(\frac{\partial^2 T}{\partial x^2} + \frac{\partial^2 T}{\partial y^2} \right) = q_r \quad (3.1)$$

where T is the temperature, t is time, ρ is specific density; c is specific heat capacity; k is heat conductivity and q_r is a heat source term.

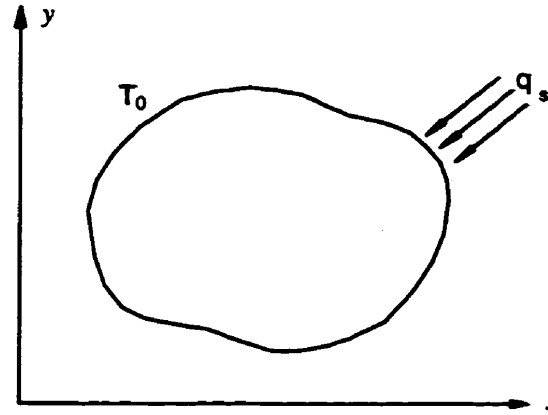


Figure 3.1 Body Subject to Heat Transfer

On the surface of the body the following conditions must be satisfied:

$$T(x, y, t) = T_0(x, y, t) \quad \text{on } S_1 \quad (3.2)$$

$$k \frac{\partial T}{\partial n} = q_s(x, y, t) \quad \text{on } S_2 \quad (3.3)$$

where q_s is the heat flux, S_1 is the temperature imposed boundary part and S_2 is the convective boundary part. There are two kinds of contribution to the heat flux at a surface: *heat flow* and *convection*. A general expression that describes both effects is:

$$q_s = f + h(T_\infty - T) \quad (3.4)$$

where f is the heat flow, T_{∞} is the ambient air temperature and h is the heat convective constant, which may depend on the environment in some cases. The initial condition given by

$$T(x, y, 0) = T_{ini}(x, y) \quad (3.5)$$

is also required for the solution of the initial-boundary-value problem given by equations (3.1-3.3). The finite element formulae can be derived using the variational principle.

For the temperature T within an element, An approximation can be written in the form

$$T = \mathbf{N}\mathbf{T} \quad (3.6)$$

where $\mathbf{N}$ is the array of shape functions and $\mathbf{T}$ is the vector of nodal temperatures. The finite element equations for heat transfer can be written as:

$$\mathbf{M}\dot{\mathbf{T}} + (\mathbf{K}_1 + \mathbf{K}_2)\mathbf{T} = \mathbf{f}_1 + \mathbf{f}_2 + \mathbf{f}_3 \quad (3.7)$$

where $\mathbf{M}$ is the heat capacity matrix; $\mathbf{K}_1$ is the heat conductivity matrix; $\mathbf{K}_2$ is the convective matrix; and $\mathbf{f}_1$, $\mathbf{f}_2$ and $\mathbf{f}_3$ are the heat load arrays. At the element level

$$\mathbf{M} = \int_e \rho c \mathbf{N}^T \mathbf{N} dv; \quad \mathbf{K}_1 = \int_e k \mathbf{B}^T \mathbf{B} dv; \quad \mathbf{K}_2 = \int_{se} h \mathbf{N}^T \mathbf{N} ds \quad (3.8a)$$

$$\mathbf{f}_1 = \int_{se} f \mathbf{N}^T ds; \quad \mathbf{f}_2 = \int_{se} h T_{\infty} \mathbf{N}^T ds; \quad \mathbf{f}_3 = \int_e q_r \mathbf{N}^T dv \quad (3.8b)$$

where S_e represents the intersection of an element's boundary and the convective boundary, and $\mathbf{B}^T$ is defined as:

$$\mathbf{B}^T = \left[\frac{\partial \mathbf{N}^T}{\partial x} \quad \frac{\partial \mathbf{N}^T}{\partial y} \right].$$

It is worth noting that a very large convective coefficient will make the boundary temperature equal to the ambient air temperature on a convective boundary. Therefore,

the given temperature boundary condition on S_1 can be introduced by an equivalent heat flow

$$q_e = \mu(T_0 - T); \quad (3.9)$$

where μ is an equivalent convective coefficient. By doing so, the boundary conditions (3.2) and (3.3) can be treated in a similar way. Once the boundary condition and initial condition of the dam are given, the transient temperature field can be solved by integrating equation (3.7).

There exist mainly two methods for solving FEM equation (3.7): the mode superposition method and the direct integration method. The method of choice is usually based on whether the long term or short-term solution is required and the scale of the problem. Here, we first use the mode superposition method to study the solution structure of transient heat transfer processes. Some typical characters of the transient heat transfer processes can be found by means of the composition of the finite element solution. Subsequently the traditional direct integration is adopted for long term temperature history analysis of the dam.

3.3 Solution Structure of Transient Heat Transfer Processes

Equation (3.7) corresponds to an eigenvalue problem:

$$[\lambda \mathbf{M} - \mathbf{K}] \mathbf{x} = 0 \quad (3.10)$$

where $\mathbf{K} = \mathbf{K}_1 + \mathbf{K}_2$. Let Φ be the mode matrix in which the columns are the eigenvectors of equation (3.10). The nodal temperature vector can be transformed to the mode coordinates via

$$\mathbf{T} = \Phi \mathbf{y} \quad (3.11)$$

where $\mathbf{y}$ is the mode coordinate array of temperature. Substituting transformation (3.11) into equation (3.10) and multiplying the two sides by Φ^T , we diagonalize matrix $\mathbf{M}$ and $\mathbf{K}$, because of the properties of the mode matrix. A set of decoupled equations in terms of $\mathbf{y}$ are obtained:

$$\dot{y}_i + \lambda_i y_i = p_i \quad i = 1, 2, \dots, n \quad (3.12)$$

where λ_i is the i th eigenvalue or the so called heat transfer frequency; n is the order of $\mathbf{M}$ and $\mathbf{K}$; y_i and p_i are the i th components of $\mathbf{y}$ and $\mathbf{p}$, respectively. In the above, $\mathbf{p}$ is defined as

$$\mathbf{p} = \Phi^T (\mathbf{f}_1 + \mathbf{f}_2 + \mathbf{f}_3). \quad (3.13)$$

The solution of equation (11) can be easily obtained as:

$$y_i = c_i e^{-\lambda_i t} + \int_0^t e^{-\lambda_i (t-\tau)} p_i(\tau) d\tau \quad (3.14)$$

or, in matrix form, as

$$\mathbf{y} = \left[e^{-\lambda_i t} \right] \mathbf{c} + \int_0^t \left[e^{-\lambda_i (t-\tau)} \right] \mathbf{p}(\tau) d\tau. \quad (3.15)$$

Using transformation (3.11) again, we obtained

$$\mathbf{T} = \Phi \left[e^{-\lambda_i t} \right] \mathbf{c} + \int_0^t \Phi \left[e^{-\lambda_i (t-\tau)} \right] \mathbf{p}(\tau) d\tau. \quad (3.16)$$

When $t = 0$, by the initial condition, we have

$$\mathbf{T}_{ini} = \Phi \mathbf{c}. \quad (3.17)$$

Using the relation

$$\Phi^T \mathbf{M} \Phi = \mathbf{I}, \quad (3.18)$$

the vector $\mathbf{c}$ can be determined to be

$$\mathbf{c} = \Phi^T \mathbf{M} \mathbf{T}_{ini} \quad (3.19)$$

Thus, the solution of equation (3.7) is completely determined.

3.4 Role of the Heat-transfer Frequencies

The mode matrix Φ depends only on the geometry and material properties of the structure and does not depend on the time and initial conditions. While initial conditions appear only in the first term of solution (3.16), contributions of heat loads accumulate through the integration of the second term. Therefore, the effects of initial conditions and heat loads are separated in this formal solution. Also, because $\mathbf{K}$ and $\mathbf{M}$ are usually positive definite, all the eigenvalues are positive and real. They can be ordered as:

$$0 < \lambda_1 < \lambda_2 \dots < \lambda_n \quad (3.20)$$

This means that the influence of the initial conditions is an exponentially decay function of time. The eigenvalues actually control the rate of decay of this influence. More precisely, the lowest decreasing speed is controlled by the fundamental eigenvalue λ_1 .

If we expect the first term of the solution (3.16), that is an influential initial condition, to decrease to a small percentage of its original value, say $\alpha\%$, the time for this decay to take place can be estimated as:

$$t_d = \frac{1}{\lambda_1} \ln \left(\frac{100}{\alpha} \right) \quad (3.21)$$

In most cases, initial conditions are hard to prescribe for structures. We have to start our analysis at a certain time prior to the time of interest, based on some approximate initial condition. equation (3.21) helps us predict how long the contribution of an assumed initial temperature will decrease to a negligible level. This is particularly important for many transient analyses. It makes certain that the solutions with different initial condition prescriptions will converge to the same solution, which is only dominated by the second term of expression (3.16). This property indicates that any type of temperature distribution, even a uniform distribution, can be chosen as the initial

condition. In a practical analysis, the time t_d can be used to find how long we need to start our analysis ahead of the time of interest.

The requirement for the time step to keep the numerical solution stable when the forward Euler integration scheme is used was well established (See Zienkiewicz 1977) and is given by

$$\lambda_1 \Delta t \leq 1. \quad (3.22)$$

Because λ_1 is the smallest heat-transfer frequency, the upper bound for the time step length should be chosen such that

$$\Delta t \leq \frac{1}{\lambda_1}. \quad (3.23)$$

The time step set by (3.23) is usually a month long. In order to capture the response of dam structures to daily variation of the ambient air temperature, half day or day long time steps have to be adopted for temperature history analyses. From the viewpoint of numerical stability, the use of these time steps is quite safe. Also, numerical investigations show that this time step length provides accurate solutions.

To use solution (3.16) for a transient heat transfer process, usually only the first n eigenvalues and mode vectors need be included. The number of modes introduced in (3.16) depends on the variation of the boundary conditions. With fast varying boundary conditions, higher order modes have to be included in solution (3.16) to capture the response of the structure, especially in the region near the boundary. For instance, if a day average scheme is used for the ambient air temperature input, the highest frequency in solution (3.16) should match a one-day time span.

So far two criteria (3.21) and (3.22) can be set. Both of them depend on the first order heat-transfer frequency of the structure. For transient heat transfer analyses of

dams, the direct integration method is preferred. However, we can make use of the information obtained from the fundamental heat-transfer frequency. Because only the fundamental frequency is needed, simple eigenvalue problem solving methods such as the vector iteration method can be used.

3.5 Direct Integration of FEM Equations

In order to integrate equation (3.7) with respect to time, we first divide the time interval into equal steps. The time steps are written as:

$$t_0 = 0 \text{ and } t_i = i \Delta t, i=1, \dots, n \quad (3.24)$$

where Δt is the time step length and $t_n = n\Delta t$ is the final time of interest. The time derivative of the temperature array, $\dot{\mathbf{T}}$, at time t_{i+1} is approximated by the backward difference scheme, i.e.

$$\dot{\mathbf{T}}_{i+1} = \frac{1}{\Delta t} (\mathbf{T}_{i+1} - \mathbf{T}_i) \quad (3.25)$$

Substituting equation (3.25) into equation (3.7), for time t_{i+1} , we obtain the following difference equation:

$$(\mathbf{M} + \Delta t \mathbf{K}) \mathbf{T}_{i+1} = \mathbf{M} \mathbf{T}_i + \Delta t \mathbf{Q}_{i+1}, \quad (3.26)$$

where

$$\mathbf{Q}_{i+1} = (\mathbf{f}_1 + \mathbf{f}_2 + \mathbf{f}_3)_{i+1} \quad (3.27)$$

For all the time steps the matrix $(\mathbf{M} + \Delta t \mathbf{K})$ is the same, so that the marching scheme:

$$\mathbf{T}_{i+1} = (\mathbf{M} + \Delta t \mathbf{K})^{-1} (\mathbf{M} \mathbf{T}_i + \Delta t \mathbf{Q}_{i+1}) \quad (3.28)$$

can be used to determine $\mathbf{T}_{i+1}$ in terms of $\mathbf{T}_i$. $\mathbf{T}_0$ is actually the initial temperature distribution array $\mathbf{T}_{ini}$. The transient behavior can be determined step by step.

The Long Spruce Generating Station was built by Manitoba Hydro between 1974 and 1978 on the Nelson River in north Manitoba, Canada. The site is in a zone of sub-arctic climate where the daily maximum temperatures in summer can be above 40 °C and the daily minimum temperatures in winter can reach -40 °C. The generating station has a rated head of 80 ft and consists of a combination of earth structures, concrete gravity non-overflow dams, a 10 unit powerhouse and a spillway. One non-overflow dam, the south transition structure, acts as a termination point for an earth dam that connects the powerhouse to the south bank of the river. The south transition consists of three unreinforced blocks bounded on each side by vertical contraction joints. Each block is 15.24 m long and approximately 28 m tall. The structural model of the south transition is sketched as in Figure 3.2.



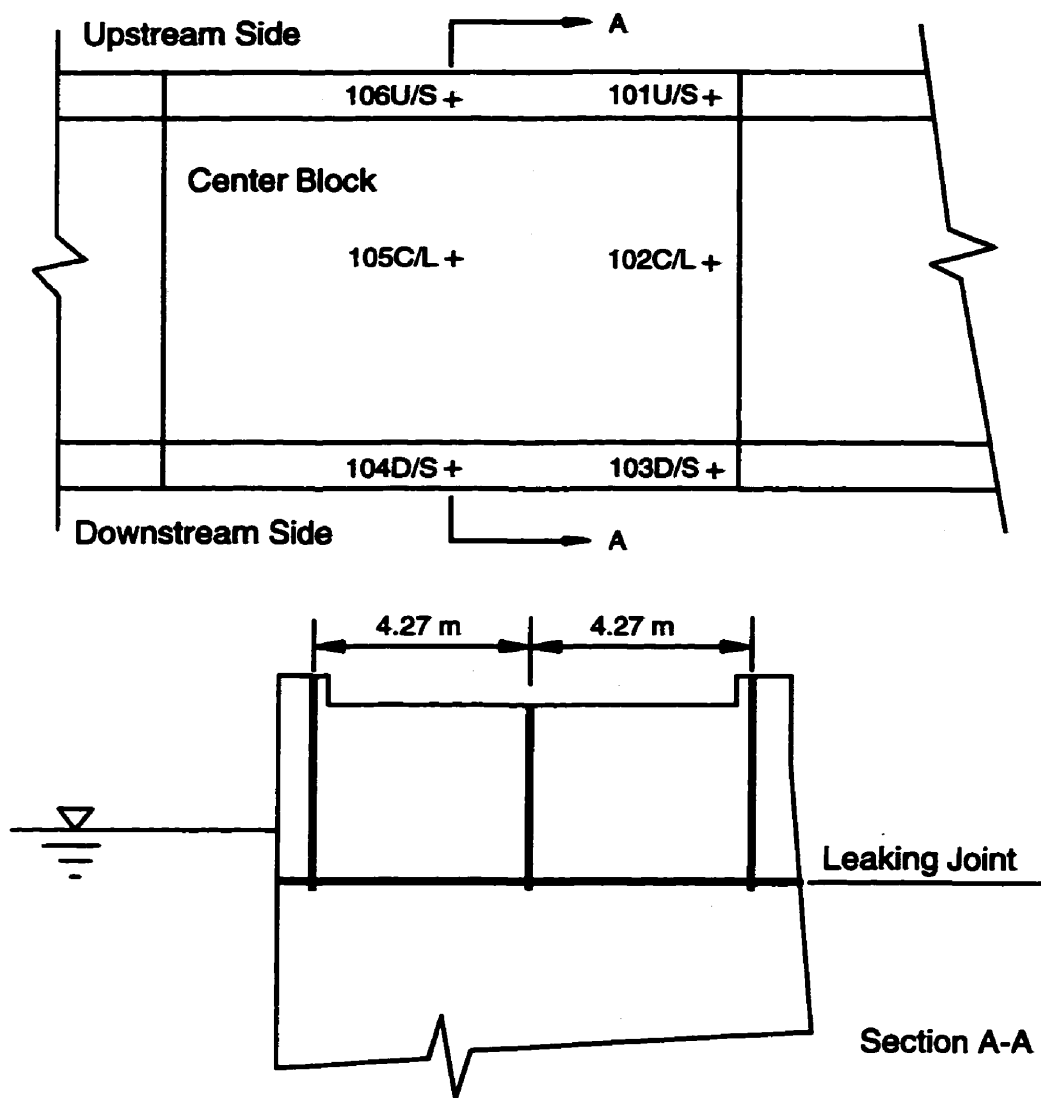


Figure 3.3 *Locations for Monitoring Instrumentation*

After a few years in service, leakage from the top-most horizontal construction joint began appearing on the downstream face of the south transition. The leakage is a result of partial debonding of the construction joint and differential movements between the concrete above the joint and the remainder of the structure. This movement is thought to be caused by the constantly varying and non-uniform temperature distribution in the dam. In late 1992, Manitoba Hydro installed instrumentation in the middle block of the south transition to monitor the in-situ displacements and temperatures at six locations along the leaking construction joint, as shown in Figure 3.3. Hourly data has been collected from these devices for years. Temperature readings for row 101U/S-102C/L-103D/S and 104D/S-105C/L-106U/S are in very good agreement correspondingly. This indicates that the temperature varies two-dimensionally inside the dam.

A thermal analysis of the south transition structure was performed, based on the structural data and ambient air temperature records supplied by *Manitoba Hydro, Canada*.

3.6.1 Finite Element Model

The thermal history of the structure is simplified by ignoring residual effects from the construction period. The effect of this assumption will be of no impact if the period of interest is far from the construction time, and stresses that incorporate the residual effect from the construction period are of no interest.

In reality, water or air can go into the leaking joint and the leaking joint should be taken as a discontinuity in the dam structure model. A water or air layer exists inside the leaking joint. However, in considering the large heat capacity of the dam, a two-dimensional continuous structure model is adopted for the temperature analysis.

The cross section of the dam is discretized using 424 eight-node isoparametric elements corresponding to 1424 degree of freedom. The mesh is shown in Figure 3.4. The mesh near the water level and the leaking construction joint are refined in order to capture the extreme temperature variation in this region. According to some preliminary analyses, the temperature variation near the base of the structure is relatively small, and therefore a coarse mesh layout is used along the vertical direction. In the horizontal direction, 10 elements are used so that some nodal points coincide with that of built-in thermocouples installed as part of the instrumentation along the leaking construction joint.

The material constants of concrete obtain from Manitoba Hydro and used in this analysis are listed in Table 3.1:

Material Properties of Concrete	Table 3.1
Density	$\rho = 2294 \text{ kg / m}^3$
Specific heat	$c = 0.225 \text{ kcal / kg}^\circ\text{C}$
Thermal conductivity	$k = 62.4 \text{ kcal / m} \cdot \text{day}^\circ\text{C}$

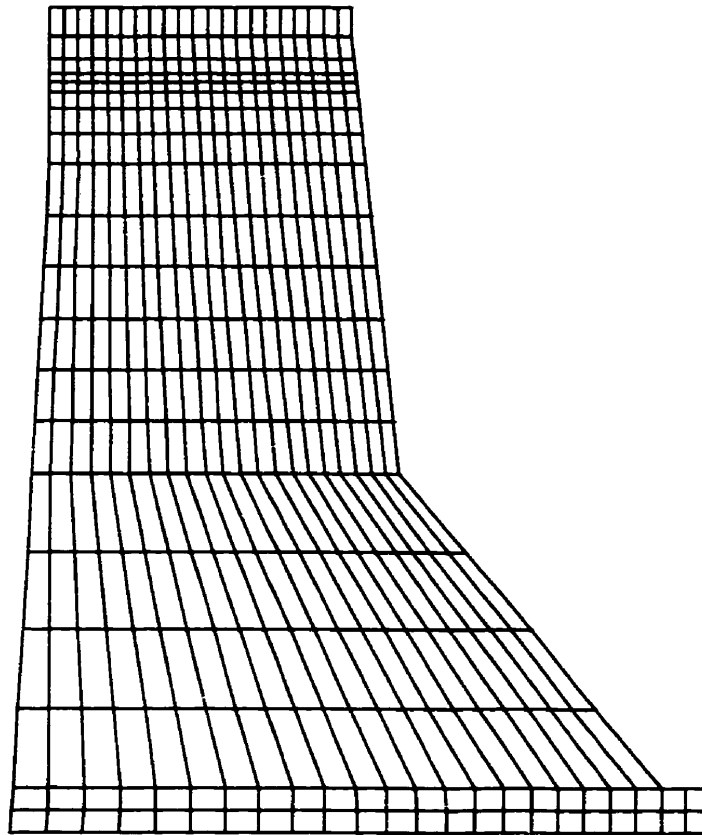


Figure 3.4 *Finite Element Mesh of the Long Spruce Dam*

3.6.2 Boundary Conditions

The boundary of the dam is divided into three parts: the *air-dam* interface, the *water-dam* interface and *dam-foundation* interface.

Dam-Foundation Boundary: Heat exchange takes place between the dam and the foundation. Strictly speaking, a part of the foundation should be included in the finite element analysis. However in order to simplify the computation, we suppose that there is no heat exchange between the dam and the foundation. A simple adiabatic boundary condition is adopted:

$$k \frac{\partial T}{\partial n} = 0, \quad (3.29)$$

where T is the temperature, $\frac{\partial}{\partial n}$ the normal gradient and k the heat conductivity coefficient. This simplification ignores the heat exchange along the dam-foundation boundary and therefore causes some errors in this region. Because this boundary line is far away from the area of interest, this type of boundary prescription incurs negligible error.

Air-Dam Boundary: The heat exchange phenomenon occurring between the dam and air is complicated. Many factors need be concerned with, such as the wind speed, solar radiation, snow coverage and light reflection. More discussion about the treatment of this type of boundary can be found in (Leger et al. 1993). To include all of the factors is extremely difficult. However, no matter how complicated these phenomena are, they only influence the amount of heat exchange. In our analysis, we use the classical convective boundary condition:

$$k \frac{\partial T}{\partial n} = h_a (T_\infty - T), \quad (3.29)$$

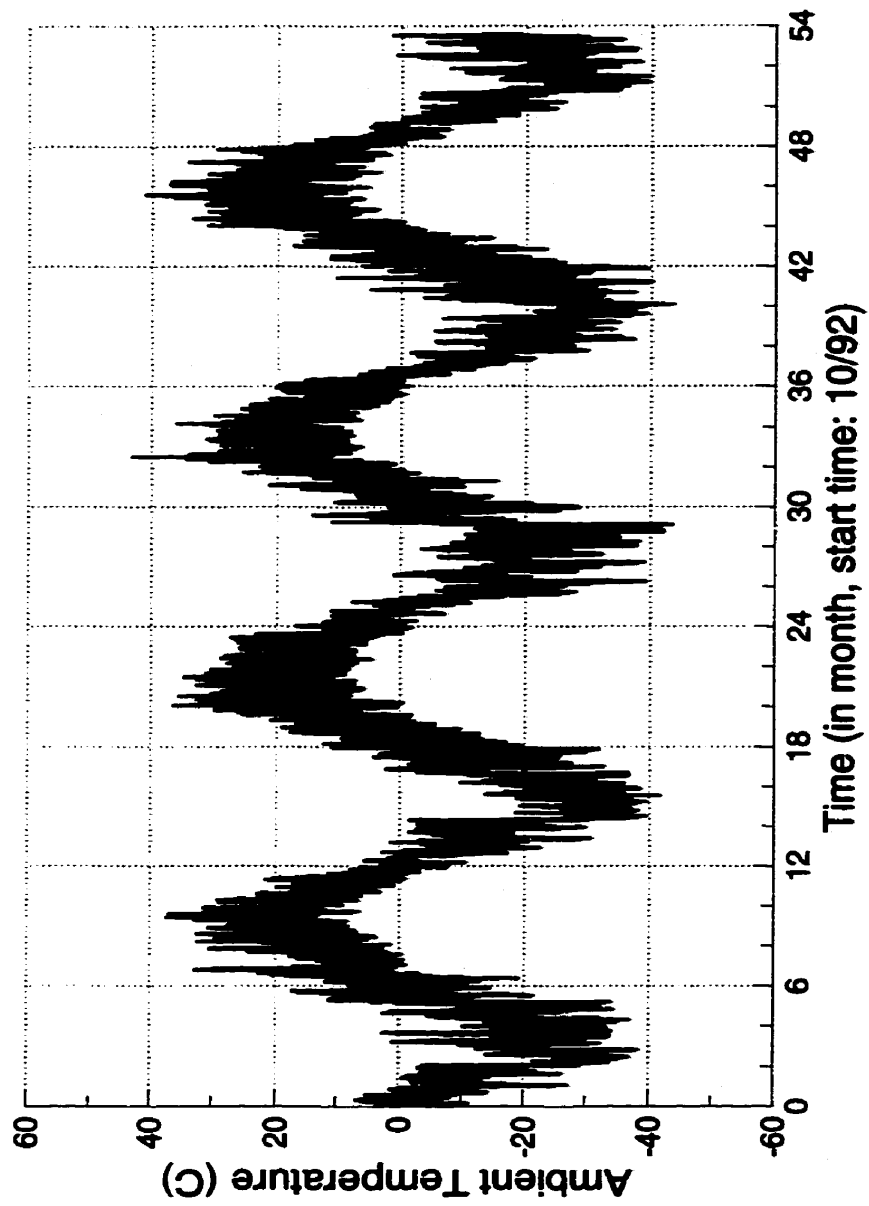


Figure 3.5 Ambient Temperature of the Site

where h_a the heat convective coefficient and T_a is the ambient air temperature. It is intended to include all the dam surface environmental influences in the choice of the convective coefficient. The influence of the variation of convective coefficient on the heat transfer behavior of the dam will be examined later in this chapter.

Hourly ambient air temperature variations have been recorded at the Long Spruce site since October 1992, and are depicted in Figure 3.5.

Water-Dam Boundary: The situation at the water-dam boundary is similar to that of the air-dam boundary. Heat exchanges convectively between the dam and water. However the rate of heat exchange is much higher than in the one occurring in the case of the air-dam boundary. For this boundary, it was suggested by Leger et al. (1993) to assume the concrete's temperature to be the same as the water temperature. In our studies, we use an equivalent heat flow defined in equation (3.9) to prescribe this boundary condition. If we choose a large value for the coefficient μ , the temperature of concrete on this boundary is made equal to the water temperature. Therefore, the water-dam boundary is treated as a convective one in this way. The convective coefficient h_w between the dam and water is much larger than h_a . The variation in the water temperature during 1994 is shown in Figure 3.6. The data of 1994 is the only available yearly water temperature record. It is taken as a template to generate the water temperature data of other years because that the yearly water temperature does not change much in the same region.

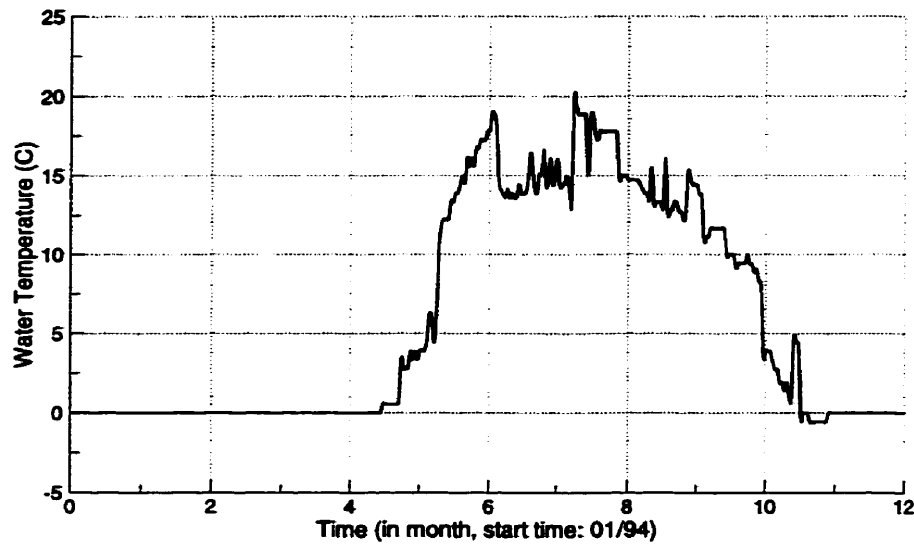


Figure 3.6 Water Temperature

3.6.3 Calibration of the Convective Coefficients

The dependence of the fundamental heat-transfer frequency on the heat convective coefficients is examined first in the numerical analysis. The variation of the fundamental heat-transfer frequency, according to different air-dam convective coefficients h_a and water-dam convective coefficients h_w , is shown in Figure 3.7. For all the h_w values, the fundamental frequency increases quickly when the value of the air-dam convective coefficient h_a is smaller than 2 ($kcal/m^2 \cdot hour \cdot ^\circ C$). Once h_a is larger than this value, the increase of the heat transfer frequency becomes slow. Finally a steady value can be approached. In another aspect, the curves corresponding to different water-dam convective coefficients show that the difference between the curves becomes very small when h_w is larger than 8 ($kcal/m^2 \cdot hour \cdot ^\circ C$), hence when h_w is larger than 8 ($kcal/m^2 \cdot hour \cdot ^\circ C$) its change causes a very small change in the fundamental heat-

transfer frequency. These trends indicate that the heat transfer behavior of the dam may be insensitive to the variation in the heat convective coefficient in certain cases. Accurate convective coefficient values are sometimes not necessary for those cases. This feature makes thermal history analysis less sensitive to these convective coefficients and has been verified by our thermal history analyses. The fundamental heat-transfer frequency of the structure can be approximately taken as $0.15/\text{month}$ to fit a large range of h_a and h_w values.

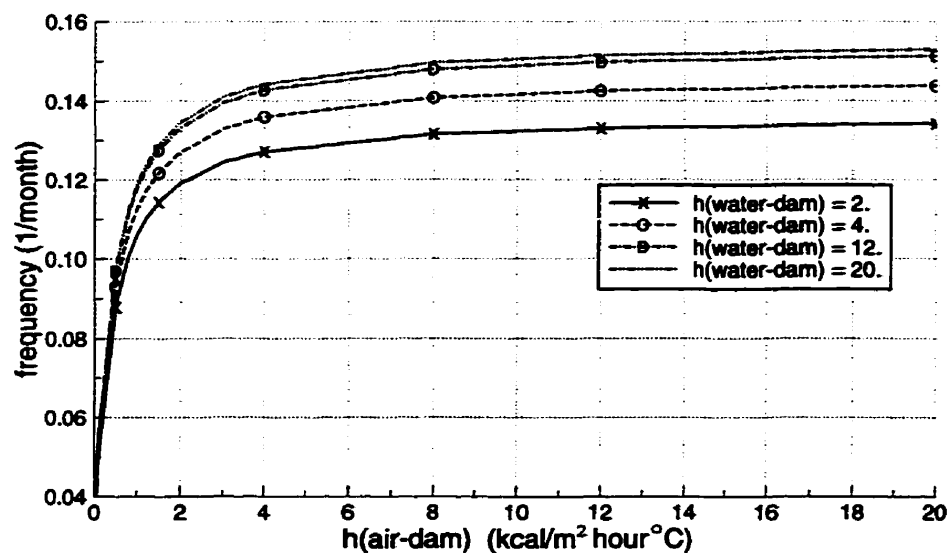


Figure 3.7 Variation of the Fundamental Heat Transfer Frequency

3.6.4 Numerical Results at Instrument Locations

The method of backward direct time integration method has been used to integrate the transient heat transfer process of the Long Spruce Dam. The starting time adopted in the analysis is October 1992, and the time integration lasted until March 1997. In order to observe the influence of the assumed initial condition on the convergence of

transient solution, different uniform initial temperature conditions of +40, +20, -20 and -40 °C are taken as the initial conditions. The temperatures at the three locations where the thermistors are installed (See Figure 3.3) are carefully examined.

The level of the error at any time t caused by an assumed initial temperature condition can be estimated according to criterion (3.21) as $\alpha = 100 e^{-0.15t}$. The upper bound time for the error to decrease to 5% is approximately 20 months. It should be mentioned that this is an overall upper-bound estimation. In a practical situation, convergence might be reached well before such a period.

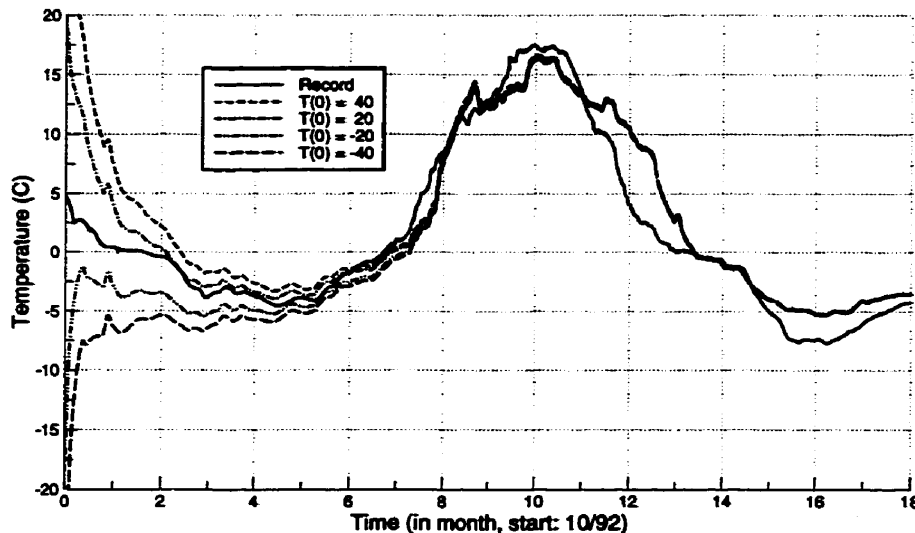


Figure 3.8 *Decay of Initial Conditions at Location 101U/S*

Temperature in the upstream side: Thermistor 101U/S is installed 0.457m from the upstream face of the dam along the leaking joint line. This location is near the water level and in an area where the temperature gradient is high. In Figure 3.8 the computed temperatures with different initial conditions are depicted against the recorded data by the

thermistor. It can be seen that the finite element results approach the record data rapidly within the first month. After 9 or 10 months all the curves, no matter what their initial values are, converge to a single curve which coincides with the field measurement curve very well. The convergence is reached much earlier than the predicted upper bound time. The fast convergence can be explained by the fact that the concrete near the boundary responds to a surface temperature variation faster than that inside the dam body.

The values of the air-dam convective coefficient were taken from 2 to 8 ($\text{kcal} / \text{m}^2 \cdot \text{hour} \cdot ^\circ\text{C}$) and the values of the water-dam convective coefficient from 6 to 40 ($\text{kcal} / \text{m}^2 \cdot \text{hour} \cdot ^\circ\text{C}$). The water-dam coefficient is expected to have more influence on the temperature on the upstream side. However, the variation of the convective coefficients causes small changes in the computed temperature values. Relatively, a value of 20 ($\text{kcal} / \text{m}^2 \cdot \text{hour} \cdot ^\circ\text{C}$) for the water-dam convective coefficient results in better overall agreement with the record curve, and the change of air-dam coefficient causes much less change in the temperature results. Very good agreement between the field measurements and the computed temperature was obtained using $h_w = 20 \text{ kcal} / \text{m}^2 \cdot \text{hour} \cdot ^\circ\text{C}$ and $h_a = 7 \text{ kcal} / \text{m}^2 \cdot \text{hour} \cdot ^\circ\text{C}$ (See figure 3.9.) It is observed that a very large water-dam convective coefficient makes the temperature of water-dam boundary equal to the water temperature.

The temperature at location 101U/S is expected to get more influence of water temperature. Because only the water temperature data of 1994 is available, the data of other years is obtained by repeating the data of 1994. In Figure 3.9, a higher error level can be found during other years than 1994. In winters, an ice sheet forms on water surface, which is close to location 101U/S. For simplicity, an ice boundary is not

considered in the analysis. It is believed that this kind of simplification is responsible for the computation error of the yearly lowest temperature at location 101U/S.

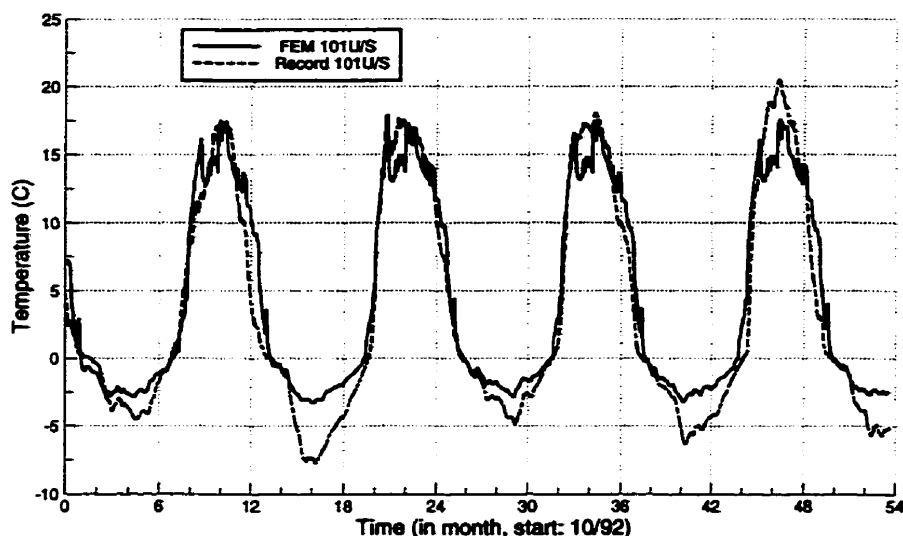


Figure 3.9 *Temperature Time History at Location 101U/S*

Temperature in downstream side: The downstream side is exposed to air. The temperature variations near the face of the structure are extreme. The response becomes smooth for points up to one meter inside the dam. The location of thermistor 103D/S is 0.54m from the downstream face along the construction joint within the zone of extreme temperature variations. The numerical results are compared with the recorded data in Figure 3.10. Different combinations of water-dam and air-dam convective coefficients have been used. The change of water-dam convective coefficient has almost no influence on the temperature results in this area. A difference in temperature within one or two degrees can be caused by different air-dam convective coefficients. A value of $7 \text{ kcal/m}^2 \cdot \text{hour} \cdot ^\circ\text{C}$ for the air-dam convective coefficient also corresponds to a better

result. The data fitting for thermistors 101U/S and 103D/S help us to choose the convective coefficients h_a as 7 and h_w as $20 \text{ kcal/m}^2 \cdot \text{hour} \cdot ^\circ\text{C}$ for the subsequent analysis.

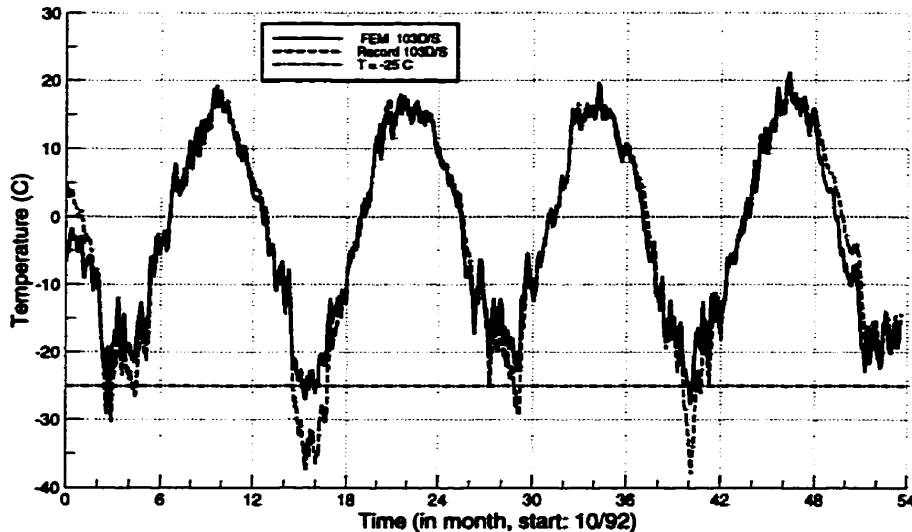


Figure 3.10 *Temperature Time History at Location 103D/S*

A large discrepancy between the numerical and recorded temperatures is found during the time when the recorded temperature is below -25°C . The hardware used in the reading of thermistor 103D/S is reported to be unreliable when its temperature drops below -25°C . Above this extreme temperature good agreements can be observed between the numerical and recorded results presented in Figure 3.10.

For this side, numerical results show that it only takes about 8 or 9 months for all the temperature curves to converge to the recorded data curve (See figure 3.11.) Because heat exchange takes place on the dam surface, the temperature response in the upstream

and downstream sides to the ambient air temperature variation is relatively quicker. The convergence time for these two cases is much shorter than *20 months* as we predicted.

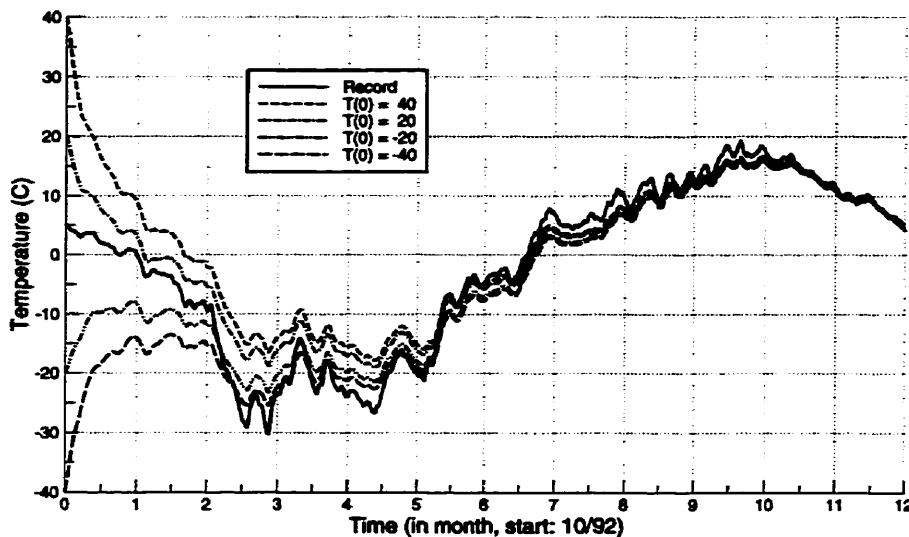


Figure 3.11 *Decay of Initial Conditions at Location 103D/S*

An electrically heated tunnel was installed outside the downstream surface in 1996. This tunnel covers the leaking construction joint to keep melt ice and drain water out of the joint. Running of this tunnel in the winter of 1996 is reflected in the Figure 3.10. It can be seen that from October 1996 (the 48th month), the recorded curve goes a few degrees higher than the computed data.

Temperature in the center: Thermistor 102C/L was installed in the center (4.72m from the upstream face) of the dam along the construction joint. The numerical results coincide well with the recorded temperature at its location (See Figure 3.12.) The temperature variation is similar to a harmonic function of time. At this location, the water-dam convective coefficient almost has no influence on the temperatures. The

change of the air-dam convective coefficient only causes slight changes in the values and phases of the temperature curves as shown in Figure 3.12.

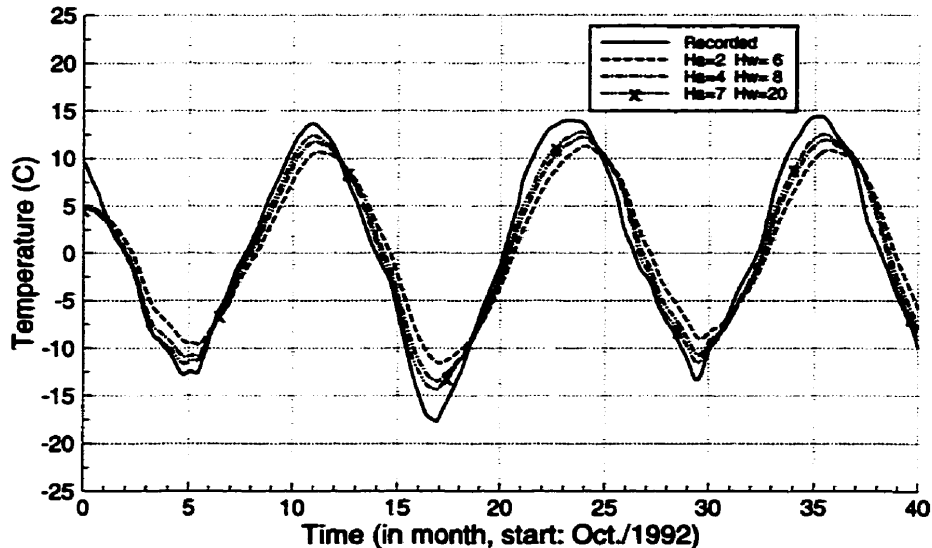


Figure 3.12 *Temperature History (center, with different H coefficients)*

Although the numerical values taken at the location of thermocouple 102C/L are in broad agreement with the recorded data, a small difference in phase between the numerical and recorded data can be found. Temperatures at other points around the location of thermistor 102C/L were compared and it was found that at the points 1 or 2 nodes (within 1 meter) downstream of the location of thermistor 102C/L, the temperature curves have a better agreement with the recorded data of 102C/L. These values are closer not only in value, but also in phase. The location of thermistor 102C/L was measured again in September 1995. It was noted that the actual location of this thermistor was further downstream than believed, by a distance of approximately 40 cm. This suggests that the theoretical results are capable of accurately predicting even the correct location of the thermocouples themselves.

Convergence of the solutions to the recorded data for the dam center is slower than for the upstream and downstream sides. Rapid convergence takes place within the first 2 or 3 months, and after 14 or 16 months all the curves converge to a single curve, as shown in Figure 3.13. This time is close to the predicted time based on the heat frequency analysis.

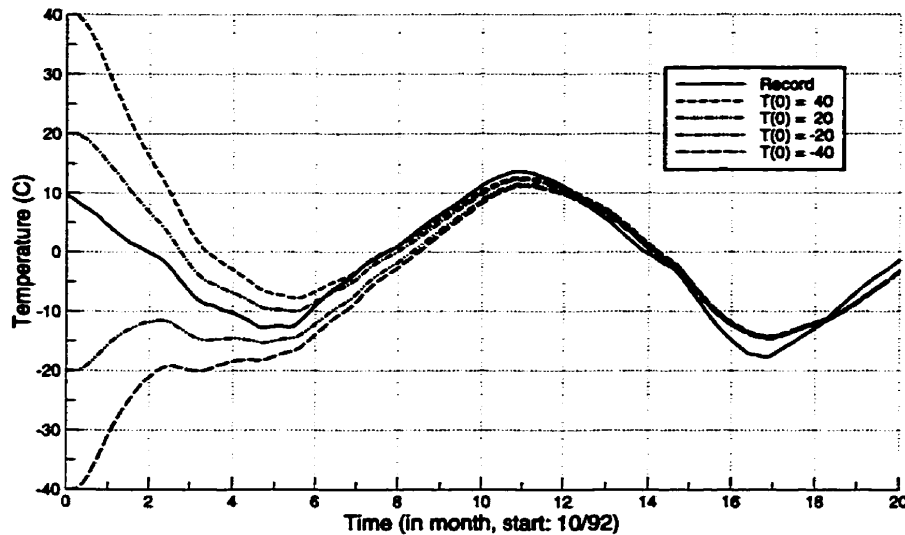


Figure 3.13 *Temperature History (center, with different initial condition)*

By criterion (3.22), the upper bound for the integration step is nearly a year. The one-day-average integration scheme is usable in analyses. However, a suitable time step must be chosen based on the variation in the ambient air temperature. The second term of solution (3.15) might be helpful for the choice of a proper time step.

3.6.5 Temperature Distributions at Different Time

The temperature distributions of the dam were output at several time points with the convective coefficients of 7 and 20 $kcal/m^2 \cdot hour \cdot ^\circ C$ for air-dam and water-dam

boundaries, respectively. Some of the iso-thermal lines are depicted in Figures 3.14.1 through 3.14.12.

The temperature contour for November 1992 is basically the assumed initial temperature field. Except for this month, the contours of the isothermal lines for the same months of different years look quite similar. In the summer season June and August, the temperature gradient within the dam is relatively small. An obvious time delay was found for the temperature inside the dam. Although the ambient air temperature can be high as 20°C , the temperature in the low center part keeps below zero. In October, the temperature gradient is not very large and the frozen zone gets smaller. A large temperature gradient comes in December. Because heat flows gradually into the center, the core of the dam keeps quite warm while the ambient air temperature is much below zero. From December to April there is a sharp temperature gradient zone along the top and downstream face of the dam. In April, a large part of the dam becomes frozen except for a zone deep low near water.

The region near the water level is a high temperature gradient zone. The leaking construction joint starts from this zone. A high temperature gradient as well as the ice loads are the critical factors to cause the relatively weak joint crack.

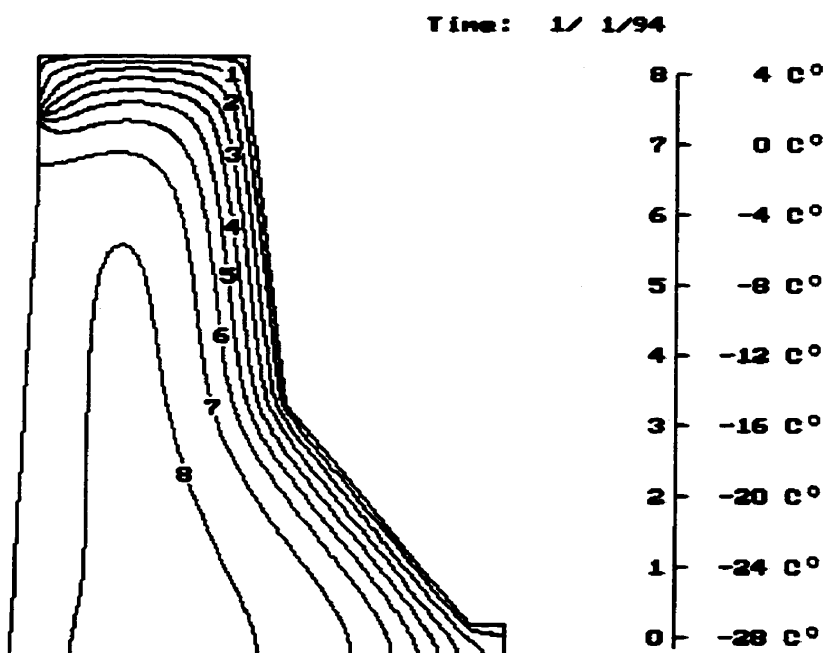
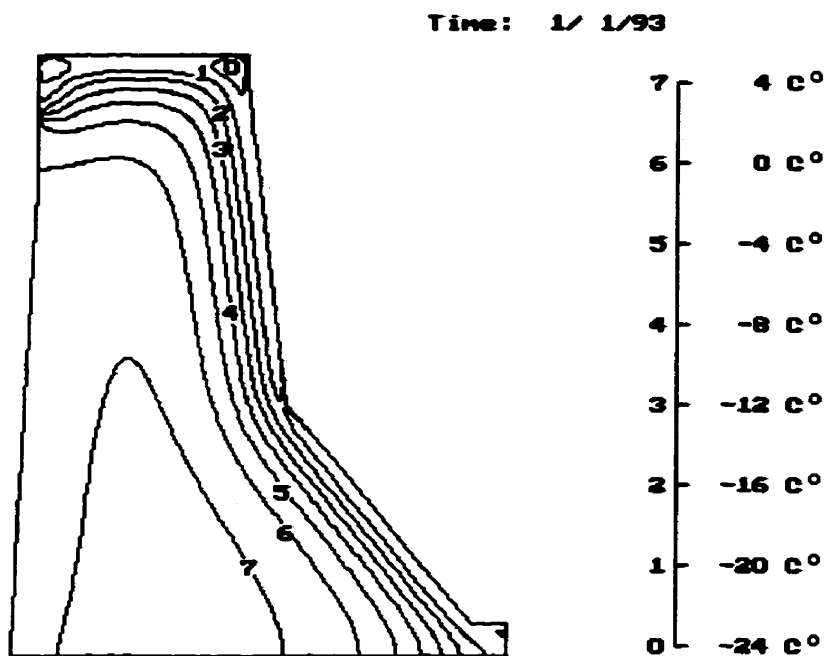
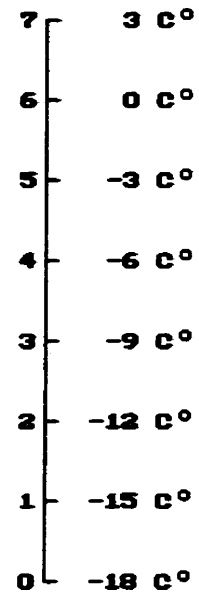
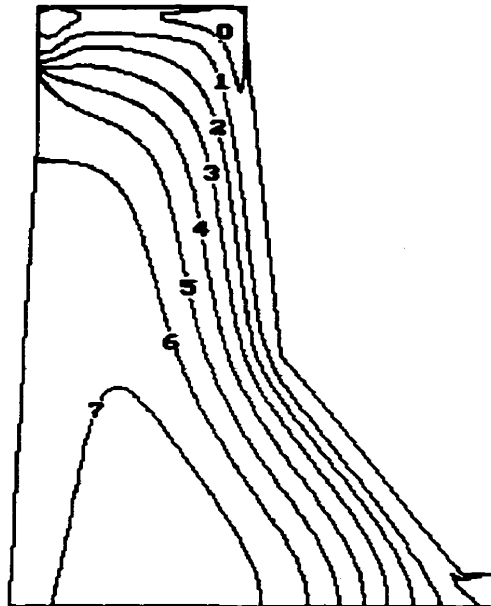


Figure 3.14.1 *Temperature Contour--January*

Time: 1/ 2/93



Time: 1/ 2/94

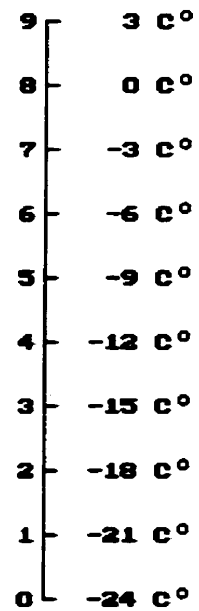
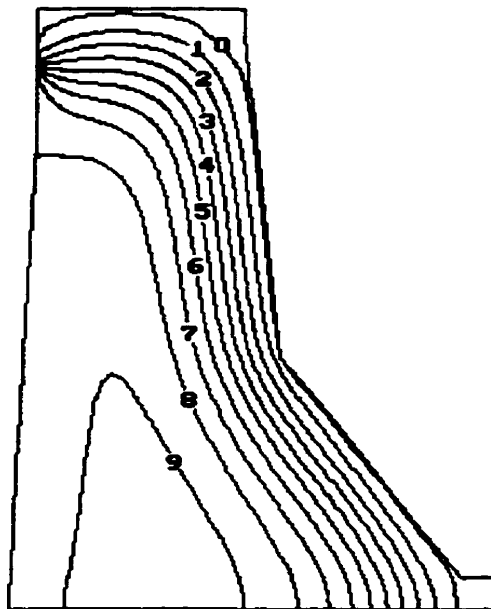


Figure 3.14.2 *Temperature Contour--February*

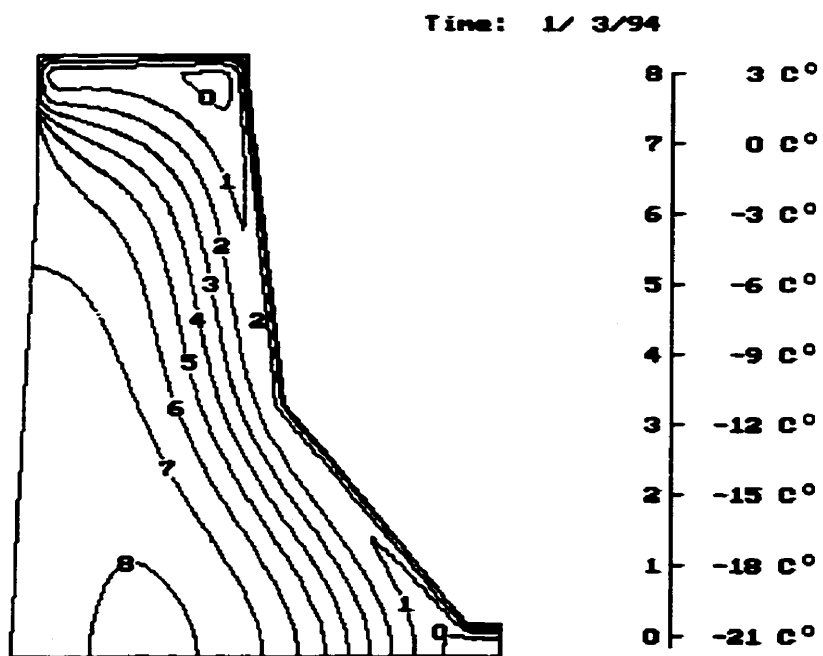
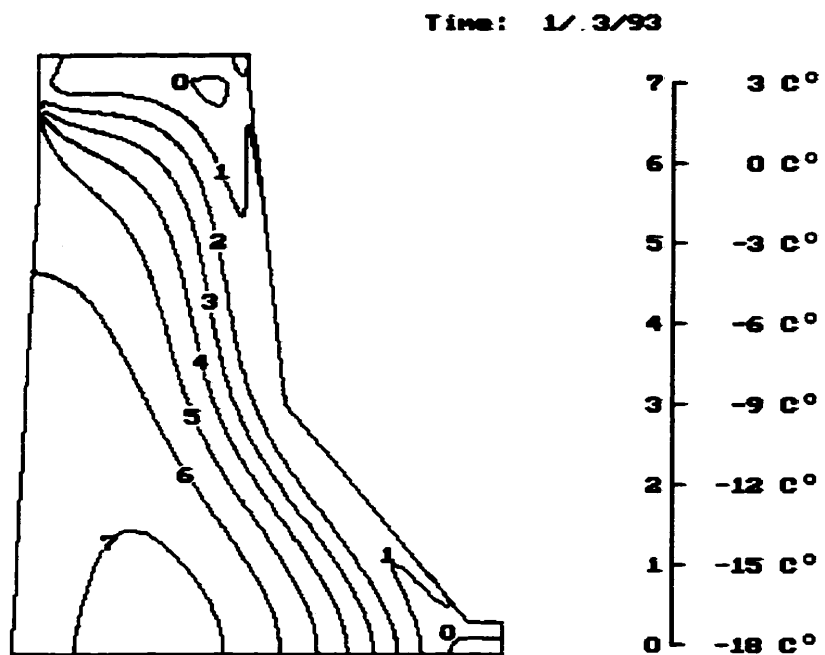
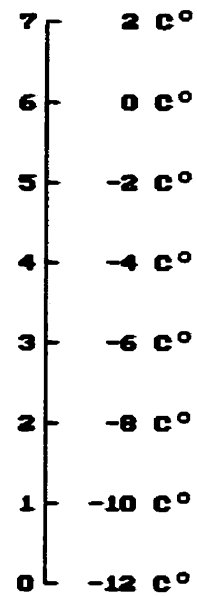
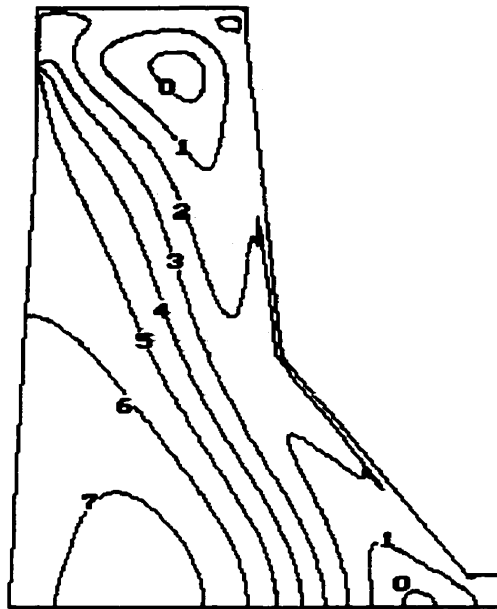


Figure 3.14.3 *Temperature Contour--March*

Time: 1/ 4/93



Time: 1/ 4/94

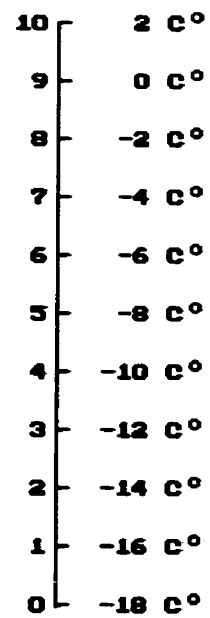
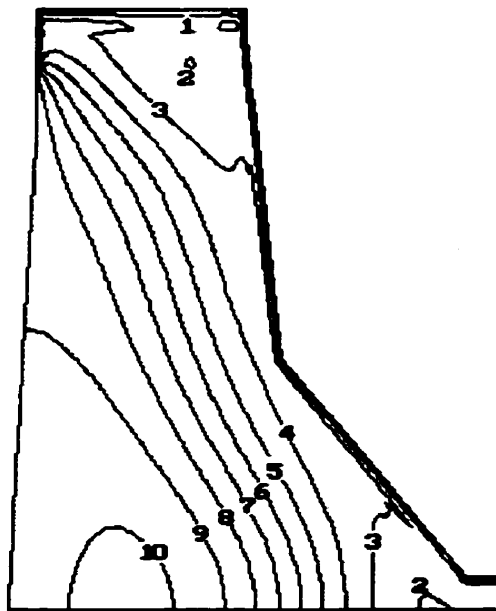
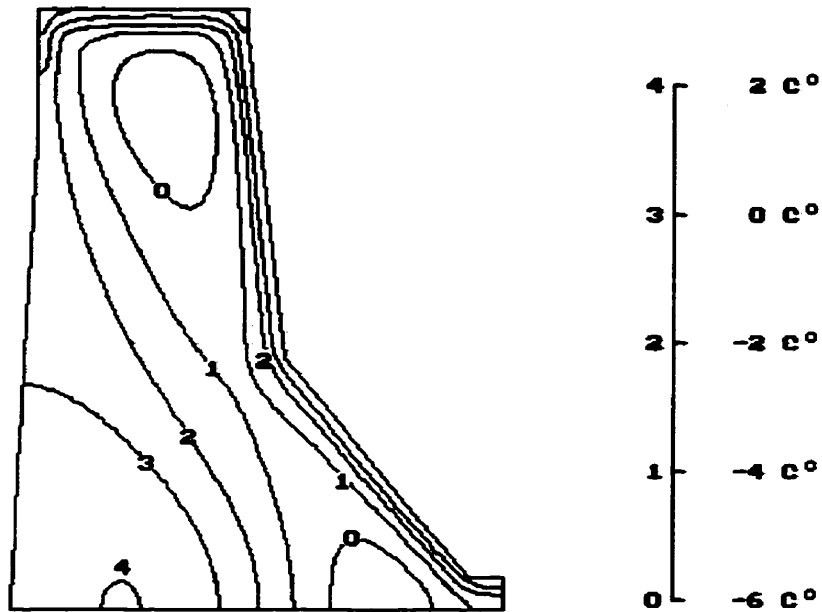


Figure 15.4 *Temperature Contour--April*

Time: 1/ 5/93



Time: 1/ 5/94

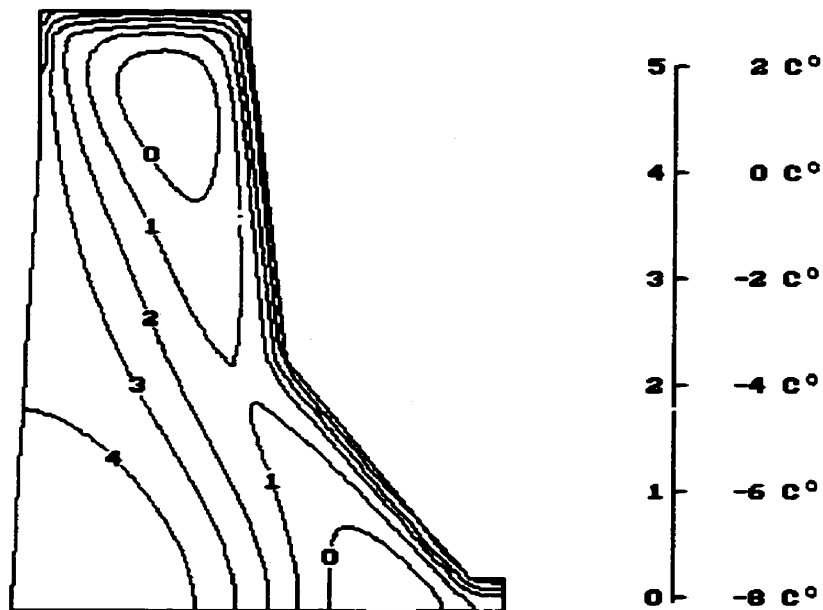


Figure 3.14.5 *Temperature Contour--May*

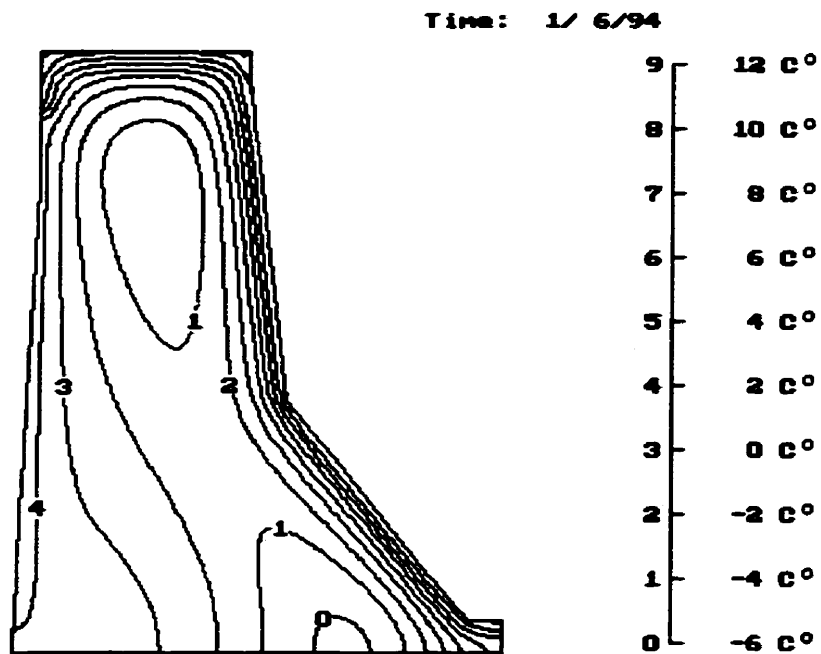
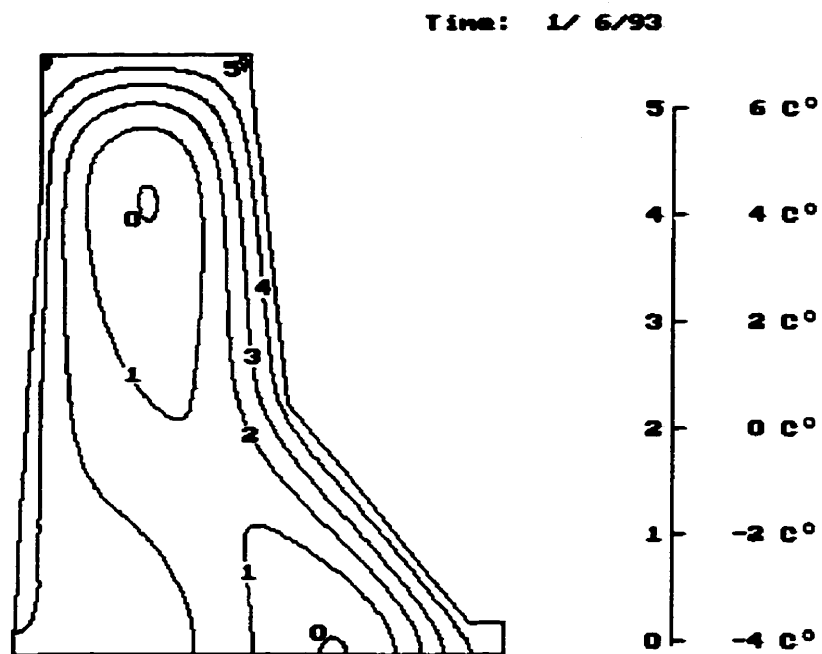


Figure 3.14.6 *Temperature Contour--June*

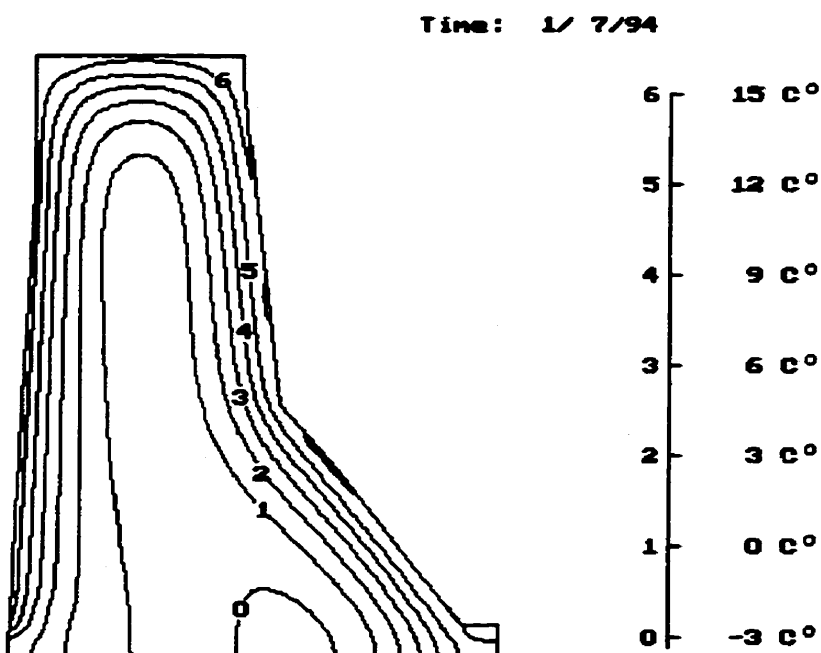
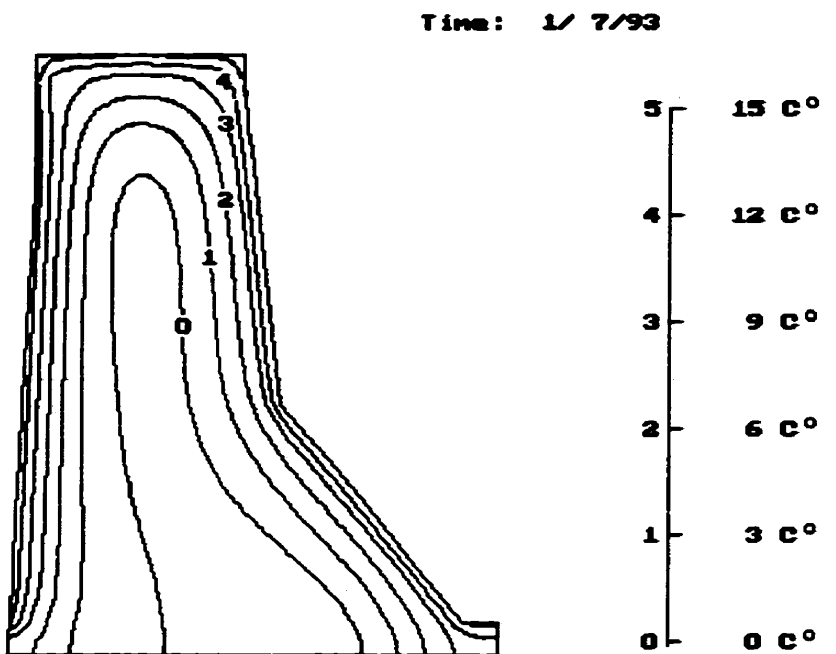
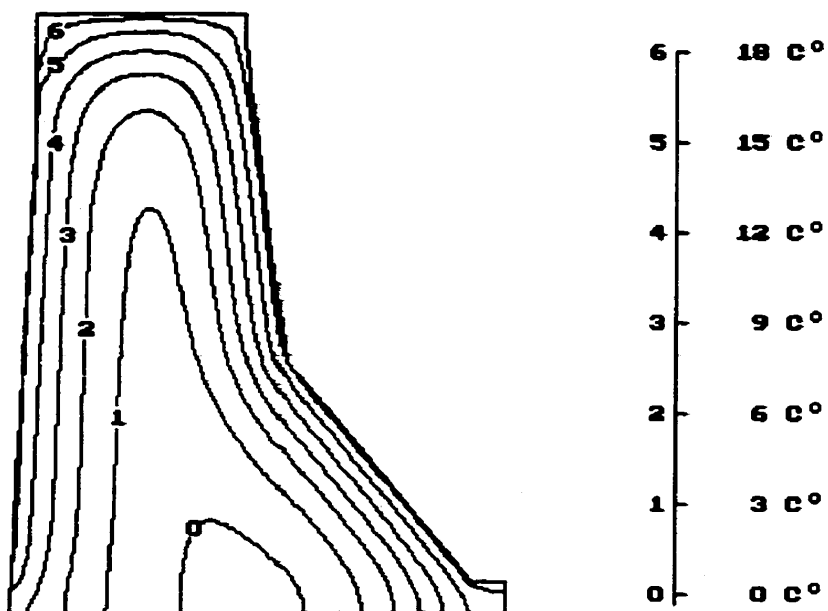


Figure 3.14.7 *Temperature Contour--July*

Time: 1/ 8/93



Time: 1/ 8/94

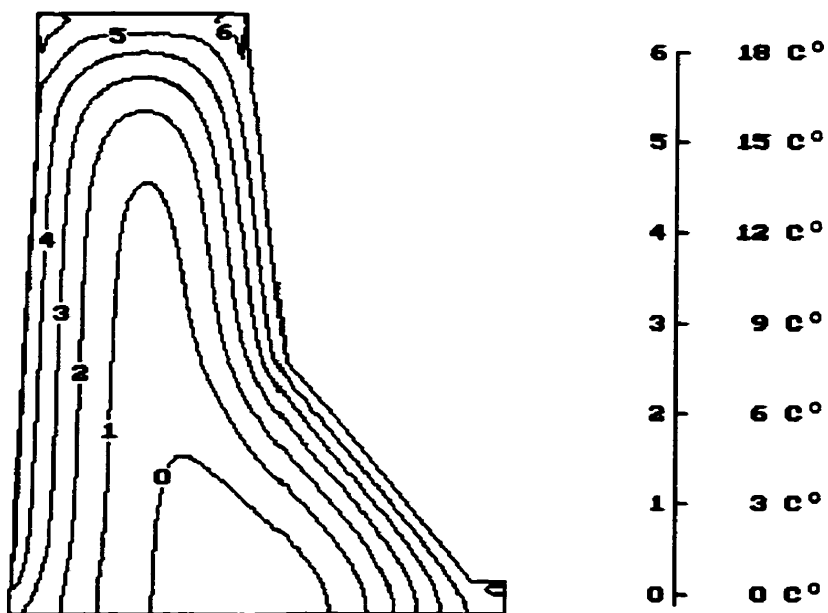
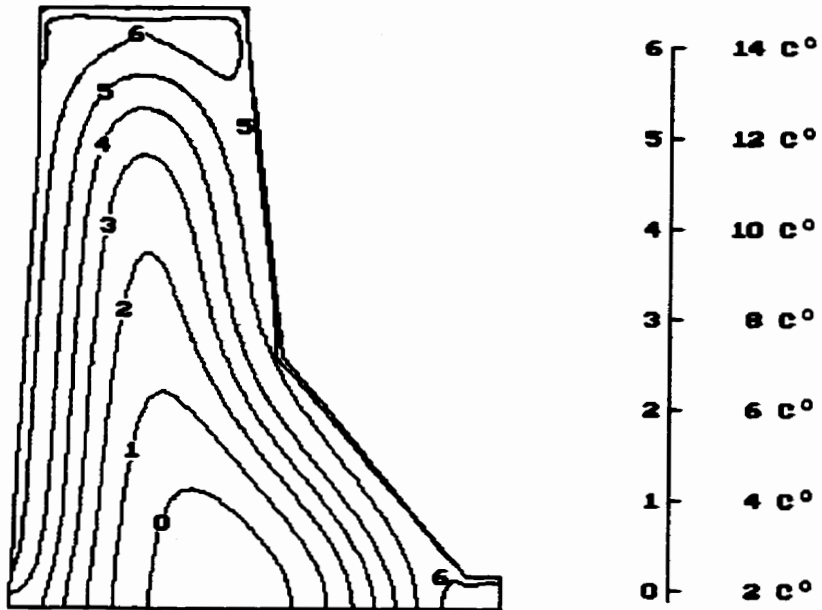


Figure 3.14.8 Temperature Contour--August

Time: 1/ 9/93



Time: 1/ 9/94

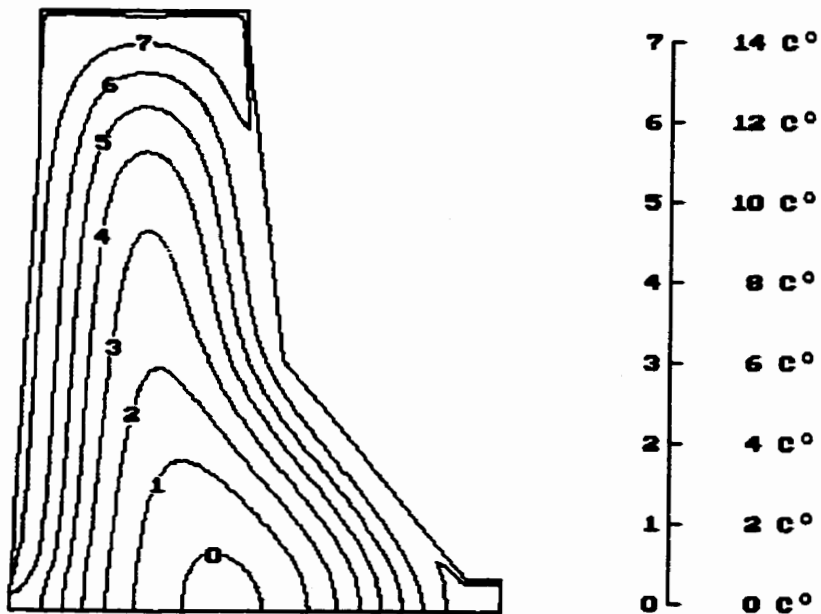
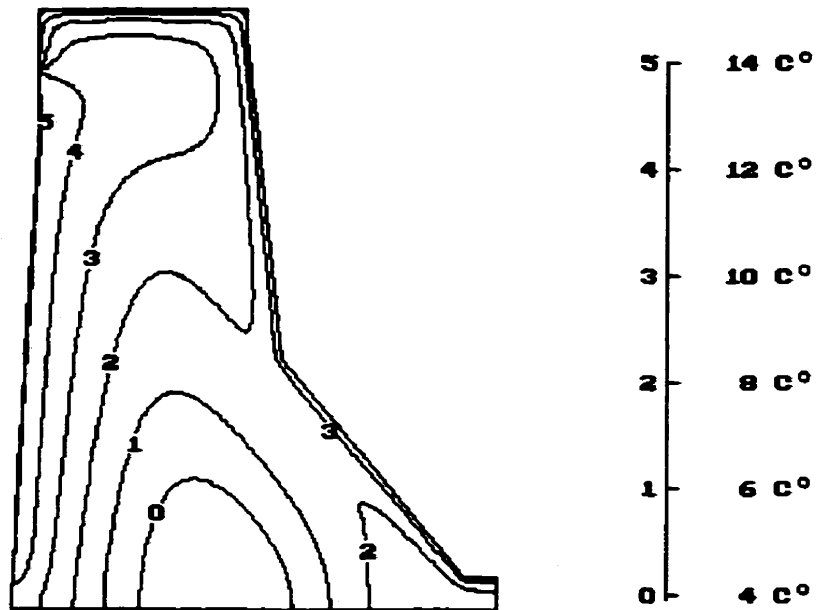


Figure 3.14.9 Temperature Contour--September

Time: 1/10/93



Time: 1/10/94

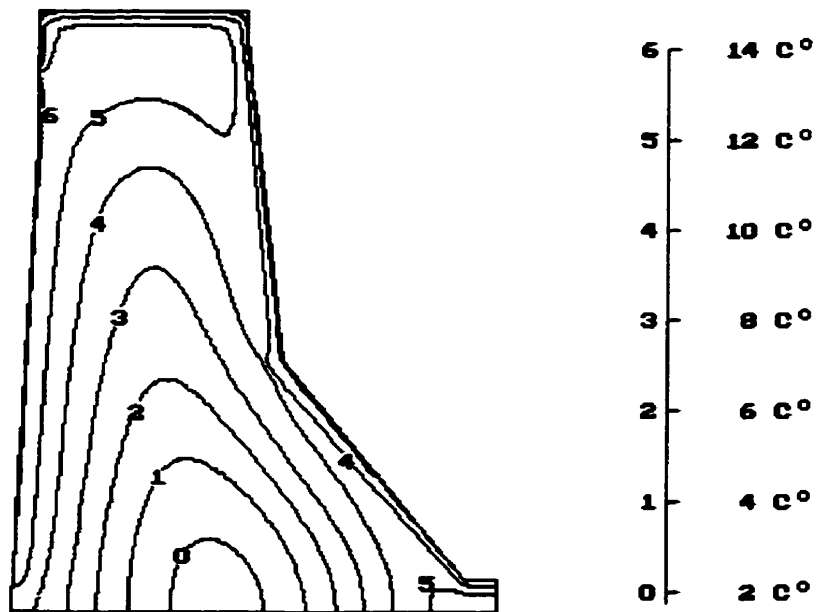


Figure 3.14.10 *Temperature Contour--October*

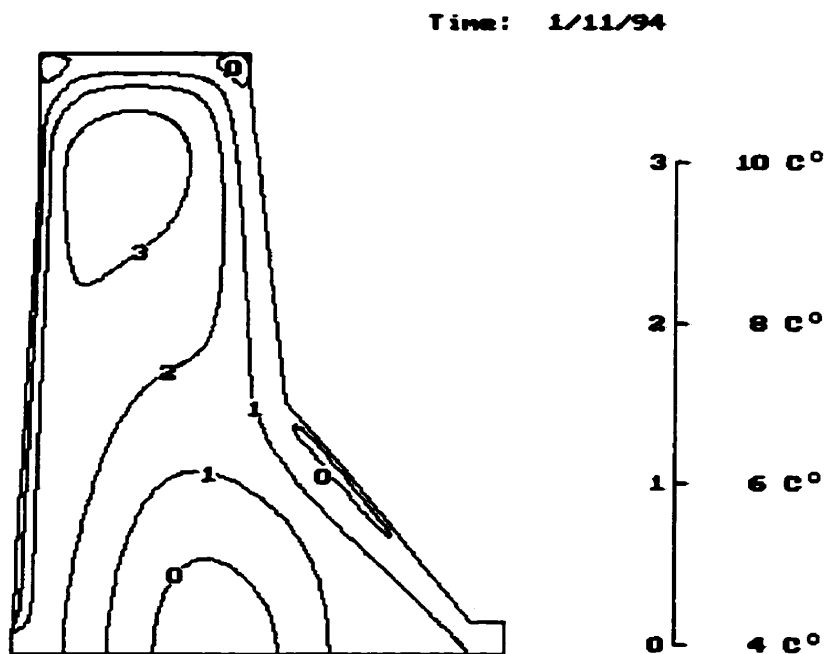
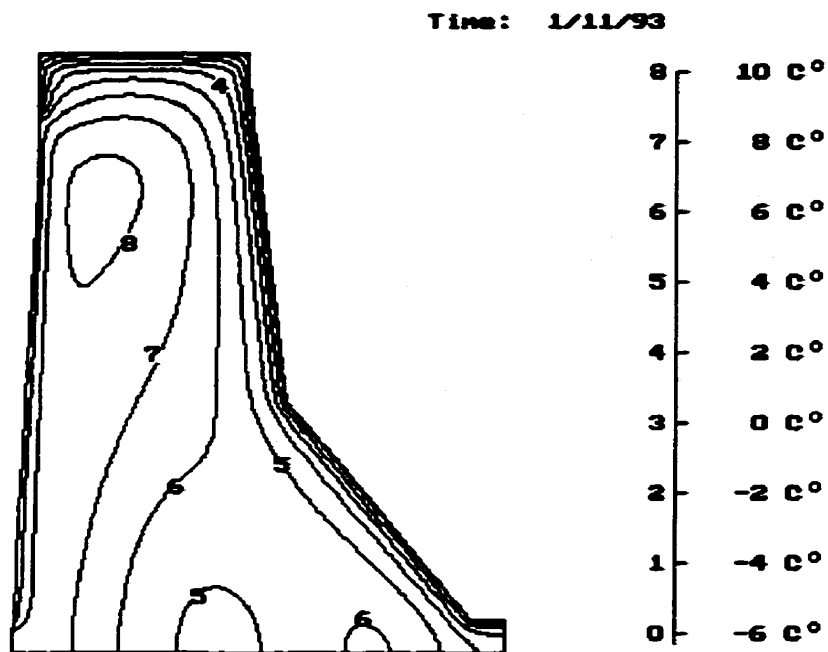


Figure 3.14.11 *Temperature Contour--November*

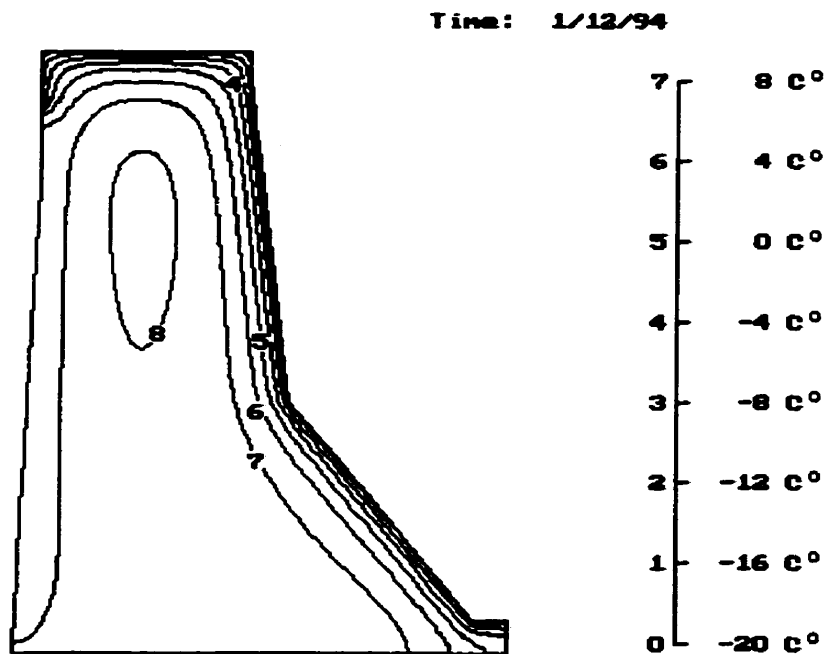
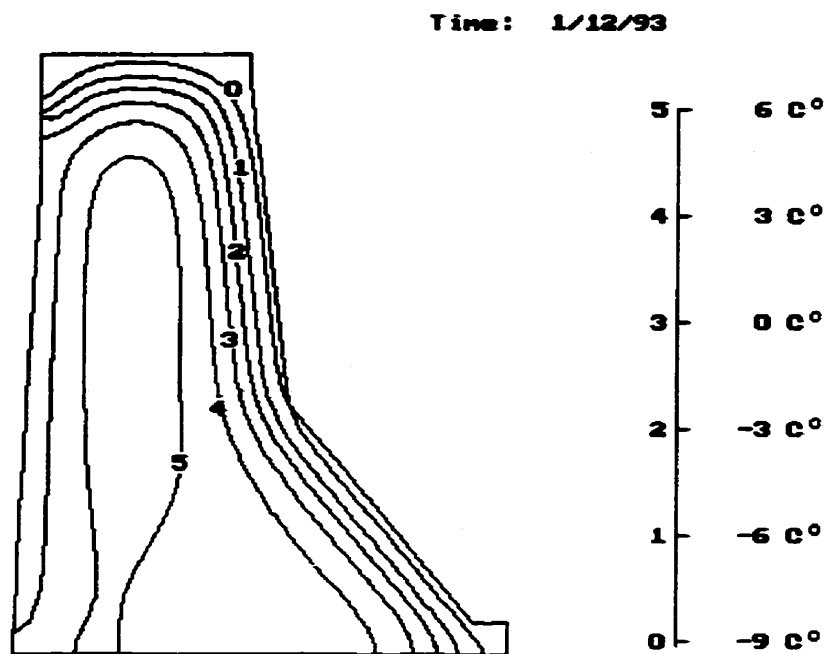


Figure 3.14.12 *Temperature Contour--December*

3.7 Conclusions

The composition of transient temperature solutions is analyzed through *the mode superposition method*. The important role of the fundamental heat frequency in the transient heat transfer analysis is emphasized. Generally, an assumed initial temperature field is used for the temperature analyses of dam structures, i.e., an error has to be introduced in the beginning of the analyses. This study shows that no matter what type of initial condition we assume, after a certain time the error will decrease to a negligible level or the solution will converge to the real solution. The influence of the initial conditions on the long term transient solutions decays exponentially. The convergence time can be well estimated by the fundamental heat frequency. The traditional data repetition criterion for temperature analyses of dams is unnecessary.

The direct integration by the finite difference method is preferred for the analysis of transient heat transfer processes for this study. The integration time step length can also be assessed by the fundamental heat frequency. Analysis shows that the day average even the week average integration scheme is usable in the sense of numerical stability. However, a suitable time step must be chosen based on the variation in the ambient air temperature.

The finite element analysis model for the structure of Long Spruce Generating Station South Transition is established. Much concern is focused on the treatment of the boundary condition, especially the effect of the convective coefficients on the temperature field. A variety of combinations of the convective coefficients between the air-dam and between water-dam are tested. The frequency analysis and transient temperature analysis prove that the thermal behaviors of the dam structure are insensitive to the change of the convective coefficients when the coefficients are in certain range.

The determination of the convective coefficients is a complicated procedure. In certain cases, this complicated process can be avoided by using less accurate values for the convective coefficients.

The transient temperature analysis of the Long Spruce Generating Station South Transition was performed from October 1992 to March 1997, based on the structural and ambient air temperature data. The leaking construction joint in the dam structure was not included in the finite element model of the dam. At the locations of three in-site thermocouples, one near the upstream face, one near the downstream face and one in the center of the dam, the numerical time history of the temperatures has very good agreement with the recorded data by the thermocouples. The good agreement between recorded data and computed results shows that the joint causes negligible disturbances on the temperature field. A continuous structure model can be used for the temperature analysis purpose.

The contours of the isothermal lines show that the temperature field changes cyclically every year. The temperatures inside the dam responds to the ambient air temperature variation quite slowly. The time delay for the temperature change inside the dam is within a few months. A high temperature gradient appears in summers and winters. The leaking joint is within the high temperature gradient zone.

This study indicates that the temperature field of dam structures can be very well predicted by the finite element method. The temperature analysis based on a simplified FEM model of dam structures can supply a reliable heat load base for the displacement and stress analysis caused by a seasonal variation.

Chapter 4

Interface Stress Evaluation Method

4.1 Introduction

Major failure and degradation of hydroelectric structures due to thermal stresses is manifested in the form of crack initiation, crack propagation and construction joint debonding. Cracking and construction joint debonding create a structure discontinuity. For the analysis of this kind of structures, special elements and stress evaluation method are needed to cope with the discontinuity at the interface. A new interface stress evaluation technique, based on the virtual work method, is developed in this chapter.

Stress evaluation is one of the most important steps in the finite element method. Generally, element nodes, which are interest locations for stress evaluation, are the worst sampling points whereas the Gauss integration points provide the locations where stresses are the most accurate. Yet these locations are not physically relevant. In contact problems, such as those encountered in failure analysis, the stresses along element interfaces are needed to be accurate to decide further potential debonding or cracking. The stresses at the Gauss points can not be used directly for this purpose.

As the displacement element method can be interpreted as a least square method with the stress error as the weighted quantity, the stresses obtained directly in the analysis can be expected to oscillate about the exact values (Hinton and Campbell, 1974). Some

general stress smoothing techniques (Oden and Brauchli 1971, Hinton et al. 1975), as well as conjugate interpolation function method (Oden and Reddy, 1973), are proposed to overcome this difficulty.

The interface stresses or the equivalent contact forces at contact surfaces are the main concern in the contact problem. Either Lagrange multipliers or penalty function methods are generally used to impose the non-penetration condition at the interface. Many interface elements or 'gap' elements were developed based on the penalty functions where the element stiffness is actually the penalty numbers (Stadter and Weiss 1979, Beer 1985, Chandrasekaran 1987 and Gens 1988, Ayari and Saouma 1991). For the interface element, the contact pressure is assumed to be proportional to the pointwise penetration. Therefore, the contact pressure strongly depends on the selected penalty numbers (the element stiffness). This usually leads to an oscillatory interface pressure distribution. Reduced integration scheme is commonly suggested. (Kikuchi 1982, Saleeb 1994 and Buczkowski 1997) to smooth this kind of oscillation.

The stress smoothing techniques are developed more or less based on the concepts of stress continuity and interpolation method. The mechanical properties of elements and their interfaces are not invoked. The equilibrium conditions may not be satisfied by the smoothed stresses. Even so, we can only obtain stress values at nodes with less order of precision with respect to that of the displacement values.

In this chapter, a new method for the evaluation of the nodal stresses is proposed. The Gauss point stresses are no longer the starting point. A virtual work principle is set up to consider the mechanics properties of the element interface. Integration processes

are used to obtain a continuous stress distribution along an element interface. An equation is to be solved for the nodal stress values with little computational cost.

4.2 Virtual Work Principle

In many analyses the stress distribution along a line for planar problems is of concern. Now we try to find out the normal and tangential stresses on a line in a two dimensional body. The line consists of element sides as shown in Figure 4.1, with its ends, A and B , on the boundary. This line might be any finite element mesh line or an interface line for contact problems. The elements connected to this line can be classified in two sets:

$\{E_R\}$ elements on the right of line AB , and

$\{E_L\}$ elements on the left of line AB .

Without losing generality, now, we take the subset $\{E_R\}$ into account. A line element set $\{L\}$ consists of the intersection of line AB and the element set $\{E_R\}$:

$$\{L\} = \{E_R\} \cup \overline{AB}: \quad (4.1)$$

If 8-node elements have been used for the FEM model, every line element $l \in \{L\}$ has three nodes as shown in Figure 4.1.

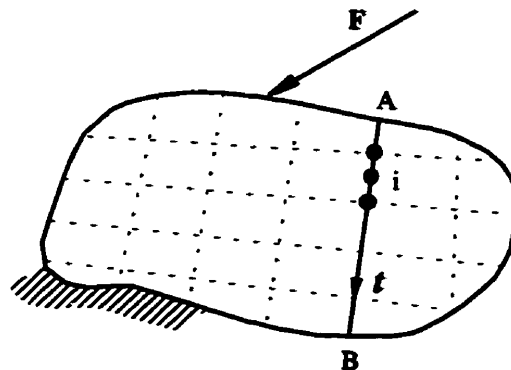


Figure 4.1 *Stress Evaluation Line AB in a Body*

The normal stress σ and tangential stresses τ on line AB are generally continuous functions of the line length t . If there are totally NL nodes on the line, the nodal values of the normal and tangential stresses can be listed in a vector form as:

$$\sigma = \{\sigma_1 \ \sigma_2 \ \cdots \ \sigma_{NL}\}^T \text{ and } \tau = \{\tau_1 \ \tau_2 \ \cdots \ \tau_{NL}\}^T \quad (4.2)$$

Traditionally information for the stress evaluation at nodes comes from the Gaussian point stresses of elements by

$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix} = DB \ \delta_e \quad (4.3)$$

where D is the constitutive matrix, B is the strain displacement relation matrix and δ_e is the element nodal displacement vector. The equivalent nodal forces are seldom used. The internal nodal equivalent force vector f_e contributed by element e is determined by

$$f_e = K_e \delta_e \quad (4.4)$$

where K_e is the element stiffness matrix. For all the nodes on the line AB, the normal and tangential nodal equivalent forces contributed by all the elements in $\{E_R\}$ can be written as:

$$F = \{f_1 \ f_2 \ \cdots \ f_{NL}\}^T \text{ and } Q = \{q_1 \ q_2 \ \cdots \ q_{NL}\}^T \quad (4.5)$$

where f_i and q_i are at the i th node. The contributions of body forces and boundary forces should be included as well, if there are those loads.

If a body is cut along an element interface, such as line AB, the relations between the interface stresses and the nodal equivalent forces should be the same as the relations between distributed boundary loads and the equivalent boundary forces. Actually, the nodal equivalent force, F and Q , can be related to the interface stresses by using the

principle of virtual work. For abbreviation, in the following part we only deal with the normal stress components. The tangential stresses can be treated in a similar way with no additional difficulties.

For any mathematically admissible virtual normal displacement $u(t)$ along the line AB , the virtual work done by the stress $\sigma(t)$ on it is

$$W_{\sigma} = \int_L u(t) \cdot \sigma(t) h dt \quad (4.6)$$

where h is the thickness. The values of the virtual displacement $u(t)$ at nodes on the line AB can be arranged as a vector

$$\mathbf{u} = \{u_1 \quad u_2 \quad \cdots \quad u_{NL}\}^T \quad (4.7)$$

On the nodal virtual displacement $\mathbf{u}$, virtual work can be done by the nodal equivalent force $\mathbf{F}$, so that

$$W_F = \mathbf{u}^T \cdot \mathbf{F} \quad (4.8)$$

The nodal equivalent force $\mathbf{F}$ can be considered equivalent to the stress distribution $\sigma(t)$ when

$$W_F = W_{\sigma} \quad (4.9)$$

or

$$\int_L u(t) \cdot \sigma(t) h dt = \mathbf{u}^T \cdot \mathbf{F} \quad (4.10)$$

Equation (4.10) means that *the virtual work done by the distributed stress on arbitrary virtual displacement is equal to the virtual work done by the equivalent nodal forces on the correspondent nodal virtual displacement*. This virtual work principle forms the base for the interface stress evaluation.

4.3 Interface Stress Evaluation

Because line AB consists of line elements, the integration in equation (4.10) can be done over element by element, *i.e.*

$$\sum_L \int_L u(t) \cdot \sigma(t) h dt = u^T \cdot F . \quad (4.11)$$

There are a few choices to integrate the virtual work over an element.

4.3.1 Gaussian Integration Strategy

The virtual displacement u and stress σ within an element can be interpolated independently by their nodal values via the shape functions:

$$u = N_I u_I + N_J u_J + N_K u_K \text{ and } \sigma = N_I \sigma_I + N_J \sigma_J + N_K \sigma_K, \quad (4.12)$$

where I, J and K stand for three local nodes, N is the shape function. Because elements share nodes, the stress is made continuous along the line AB as shown in Figure 4.2.

Inserting (4.12) to equation (4.11), we obtain

$$\int_L u(t) \cdot \sigma(t) h dt = u_e^T S_e \sigma_e \quad (4.13)$$

where the local virtual displacement u_e and stress σ_e arrays are defined as

$$u_e = \{u_I \quad u_J \quad u_K\}^T \text{ and } \sigma_e = \{\sigma_I \quad \sigma_J \quad \sigma_K\}^T . \quad (4.14)$$

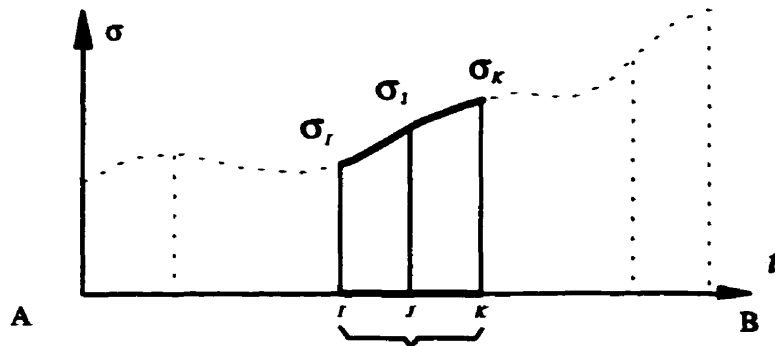


Figure 4.2 Continuous Stress Distribution along Line AB

The *local stress evaluation matrix* S_e is defined as:

$$S_e = \left[\int_l h N_i N_j dt \right]_{3 \times 3} \quad (4.15)$$

The integration in formula (4.15) can be carried out by using 3-point Gauss integration rule. When the thickness h is constant, we have:

$$S_e = \frac{hl}{30} \begin{bmatrix} 4 & 2 & -1 \\ 2 & 16 & 2 \\ -1 & 2 & 4 \end{bmatrix} \quad (4.16)$$

where l is the length of the line element.

The total virtual work done by the stress on line AB , then is:

$$W_\sigma = \sum u_e^T S_e \sigma_e \quad (4.17)$$

where the summation is over all the line elements of set $\{L\}$. The local stress and displacement vectors can be expanded to the order of u and σ , respectively. Therefore, equation (4.17) becomes

$$W_\sigma = u^T S \sigma \quad (4.18)$$

where S is the *stress evaluation matrix* for line AB . The process for assembling matrix S from the elemental matrix S_e is the same as the assembling process of the global stiffness matrix. Finally, from equation (4.10) and (4.18) there is:

$$u^T S \sigma = u^T F \quad (4.19)$$

Because of the arbitrariness of u , we obtain

$$S \sigma = F, \quad (4.20)$$

by which we can solve for the nodal stress values.

It should be mentioned that for the integration in equation (4.15), 3-point Gauss integration has to be used (accurate integration when h is constant), otherwise the stress evaluation matrix S could be singular.

4.3.2 Newton-Cotes Integration Strategy

For the Newton-Cotes integration strategy, the integration is evaluated at nodal points. An immediate advantage of this strategy is that no interpolation of the virtual displacement and the stress is required. Within a line element there is

$$\int_l u(t) \cdot \sigma(t) h dt = \frac{hl}{6} [u_I \sigma_I + 4u_J \sigma_J + u_K \sigma_K] \quad (4.21)$$

when the thickness h is constant. The local stress evaluation matrix S_e now is a third order diagonal matrix,

$$S_e = \frac{hl}{6} \begin{bmatrix} 1 & 0 & 0 \\ 0 & 4 & 0 \\ 0 & 0 & 1 \end{bmatrix}. \quad (4.22)$$

Therefore, it is easy to find that the global stress evaluation matrix S is also a diagonal one. This feature enables us to solve the stress equations (4.20) with very little computational cost.

In another aspect, the Gauss integration strategy results in a tri-diagonal global stress evaluation matrix. The error of the nodal equivalent force at one node is able to transfer to others. We can see later that the poor stress quality near the boundary will influence the stress values at the nodes near the boundary. A similar phenomenon was reported by Gens (1988), when a 2-D interface element was developed. However, with the Newton-Cotes integration strategy, there is no such error transfer between nodes

along the line *AB*. Numerical result shows that the Newton-Cotes strategy is superior to the Gauss strategy.

4.4 Numerical Results

Two typical problems have been chosen to test the difference between the two integration strategies and the efficiency of the stress evaluation method.

4.4.1 Problem 1. *Cantilever Beam*

The first problem is a cantilever beam under a distributed load as shown in Figure 4.3. The shear stress distribution along the neutral axis is examined. This problem was used by Zienkiewicz (1977) to show the traditional way for stress evaluation. He demonstrated that the quality of the stresses is very poor at the nodal points.

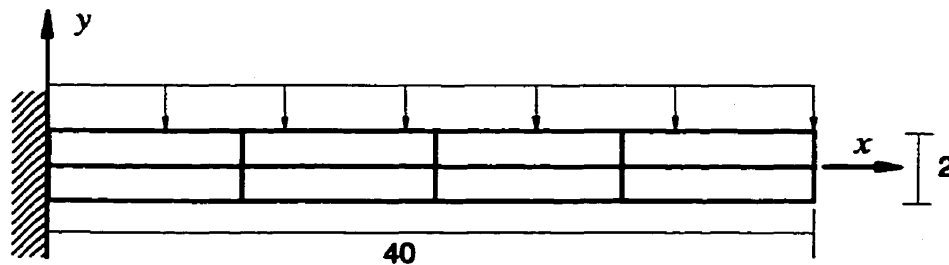


Figure 4.3 *A Cantilever Beam under Distributed Load*

In order to make the neutral line as an interface line, two 8-node elements are placed along the depth. Figure 4.4 shows the shear stress values at nodes obtained by both integration strategies with 4 elements along the length direction. Comparisons have been made between the finite element results and the closed form solution by the beam theory. It can be seen that the points obtained by Newton-Cotes strategy are generally closer to the beam solution line than the points by Gauss strategy. The stress oscillation can be found near the two ends, whereas in the center of the line the stress values are very

close to the beam solution. A significant phenomenon is that the stress quality is consistently good at the mid-side nodes of elements. Even with the coarse mesh the quality of the stress values at the mid-side nodes is satisfactory.

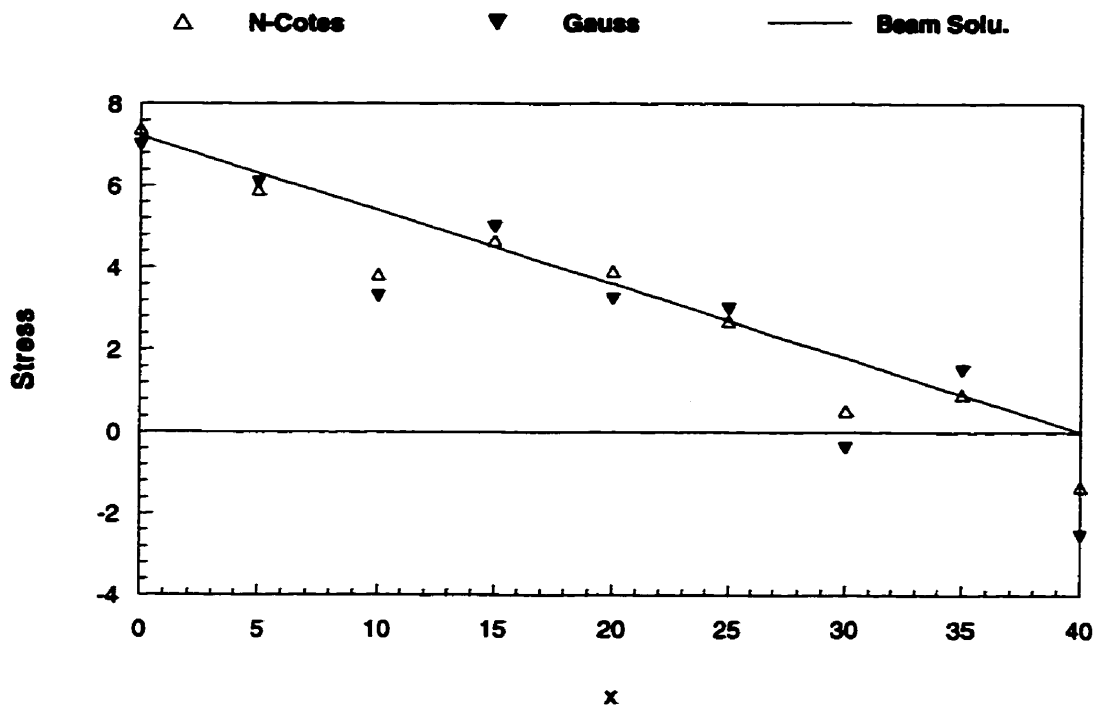


Figure 4.4 *Shear Stress Values at Nodal Points (4×2 mesh, problem 1)*

A refined 8×2 mesh is also used. The result is shown in Figure 4.5. In this case, the stress oscillation is even more serious near the fixed end. The stress points obtained by Gauss strategy are worse than those from the Newton-Cotes strategy, especially for the end-of-side nodes of elements. The beam solution could not be correct near the fixed end. A local fast variation stress state is expected, which is mainly responsible to the stress oscillation. However, we can still find the stresses in the mid-side points are almost on the close form solution line.

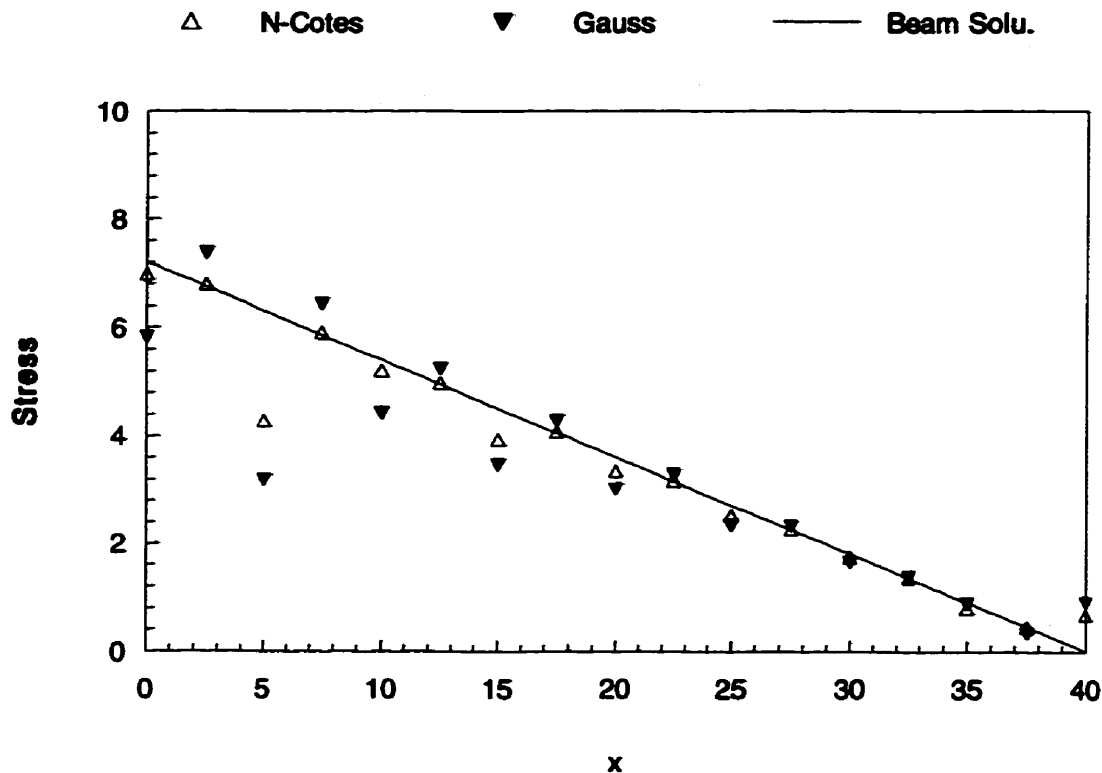


Figure 4.5 *Shear Stress Values at Nodal Points (8×2 mesh, problem 1)*

The good stress quality at the mid-side nodes is actually the heritage of the good quality of the nodal equivalent force at the mid-sides nodes. It is reasonable for us to only use the stresses at mid-side nodes as the sample points. Better end-of-side node stress values can be obtained by interpolation of the sample points.

4.4.2 Problem 2. *Square Plate with a Circular Hole*

The problem of a square plate with a circular hole is often used as a test problem because of the existence of the closed form solution. A quarter of the plate is meshed as shown in Figure 4.6, and a unit uniform distributed stretch σ is put in the y -direction. The tensile stress σ_y along the x -axis is well known as (Timoshenko 1954):

$$\sigma_y = \frac{1}{2} \left[2 + \left(\frac{a}{x} \right)^2 + 3 \left(\frac{a}{x} \right)^4 \right] \sigma \quad (4.19)$$

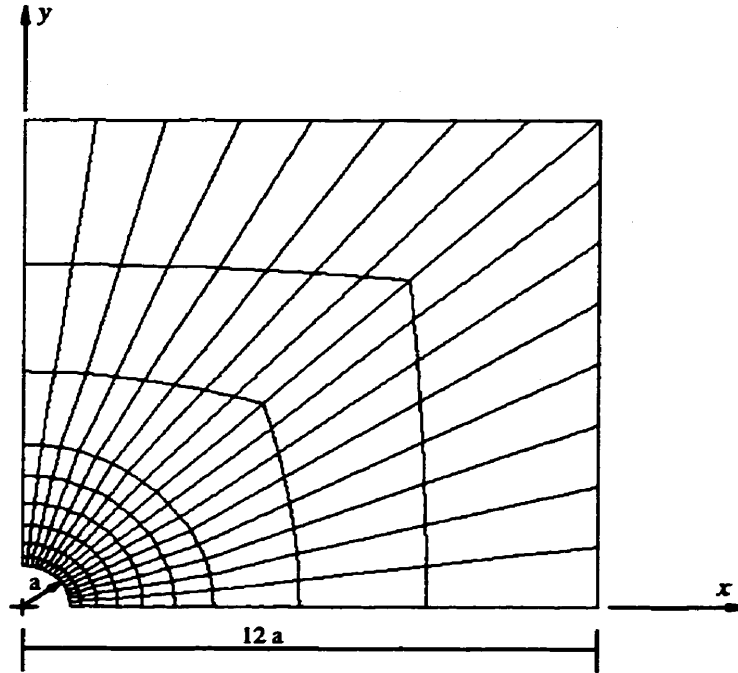


Figure 4.6 *Element Mesh for Problem 2 (coarse mesh)*

We take the stress values at the mid-side nodes as the sample points. For the end-of-side nodes, the stress values are obtained by Lagrange interpolation from the three nearest sample points. Although the mesh in Figure 4.6 is relatively coarse, very good stress values can be seen in Figure 4.7. The relative error distribution is depicted in Figure 4.8. The maximum relative error is less than 6%. In Figure 4.7, the dashed curve corresponds to the so-called consistent stress (Oden and Reddy 1973). From the boundary of the hole to one time of the radius distance inside, only three elements are laid out, in comparison with ten elements in (Oden and Reddy 1973). Better stress values have been obtained.

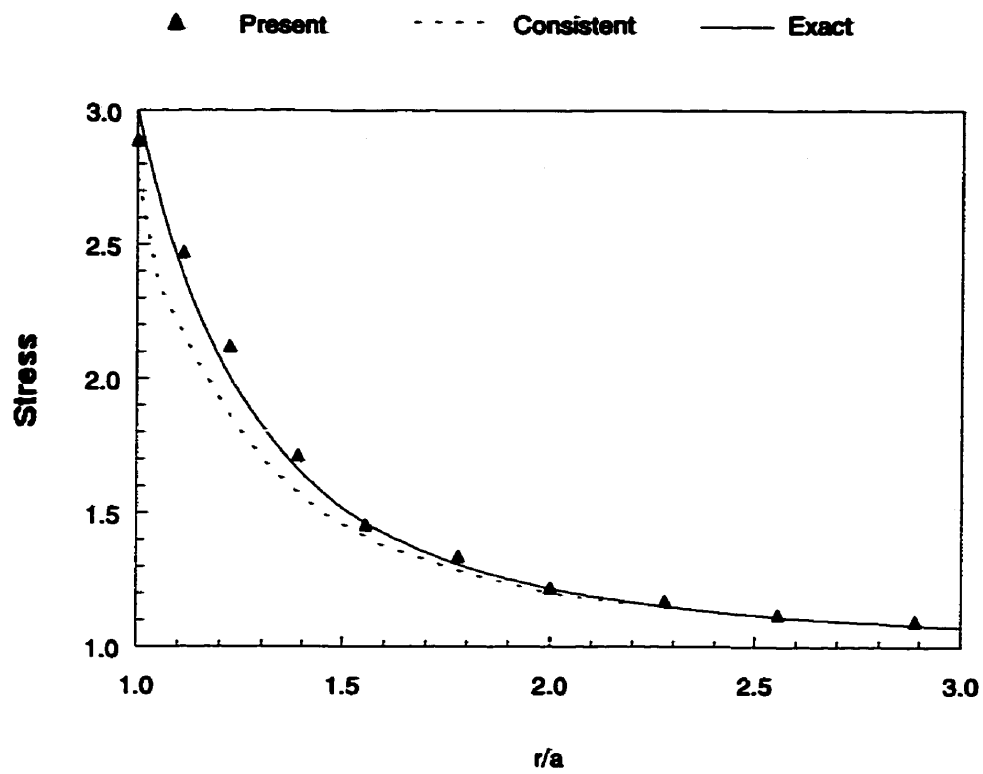


Figure 4.7 *Tensile Stress Values at Nodal Points (coarse mesh, problem 2)*

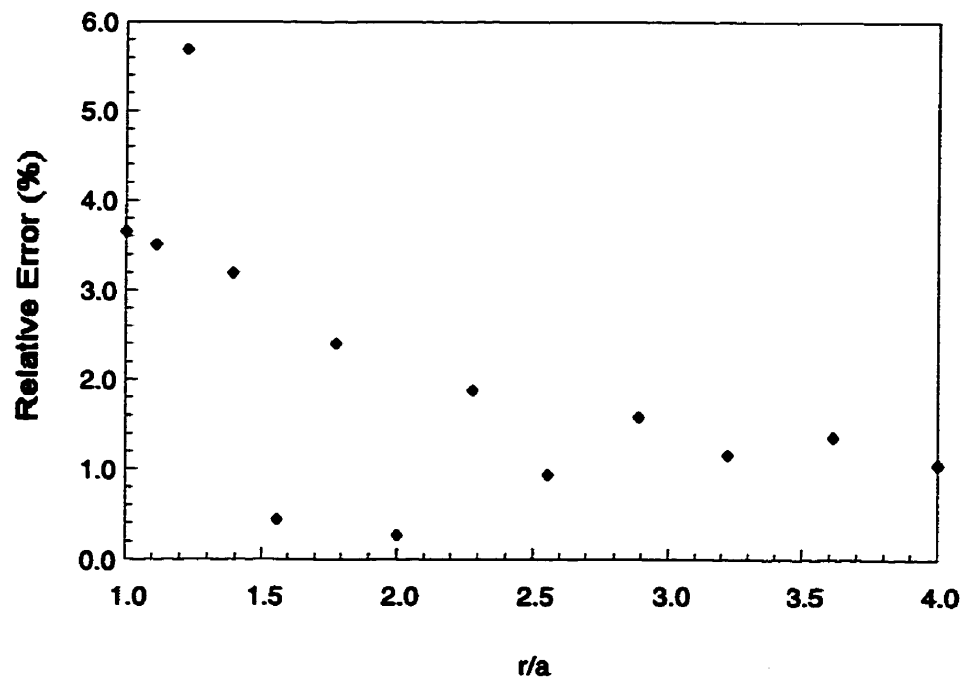


Figure 4.8 *Error Distribution for Coarse Mesh (problem 2)*

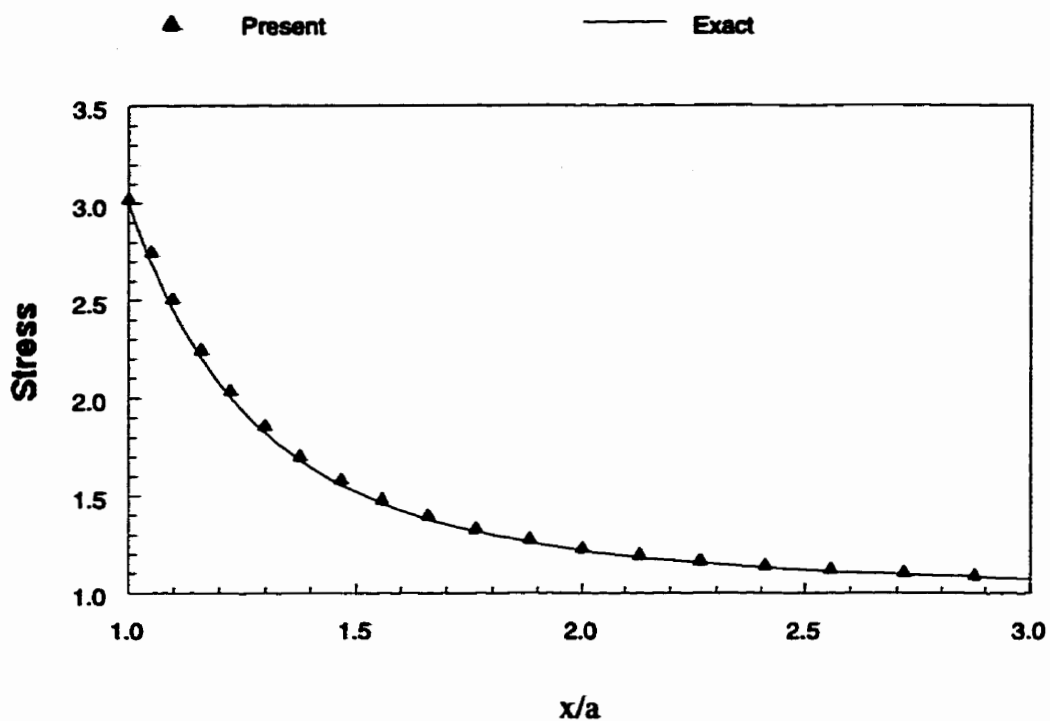


Figure 4.9 *Tensile Stress Values at Nodal Points (refined mesh, problem 2)*

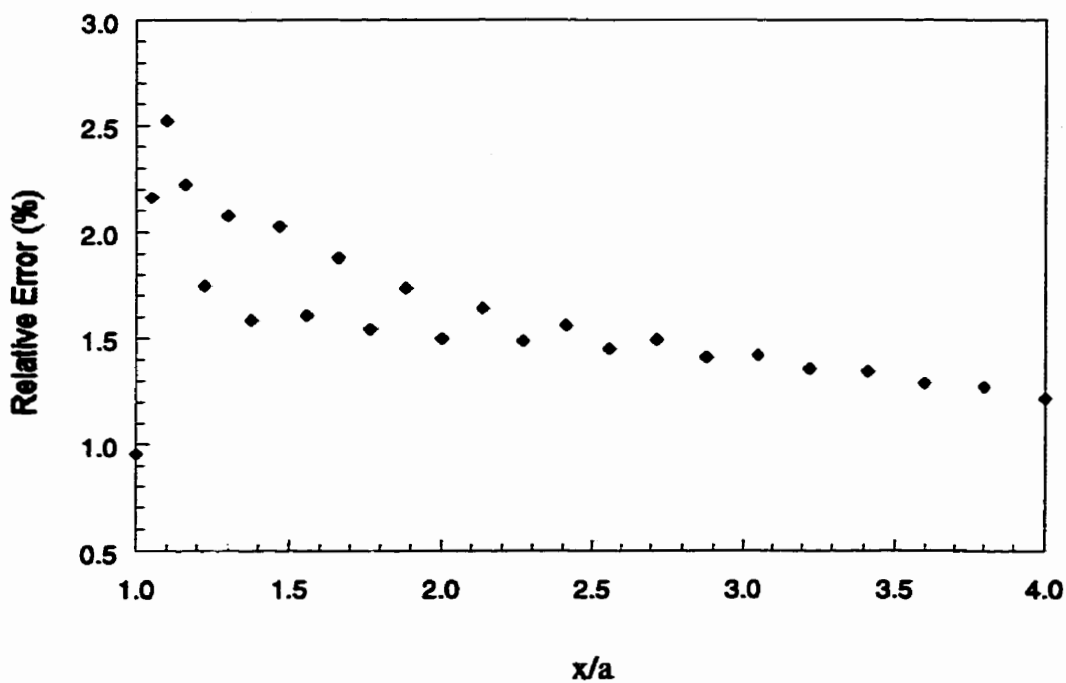


Figure 4.10 *Error Distribution for Refined Mesh (problem 2)*

The mesh in Figure 4.6 is doubly refined. The corresponding results can be seen in Figure 4.9 and 4.10. In this case the stress points are almost on the solution curve. The relative errors are uniformly distributed among the nodes within a maximum of 2.5%.

4.5 Conclusions

For this new interface stress evaluation method, the starting point is the virtual work principle. The Gaussian point stress is never used. The stress is made continuous by means of a physical law rather than a purely mathematical technique. The stress evaluation equations are derived from integration. No differentiation step is involved during the stress evaluation process. Therefore, no more numerical errors will be introduced to the final stress results. The accuracy of the stress values depends mainly on the nodal equivalent forces of element. Generally at the mid-side nodes the accuracy is of higher order than at the end-of-side nodes. To keep the higher order accuracy, it is better to take the stress values at the mid-side nodes as the sample points. Although in some case the stress quality is good at the end-of-side nodes, we evaluate the stresses by interpolation of stress values at the mid-side nodes.

Numerical results show that the Newton-Cotes integration strategy is more suitable for this stress evaluation method than the Gauss integration strategy. The stress evaluation process becomes independent of the choice of interpolation pattern between nodes when the Newton-Cotes integration strategy is used.

In our analysis, we only consider the stress evaluation line as a straight line for simplicity. It is not difficult to extend this method to the case of a curved stress evaluation line. It is expected that this new stress evaluation method will find more applications in a variety of interface stress analyses and contact problems.

Chapter 5

Motion Analysis of Dam Structure with Cracked Joint

5.1 Introduction

The temperature analysis in Chapter 3 shows that seasonal weather changes cause severe temperature variations in dams. Thermally induced cyclic deformations and stresses, combined with the freeze-thaw process, are the main driving forces for concrete cracking and strength degradation of dam structures. Temperature variations also cause the movement of joints and result in progressive debonding of construction joints. Cracks created either by joint debonding or caused by material failure severely deteriorate the structural integrity and operational safety of concrete dams. Moreover, penetration of water through cracks increases the moisture of concrete, incurs chemical reactions and accelerates the deterioration processes. Water leakage can be observed in many dams. Extensive rehabilitation programs need be performed to prolong the operation lives of concrete dam structures.

Long Spruce Generating Station was constructed between 1974 and 1978. The south transition structure has leaked seasonally at the second last horizontal construction joint since 1980 (Manitoba Hydro Report, 1994). Although the joint was grouted in 1986 and initially there was a reduction in leakage through the construction joint, leakage has

reoccurred. It seems that continuous thermally induced movement also damaged the injected grouting. Instrumentation was installed across the joint to monitor the thermally induced movement. Analyses of the recorded displacement data at different locations along the construction joint show that the connection of the joint was totally broken. The concrete above the construction joint acts as a block piled on the dam body below the joint. The block curls during cold weather, and water freezes in the resulting opening. During a warm spell, the block uncurls but is unable to flatten due to ice build up. Therefore the center of the block is jacked up, as shown schematically in Figure 5.1.

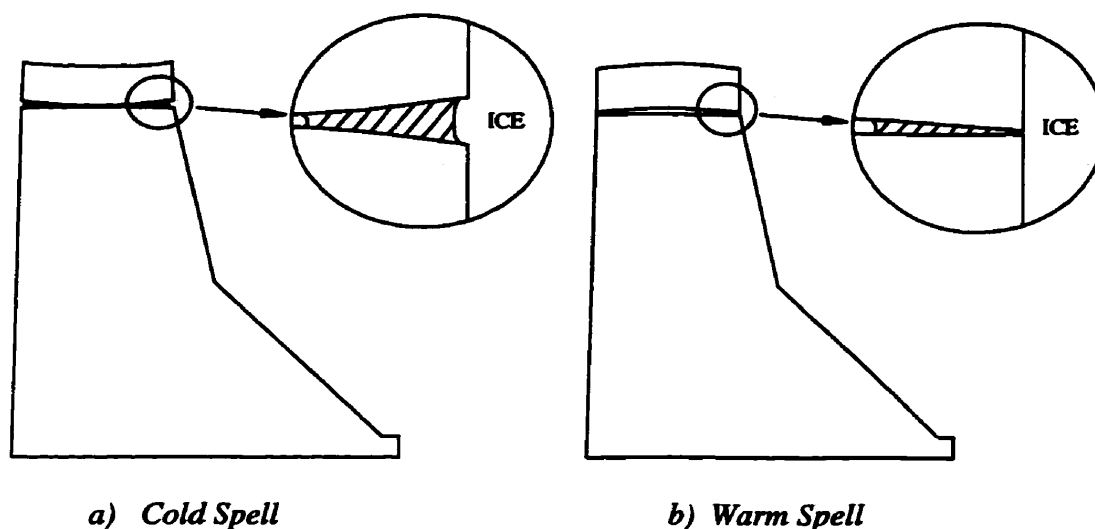


Figure 5.1 *Deformation Pattern of the Dam with Debonded Joint*

The knowledge of the deformation, especially the opening and closing displacements of the joint surface is very important for the rehabilitation studies of this type of dam. Motion analysis of the damaged structure that enables us to predict the response of the dam structure to the environmental temperature variation is an urgent task.

The debonded construction joint actually formed a through-thickness crack in the dam. The opening and closing displacement analysis of the crack surface is a type of contact problem between two solid bodies. In order to follow the seasonal motion of the dam with a debonded construction joint, an efficient numerical method has to be developed to deal with the contact problem associated with the possible opening and closing of the joint.

In this chapter, a new interface element is developed to model the motion of a dam structure with a debonded construction joint. The new element is robust and may be used in many kinds of interface and crack analysis problems. A special iteration process is used to find the contact zone of the two surfaces of the debonded joint. The interface stress evaluation method developed in Chapter 4 is used to calculate the contact stress. The motion of the upper block of the south transition of the Long Spruce Generating Station, due to the thermal load established in Chapter 3, is then studied.

5.2 An Interface Element Formulation

Contact problems have recently been of considerable interest. Extensive research has been carried out for solving a variety of simple to complex elastic boundary value problems. The contact surface may be initially open or closed, and with or without friction. Contact conditions can be imposed directly from kinematic considerations by imposing geometric compatibility of the contacting surface during the incremental loading process (Francavilla and Zienkiewicz 1975, Satcheva and Ramakrishnan 1981, Bathe and Chaudary 1985, and Ayari and Saouma 1991). The advantage of this method is that the interface displacement condition can be easily imposed and the algorithms can be made independent of interface material constitutions. Usually pairs of contactor nodes are

laid out along the potential contact surface. The compatibility of displacement is imposed by means of the relative normal and tangential nodal displacements.

Many interface elements or 'gap' elements have been developed based on the penalty functions where the element stiffness is actually the penalty numbers (Stadter and Weiss 1979, Beer 1985, Chandrasekaran 1987 and Gens 1988). For an interface element, the contact stress is usually assumed to be proportional to the pointwise penetration. Strictly speaking, the pointwise penetrations should be zero or of negligible error order. Therefore the contact stresses strongly depend on the selected penalty numbers (the element stiffness). A new interface stress evaluation method is developed in Chapter 4. It is expected that this new method can yield higher quality interface stresses and supply a good base for a new interface element. Meanwhile, for the present problem, the cracked joint might be either totally debonded when the temperature inside the dam is high enough in the summer, or bonded by ice and grout. An element that can simulate a variable thickness interface layer with zero to certain linkage stiffness is needed. Various kinds of linkages between the opposite nodes on interfaces are going to be set up for the new element. The open-close displacement of a crack or joint interface is modeled by changing the stiffness of the interface element. The compatibility of the interface displacements is easily imposed.

5.2.1 Kinematic Relations of the Interface Element

The interface element developed here consists of six nodes on a straight line. The element local numbering and t - n coordinates under global x - y coordinates are illustrated in Figure 5.2. Local nodes 1,2,3 on one side correspond to local nodes 4,5,6 on the other side of the element, respectively. For the time being, the thickness of the interface

element is zero, i.e., the correspondent nodes have the same coordinates. The local nodal displacement vector U and the global nodal displacement vector δ are connected by the coordinate transformation matrix Q as:

$$U = Q \delta \quad (5.1)$$

where

$$U = \{U_1, V_1, \dots, U_6, V_6\}^T, \quad \delta = \{u_1, v_1, \dots, u_6, v_6\}^T$$

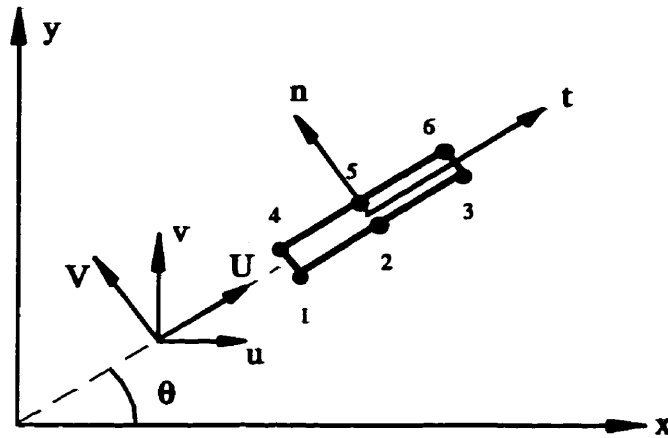


Figure 5.2 *Coordinates and Node Numbering for the Interface Element*

The coordinate transformation matrix is a diagonal block matrix:

$$Q = \begin{bmatrix} T & & \\ & \ddots & \\ & & T \end{bmatrix}, \quad \text{with } T = \begin{bmatrix} \cos \theta & \sin \theta \\ -\sin \theta & \cos \theta \end{bmatrix} \quad (5.2)$$

The local relative nodal displacement vector ΔU can be expressed by vector U via

$$\Delta U = H U \quad (5.3)$$

where

$$\Delta U = \{U_4 - U_1, V_4 - V_1, \dots, U_6 - U_3, V_6 - V_3\}^T$$

and

$H = [-I \ I]$, I being the identity matrix of 6th order.

The relative displacements between side 1-2-3 and side 4-5-6 are achieved by means of the standard quadratic shape functions $N_i(r)$:

$$\begin{Bmatrix} \Delta U \\ \Delta V \end{Bmatrix} = N \Delta U \quad (5.4)$$

where

$$N = \begin{bmatrix} N_1 & 0 & N_2 & 0 & N_3 & 0 \\ 0 & N_1 & 0 & N_2 & 0 & N_3 \end{bmatrix}$$

The non-penetration condition requires that

$$\Delta V \geq 0 \quad (5.5)$$

at any point on the interface. Strictly, the condition (5.5) is not guaranteed by confining the normal relative nodal displacements to be positive or zero. Small overlap of the two sides of the interface element can occur even when none of the normal relative nodal displacements is negative. However, the amount of overlap is of a small order in comparison with the relative nodal displacements and, therefore, can be neglected.

5.2.2 Stiffness Matrix

The two sides of the interface element are linked by zero-thickness springs in both normal and tangential directions. The choices of the stiffness of the springs in the normal and tangential directions allow us to form various types of interface elements. The normal or tangential displacements can be made either continuous or discontinuous across the interface independently. The general constitutive matrix of the interface element can be expressed as

$$D = \begin{bmatrix} k_t & k_{nt} \\ k_{nt} & k_n \end{bmatrix} \quad (5.6)$$

where k_t is the tangential stiffness and k_n is the normal stiffness, k_{nt} corresponds to a certain kind of coupling effect between the normal and tangential stiffness. These stiffness coefficients are generally functions of many factors, such as the interface traction, relative displacements, surface smoothness, temperature, etc. For the present case the stiffness coefficients, k_n and k_t , are considered to be functions of the interface traction only. The coupling effect is not taken into account, so that k_{nt} is set to zero. More consideration about the choice of the spring stiffness will be discussed later in this chapter.

The stiffness matrix of the interface element can be determined by the virtual work principle. Let F be the nodal force vector of the element in the local coordinate system,

$$F = \{F_{t1}, F_{n1}, \dots, F_{t6}, F_{n6}\} \quad (5.7)$$

The virtual work done by F on the virtual nodal displacement U must be equal to the internal virtual work of the interface element, that is

$$\int_l (N\Delta U)^T D (N\Delta U) dl = U^T F \quad (5.8)$$

where l is the length of the interface element. Equation (5.8) results in

$$K_l U = F \quad (5.9)$$

by the arbitrariness of the virtual displacements. Therefore, the stiffness matrix of the element in local coordinates should be

$$K_l = \int_l H^T N^T D N H dl \quad (5.10)$$

The local nodal force vector F can be transformed to the nodal force vector f in global coordinate by

$$f = Q^T F \quad (5.11)$$

Using transformations (5.1) and (5.11), we express equation (5.9) in the global coordinate system as:

$$Q^T K_l Q \delta = f \quad (5.12)$$

Finally, combining equations (5.10) and (5.12), we obtain the element stiffness K in global coordinate system as

$$K = \int_l (NHQ)^T D (NHQ) dl \quad (5.13)$$

The integration schemes for the stiffness matrix were studied by Gens, et al. (1988) and Hohberg (1988) for their interface elements. It was found that, with the traditional Gauss integration, the interface stresses exhibit large oscillations. If the Newton-Cotes integration scheme is adopted, the results become quite satisfactory. For the three point Newton-Cotes integration scheme, the integration points are located at the nodes and the stiffness matrix is made diagonal. This formulation is of the so-called linkage type of interface element as classified in (Gens, et al. 1988).

5.2.3 Iteration Process

The non-linearity of a contact analysis mainly comes from the uncertainty of the contact zone and the contact forces. An incremental loading process is usually invoked to find the contact zone step by step (Francavilla and Zienkiewicz 1975, Sachdeva and Ramakrishnan 1981, Rahman and Rowlands 1984, and Bathe and Chaudary 1985). The configuration and the global stiffness matrices of structures are more or less modified in

the process to find the contact zone. In the present model, it is intended that modification is made only by the change of the interface element stiffness.

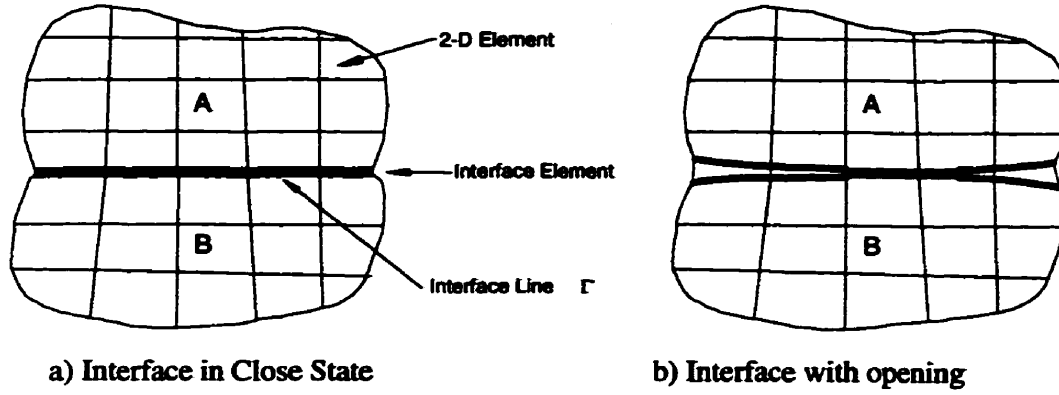


Figure 5.3 *Interface Elements in a Body*

The potential energy of a structure with an interface inside can be formulated as:

$$\Pi = \frac{1}{2} \Delta^T K_{AB} \Delta + \frac{1}{2} \delta^T K_{int} \delta - \Delta^T f \quad (5.14)$$

where K_{AB} is the global stiffness matrix contributed by bodies A and B as in Figure 5.3, Δ is the nodal displacement vector of the total structure, K_{int} is the interface stiffness matrix and f is the equivalent load vector. The interface nodal displacement vector δ is a subset of the structural nodal displacements Δ . In the potential energy expression (5.14), the first term comes from the deformation energy of the body A and B, where the matrix K_{AB} is independent of the deformation. The second term is the internal work done by the contact force on the interface relative displacement, where the matrix K_{int} is deformation dependent. The third term represents the work done by the applied load. It is the second term that causes the non-linearity in the potential energy minimizing process.

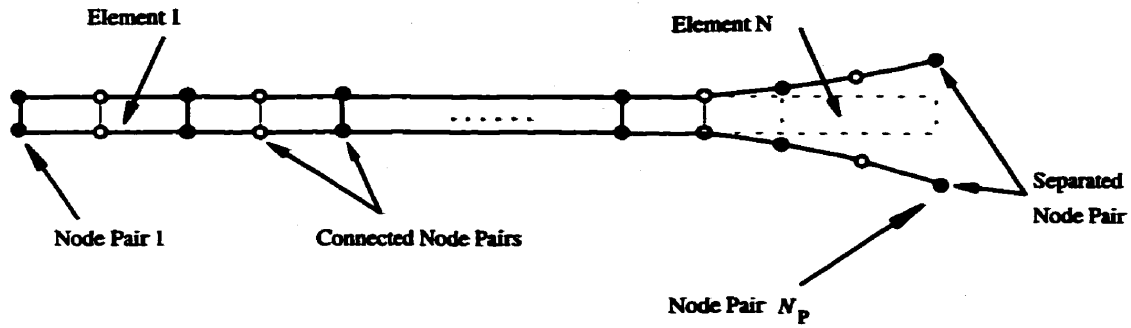


Figure 5.4 *Split of Interface Line*

For a contact problem, no normal tensile traction can be transferred across the interface. On condition of the normal displacement $\Delta V > 0$, the stiffness of an interface element must be reduced to zero, so that no linkage exists between the two sides of the interface. In this case, the interface element splits and free surface conditions can be resumed as illustrated in Figure 5.4. When the two surfaces are in contact, the compatibility requires that the normal relative displacement be zero ($\Delta V = 0$). Theoretically this requirement can be satisfied when the normal stiffness of the element is infinite. If the interface is friction free, in the above two cases there is:

$$\delta^T K_{int} \delta = 0 \quad (5.15)$$

because of either a zero contact force or a zero relative displacement. In practical computation, this term may act as a penalty function to force the compatibility condition in the minimization of the potential energy (5.14). It is easy to make the stiffness of an interface element zero by setting $k_n=0$. However in the case of contact, only finite stiffness can be achieved numerically by setting k_n as a very large number M_p . Figure 5.5 shows the relation between the normal stiffness and the relative displacement.

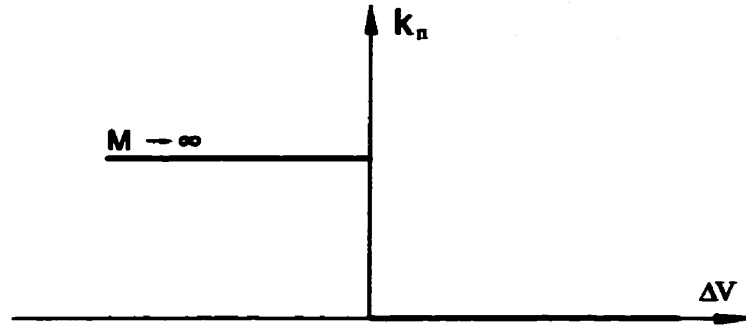


Figure 5.5 *Constitutive Law for Normal Stiffness*

A certain amount of penetration displacement V_p has to be allowed, however, it should be controlled to be lower than a certain level. The intensity of the internal work done on the interface in a numerical process can be estimated as:

$$W_{int} = \frac{1}{2} M_p V_p^2 \quad (5.16)$$

In the practical computational process, W_{int} need be made as small as possible to minimize the potential energy. A larger value of M_p results in a high internal work level.

Suppose the interface line is Γ with N_p pairs of nodes on it (See Figure 5.4.) The interface is divided by N elements. Because the stiffness matrices of the interface elements are actually evaluated at their nodes, the node pair set $\{N\}$ is divided into two subsets: $\{N_o\}$ and $\{N_c\}$, so that

$$\{N\} = \{N_c\} \cup \{N_o\} \quad (5.17)$$

For the node pairs in $\{N_c\}$

$$k_n = M_p \quad (5.18)$$

and for the node pairs in $\{N_o\}$

$$k_n = 0 \quad (5.19)$$

The Algorithm can be summarized as follows:

Step 0 :

- *Set initial states of all interface elements to close state, i.e.,*

$$\{N\} = \{N_c\}, \quad \{N_o\} = \{\emptyset\} \quad \text{and} \quad k_n = k_{n0} \quad (5.20)$$

- *Then*

$$\text{Min: } \Pi = \frac{1}{2} \Delta^T K_{AB} \Delta + \frac{1}{2} \delta^T K_{int} \delta - \Delta^T f; \quad (5.21)$$

- *Evaluate the interface stresses at every node pair $P \in \{N_c\}$, according to the displacement results of this step;*
- *Modify the normal stiffness k_n (as well as k_t , if necessary) in terms of the stresses to*

$$k_n = \begin{cases} M_p & \text{for normal stress} \leq 0 \\ 0 & \text{for normal stress} > 0 \end{cases}; \quad (5.22)$$

- *Update the sets $\{N_o\}$ and $\{N_c\}$ by the value of k_n .*

In the initial stage, the interface is supposed to be sewn by interface elements with a certain stiffness. The structure is treated as if there is no interface. The structure and boundary condition become definite. The solving process is the same as that in a linear elastic problem. As mentioned before, the opening of the interface is represented by the separation of the node pairs. Node pairs in tension are going to split, and the related stiffness is going to be released. In the next step, these node pairs are changed to the subset $\{N_o\}$.

Step k :

- *Calculate the interface element stiffness and assemble them to reform the interface stiffness matrix K_{int} , while matrix K_{AB} and load vector f are kept unchanged;*

- *Then*

$$\text{Min: } \Pi = \frac{1}{2} \Delta^T K_{AB} \Delta + \frac{1}{2} \delta^T K_{int} \delta - \Delta^T f; \quad (5.23)$$

- *Evaluate the interface stresses at every node pair $P \in \{N_c\}$, according to the displacement results of this step;*
- *Modify the normal stiffness k_n (as well as k_t , if necessary) in terms of the stresses to*

$$k_n = \begin{cases} M_p & \text{for normal stress} \leq 0 \\ 0 & \text{for normal stress} > 0 \end{cases}; \quad (5.24)$$

- *Update the sets $\{N_o\}$ and $\{N_c\}$ by the value of k_n .*
- *If the normal stress at every node pair $P \in \{N_c\}$ is smaller than zero, stop the iteration process; otherwise go to step $k+1$.*

In *step k* of the iteration process, also a linear elastic problem needs to be solved. After assembling the interface stiffness matrix K_{int} , every operation is similar to that in *step 0*. The shear stiffness k_t can also be changed at every node pair for every iteration step according to some special requirement.

The iteration process is actually an interface traction release process. In every step the release of the spurious tensile traction makes the potential energy of the structure monotonically decrease, while the external load remains unchanged. In the final stage of the iteration process, it is possible that at the end of the contact zone, the stiffness release at this end may result in penetration at a neighbor node over the allowable level. However, recovery of the previous stiffness at this end may result in an interface tensile stress over its tolerance. In this case, the contact zone must end between these two nodes. The error of the contact zone range is within the size of an interface element.

The choice of the interface stiffness M_p and the evaluation of the interface stress play crucial roles for a successful iteration process. A small value of M_p will result in a large penetration displacement V_p , while a too large M will ruin the iteration. As the opening is actually determined by the interface stresses, high quality interface stress is critical for the iteration process. In typical finite element methods, stresses are only output at the Gauss points. The stress quality at a nodal point is usually poor. The interface stress evaluation method developed in Chapter 4 now can be used for this task.

5.2.4 Constitutive Law of Interfaces

The normal relative displacement ΔV of an interface can be related to the normal stress σ by

$$\sigma \cdot h = k_n \Delta V \quad (5.25)$$

where h is a geometry parameter, usually the thickness of the element to which the interface element is connected. A simple constitutive relation for k_n has been given in Figure 5.5. If we want to control the interface penetration displacement V_p at a certain level V_{pc} , i.e.,

$$V_p < V_{pc} \quad (5.26)$$

so that from equation (5.16), there is

$$M_p > \frac{2 W_{int}}{V_{pc}^2}$$

This means that

$$M_p \propto \frac{1}{V_{pc}^2} \quad (5.27)$$

A very large value of M_p has to be used. In another respect, the simple interface stiffness model shown in Figure 5.5 causes a severe stiffness jump near the end of a contact zone. Oscillatory stresses are often encountered. To overcome this shortcoming, we can take M_p as a variable instead of a constant.

The normal pressure at the end of a contact zone must be zero because the surfaces in the separation zone are stress free. To keep the penetration displacement small, a large M_p is unnecessary. If we let the normal stiffness relate to the normal stress, the stiffness jump can be overcome. Figure 5.6 shows a new interface constitutive relation. The normal stiffness k_n now is set to depend on the normal pressure of the interface p linearly, i.e.

$$k_n = \lambda p \quad (5.28)$$

where λ is a constant and $p = -\sigma$. In this way, the stiffness coefficient k_n is made a continuous function of the interface pressure.

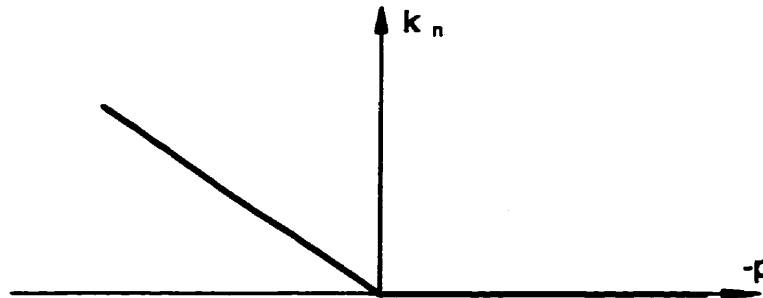


Figure 5.6 *Modified Interface Constitutive Law*

Substituting equation (5.28) into equation (5.25), we get

$$\Delta V = -\frac{h}{\lambda} \quad (5.29)$$

in the contact zone. The penetration displacement in the contact zone now is made not explicitly related with the interface stress, and its level is controlled by the constant λ . In

comparison with equation (5.27), to keep the penetration displacement smaller than V_{pc} , λ only needs to be

$$\lambda \propto \frac{1}{V_{pc}} \quad (5.30)$$

rather than to be proportional to the inverse of V_{pc}^2 . Thereafter, to choose a value for λ is much easier than to choose a value for M_p . The intensity of the internal work done on the interface can be estimated as

$$W_{int} = \frac{hp}{2\lambda} \quad (5.31)$$

By integrating above equation along the interface, we obtain:

$$w = \int W_{int} ds = \frac{1}{2\lambda} \int hp ds \leq \frac{f_L}{2\lambda} \max(h) \quad (5.32)$$

where w is the total internal work done by the interfacial normal stress on the penetration displacement, f_L is the normal resultant of the interfacial stress. It is easy to find the value of f_L from the global equilibrium conditions of either body A or B . In practical computations, the internal work done in the interface should be made as small as possible, but can not be made zero. Equation (5.32) shows that the internal work can also be well controlled by the constant λ .

5.3 Model Test

The numerical processes described above have been tested by the following example. There are two elastic blocks of $20m \times 4m$ placed up on the ground, as shown in Figure 5.7. Suppose the blocks are under the action of gravity and distributed forces.

In case 1: the upward forces intend to lift the corners of the upper block. A single segment contact zone will form.

In case 2: the force is put on the middle of the upper block. The separation will happen in the middle of the contact line, therefore, the contact zone consists of two segments.

Every block is meshed by 8-node plane strain elements with 20 elements in length and 4 elements in height directions, respectively. Twenty interface elements are laid out along the contact line.

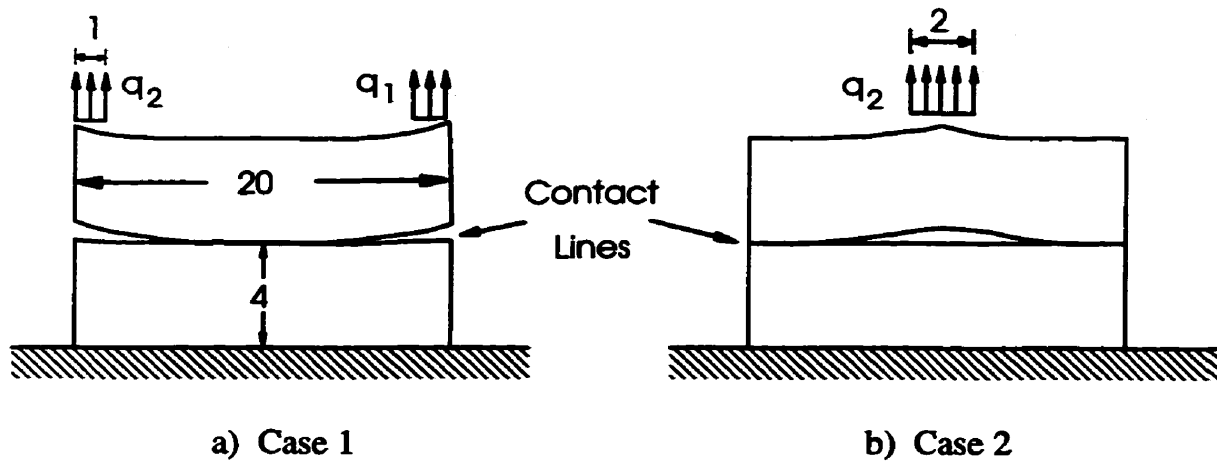


Figure 5.7 Model Test Problem

The parameters are listed in Table 5.1

Material Properties Used in Model Test		Table 5.1	
E	10 Gpa	λ	10^6
ρ	2000. kg/m ³	$k_{n\sigma}$	1.0 GPa
ν	0.3	q_1	300 kN/m ² ,
g	9.8 m/sec ²	q_2	500 kN/m ²

The magnitude of coefficient λ was chosen as 10^6 to keep the penetration displacement as small as $O(10^{-6})$. Too large values should be avoided, otherwise a stress oscillation will appear near the end of the contact zones.

For this problem, a tangential stiffness has to be introduced to eliminate the rigid horizontal motion of the upper block. A simple friction model

$$k_t = \beta k_n \quad (5.45)$$

has been used. The reduction coefficient β was taken as small as 10^{-4} to simulate a friction free contact surface.

In case 1, the deformation is unsymmetrical, the iteration converges after 10 step iterations. Seven steps are needed for the iteration to converge in case 2. The deformed meshes for the two cases are depicted in Figures 5.8 and 5.9, respectively. The displacement and the stress distributions are depicted in Figures 5.10 through 5.14. Very smooth interface displacement and stress distributions can be found. Figure 5.14 also shows that the shear stress is negligible with respect the normal stress level.

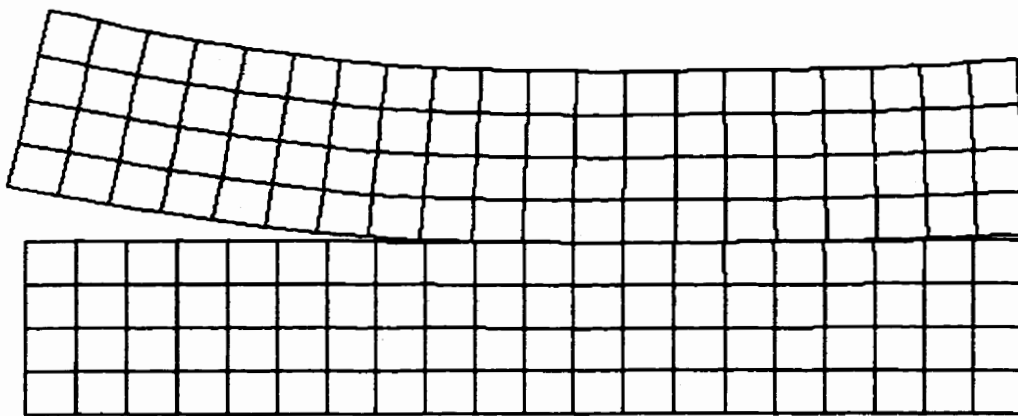


Figure 5.8 *Deformed Mesh for the Tested Model (Case 1)*

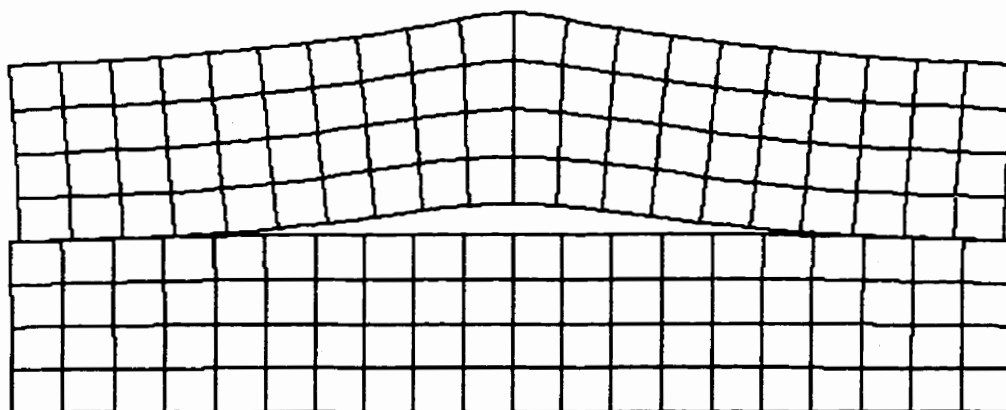


Figure 5.9 *Deformed Mesh of the Tested Model (Case 2)*

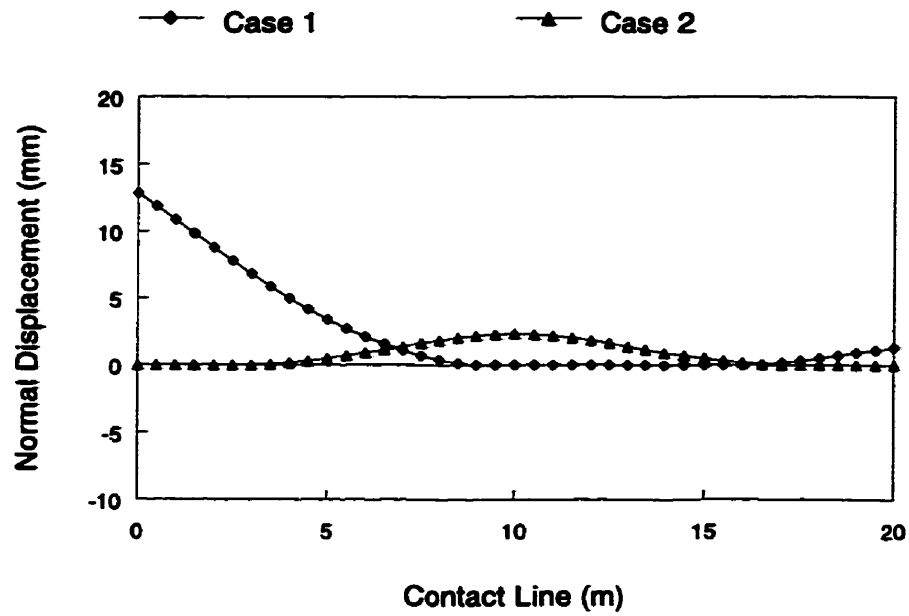


Figure 5.10 *Normal Displacements of Tested Model*

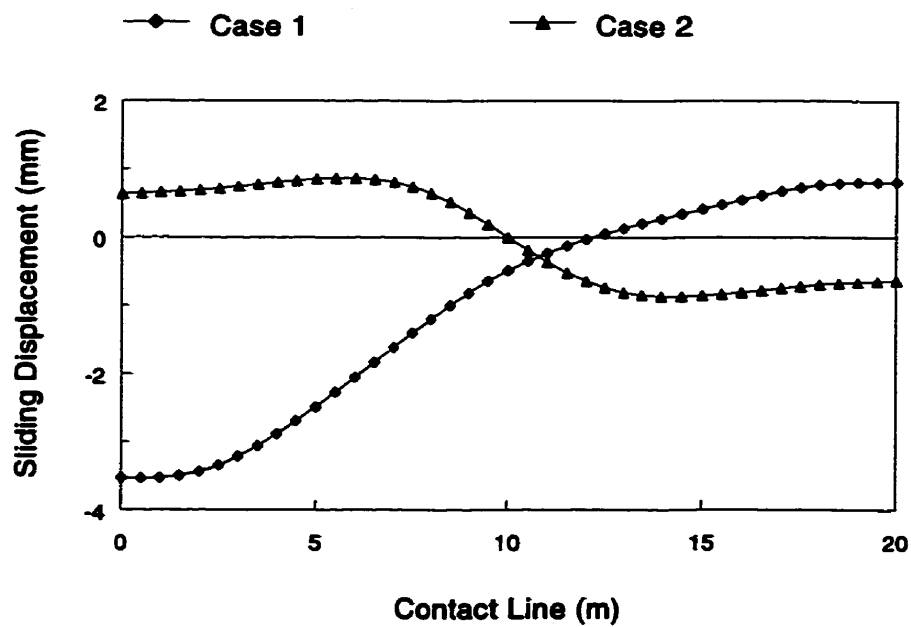


Figure 5.11 *Sliding Displacement of Tested Model*

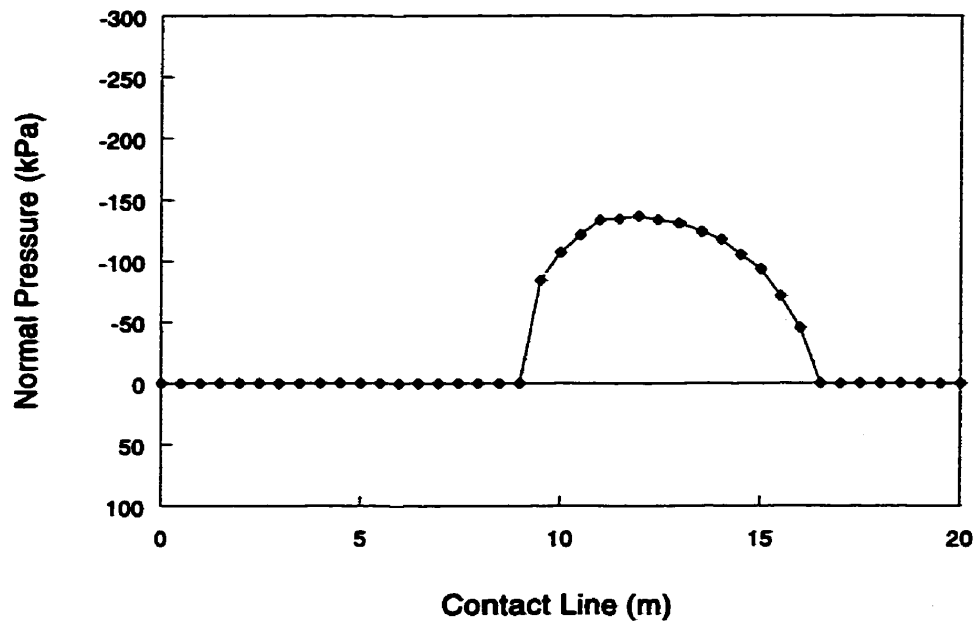


Figure 5.12 *Normal Pressure of Tested Model (Case 1)*

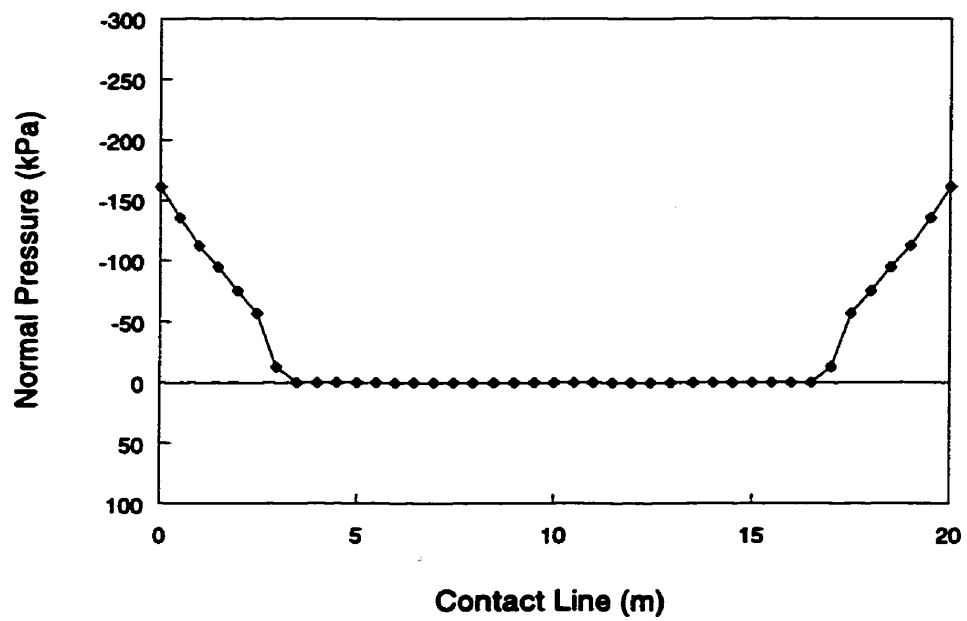


Figure 5.13 *Normal Pressure of Tested Model (Case 2)*

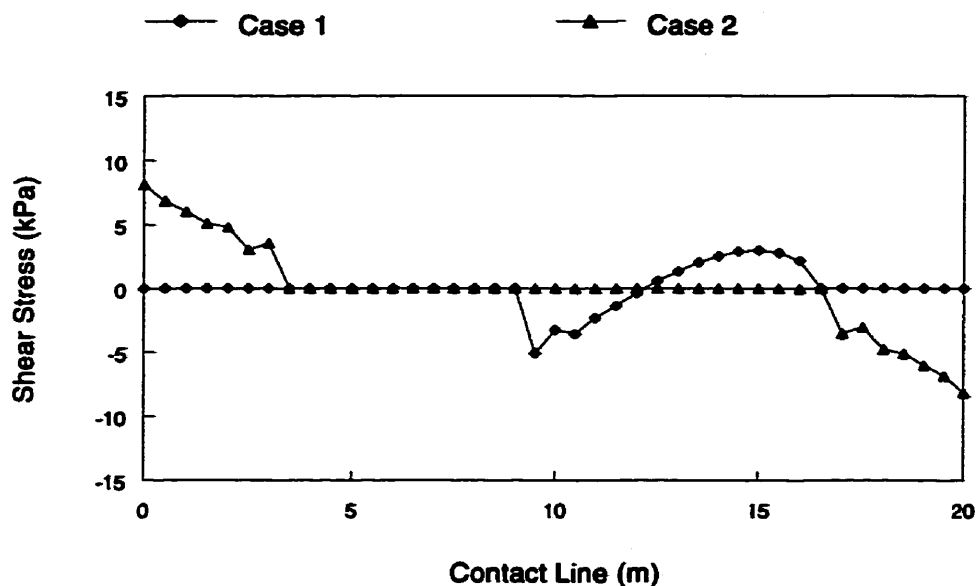


Figure 5.14 *Shear Stress of Tested Model (error order)*

5.4 Motion Analysis of the Leaking Joint in Long Spruce South Transition Structure

As a typical damaged dam structure, the motion of the Long Spruce South Transition structure is analyzed by the methods developed in previous sections. The cross section of the South Transition is shown schematically in Figure 3.2. Displacement instrumentations are installed along the leaking construction joint at the same locations of the thermistors as shown in Figure 5.15. Because of the debonded joint, the dam structure actually consists two blocks. The upper block rests on the top of the lower one. The joint opening displacements have been recorded hourly since October 1992 at six locations. The recorded joint opening displacements at two upstream locations (101U/S and 106U/S), dam center locations (102C/L and 104C/L) and downstream locations (103D/S and 104D/S) are depicted in Figures 5.16, 5.17 and 5.18, respectively.

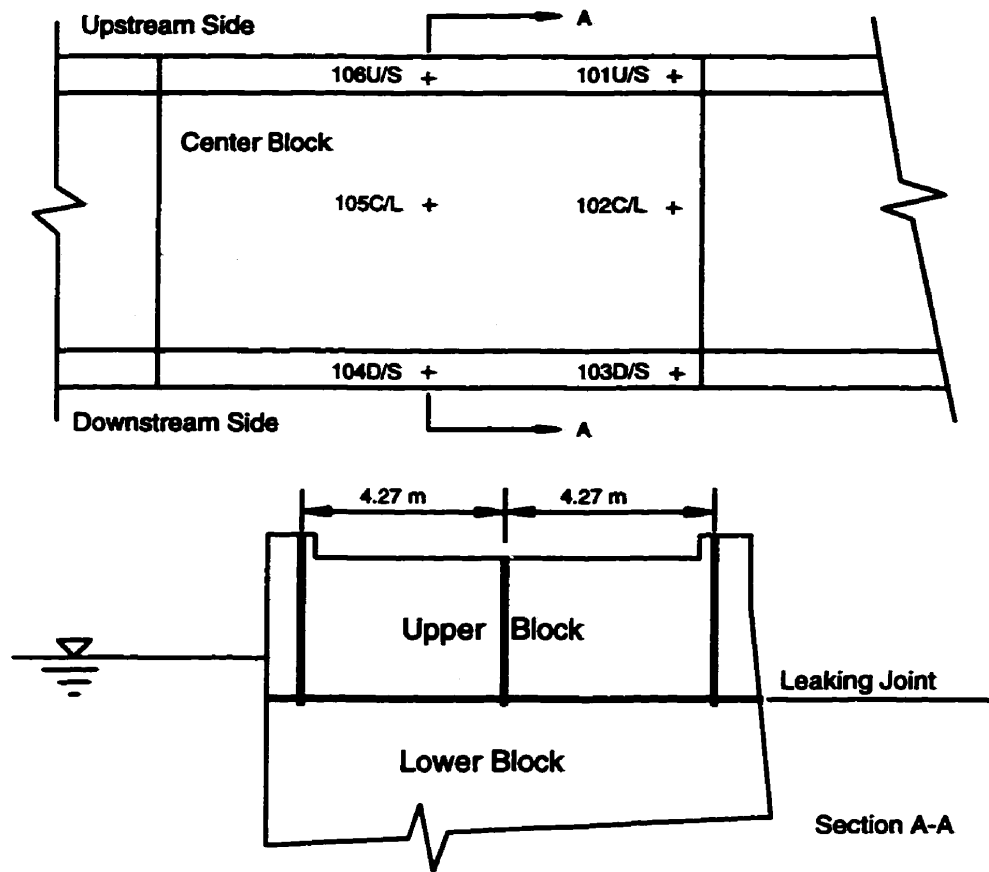


Figure 5.15 *Locations for Monitoring Instrumentation*

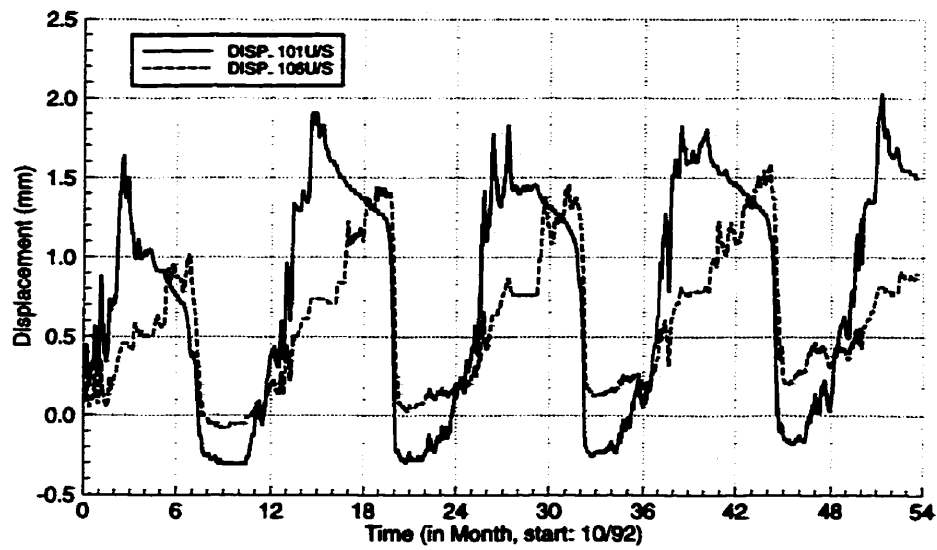


Figure 5.16 *Measurement of Joint Opening Displacement (upstream side)*

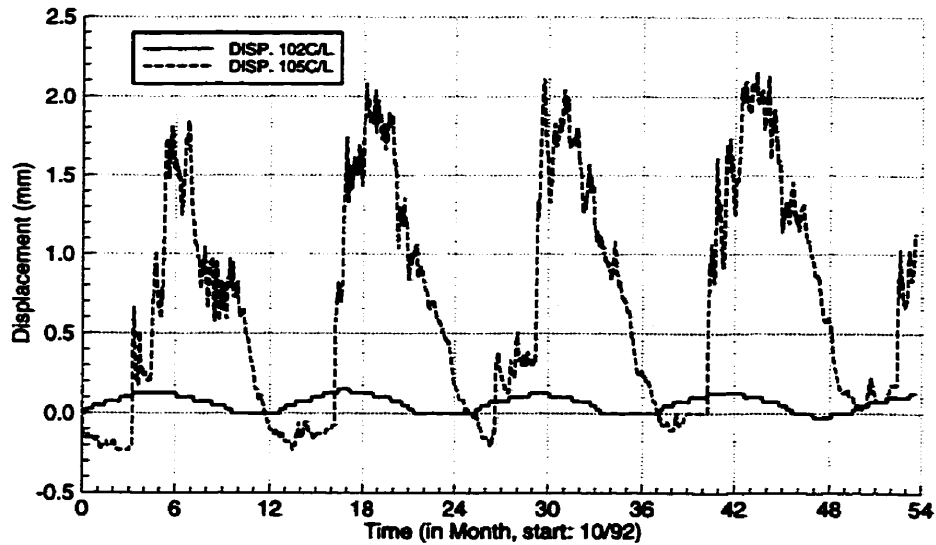


Figure 5.17 *Measurement of Joint Opening Displacement (dam center)*

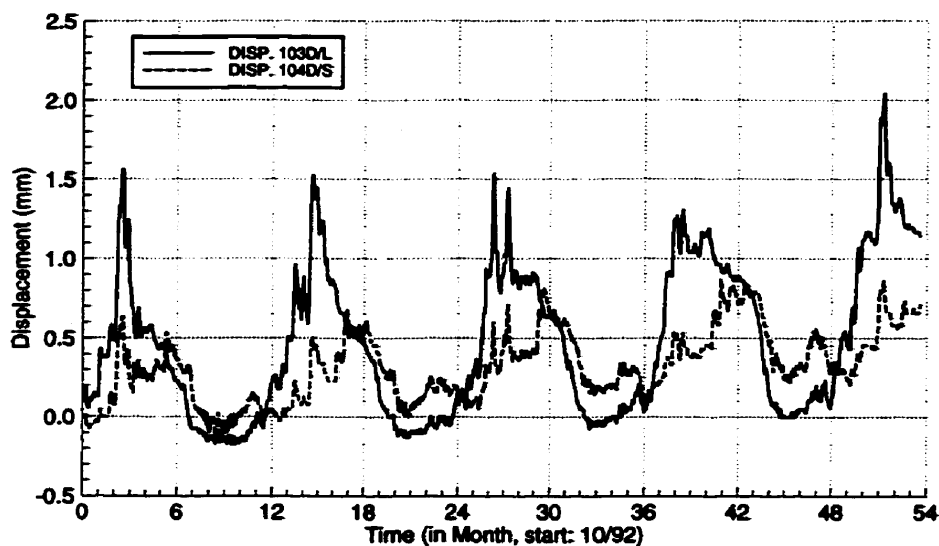


Figure 5.18 *Measurement of Joint Opening Displacement (downstream side)*

It can be observed from Figure 5.18 that the displacement peaks for the upstream side and downstream side appear almost simultaneously, every year in the winter. The peaks come about four months later in the dam center. This kind of displacement pattern indicates the curl-uncurl deformation of the dam structure. The displacement reaches its peak values quickly. When it goes down, there is a relative slow stage between two fast drop stages. The displacement sensor at 102C/L is out of order. No displacement readings come from its location. Therefore, the only displacement reading at the dam center comes from location 105C/L. It can also be seen that there are some negative readings. The reason for that is not quite clear. Probably installation errors are responsible for it.

Not similar to the situation of temperature, the measured displacements along line 101U/S-103D/S (near the side of the block) are different from those along line 106U/S-104D/S (in the center of the block). For the upstream side (U/S), the opening

displacements at corresponding locations 101U/S and 106U/S go up and down almost at the same time. The amplitude of 106U/S is generally smaller, as can be seen in Figure 5.16. Similarly, the amplitude of the reading of 104D/S (downstream side) is smaller than that of 103D/S (See Figure 5.17.) The recorded crack opening displacement data shows that the dam structure deforms three-dimensionally, even if its the temperature field is two dimensional. The temperature induced curl-uncurl deformation happens not only in the dam thickness direction, but also in the dam span direction. Generally speaking, a three-dimensional analysis has to be carried out to capture the motion history of this sort of structure.

Much work is needed to deal with this problem. However, dam structures are traditionally simplified by using a two-dimensional model (Leger et al. 1993). Plan strain theory is adopted to avoid the large amount of three-dimensional analysis. For present studies, we still use plan strain hypothesis. It is expected that the two dimensional analysis will yield reasonable approximation of the motion of the dam structure.

The same finite element mesh for the temperature analysis (See Figure 3.4) is used for the motion analysis, except that a layer of 21 interface elements is laid out along the leaking construction joint. The interface model given in section 5.2 was applied to this analysis. Because of the water pressure in the horizontal direction, a larger value was given to the coefficient β , as $\beta = 10^4$. The initial interface stiffness and the coefficient λ were chosen as:

$$k_{n0} = 100 \text{ GPa} \quad \text{and} \quad \lambda = 10^8$$

The material constants used for the analysis are listed Table 5.2

Material Properties of Long Spruce Dam Structure Table 5.2

E	ν	ρ	G	α
30. GPa	0.2	2294. Kg/m ³	9.8 m/sec ²	1.×10 ⁻⁵ 1/C

The static loads acting on the dam are due to gravity and the water pressure. The transient thermal analysis process described in Chapter 3 was made coupled with the motion analysis process. At every *12 hours*, the temperature field results obtained from the transient analysis were used to produce the thermal load for the motion analysis. In order to analyze the influence of the thermal loading and ice loading on the structure, these two loading cases are treated separately. Ice loading and its effect will be treated in the following chapters. For the time being, only the thermal loading is taken into account. The start time of the analysis is *October 1992*, and time span of the analysis is *54 months*. The results of the numerical analysis are given in the net section.

5.4.1 Displacements at Upstream Side Locations

The joint opening displacements at locations 101U/S (or 106U/S), predicted by the FEM is shown in Figure 5.19, with recorded displacements as comparison. For the first year, the finite element results can not coincide well with the recorded data, because the numerically predicted temperature field needs time to converge to the real situation. For the other years, better results are expected.

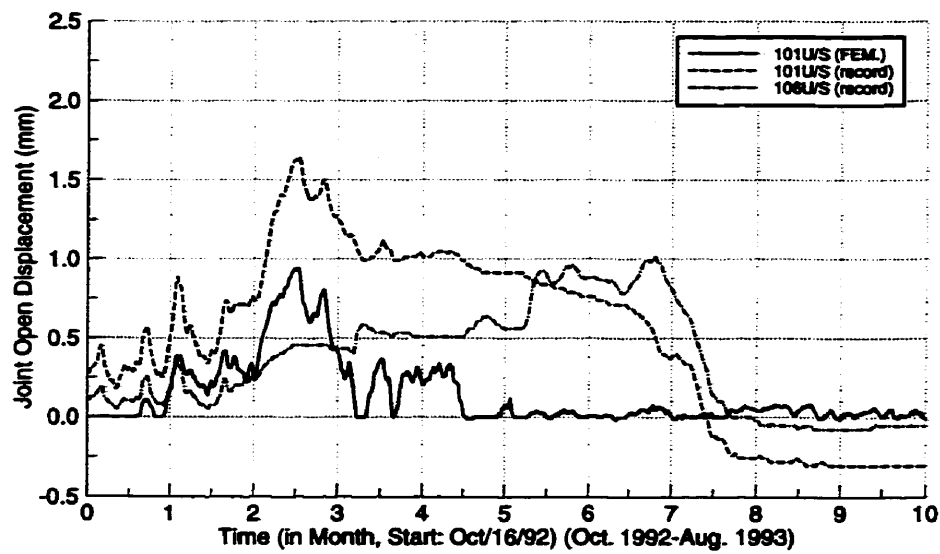


Figure 5.19 (a) *Joint Opening Displacement (upstream side)*

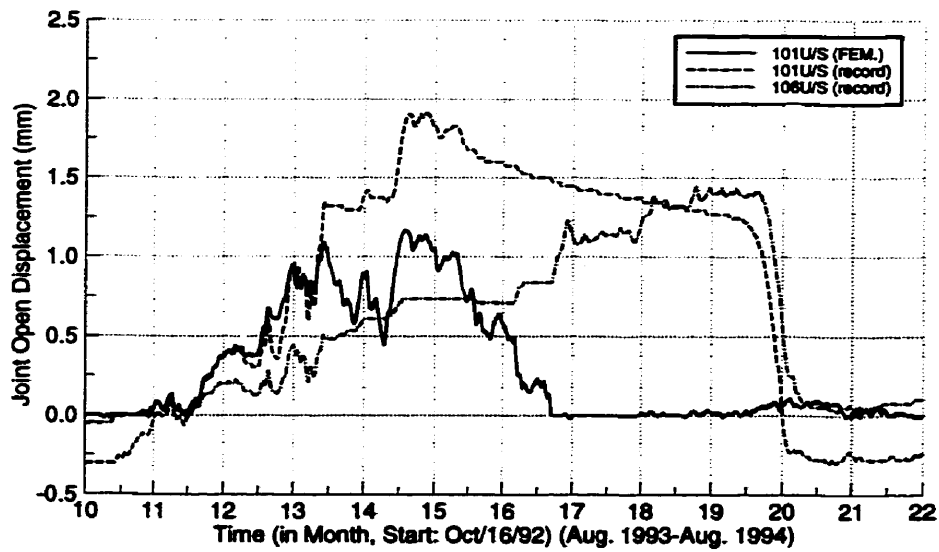


Figure 5.19 (b) *Joint Opening Displacement (upstream side)*

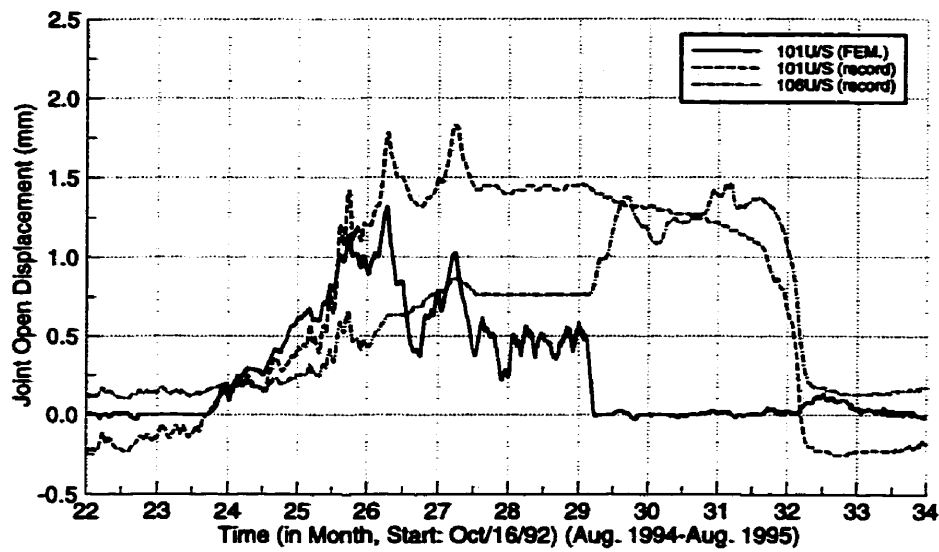


Figure 5.19 (c) *Joint Opening Displacement (upstream side)*

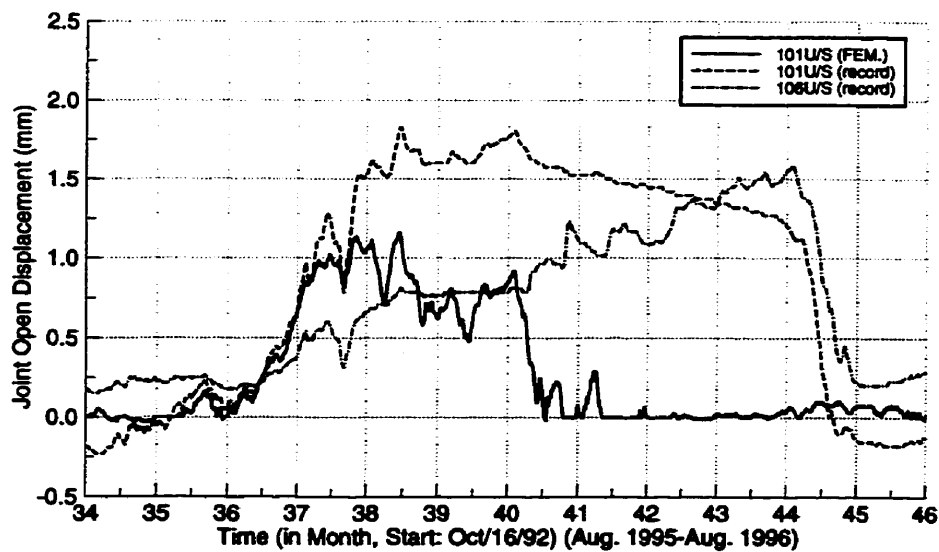


Figure 5.19 (d) *Joint Opening Displacement (upstream side)*

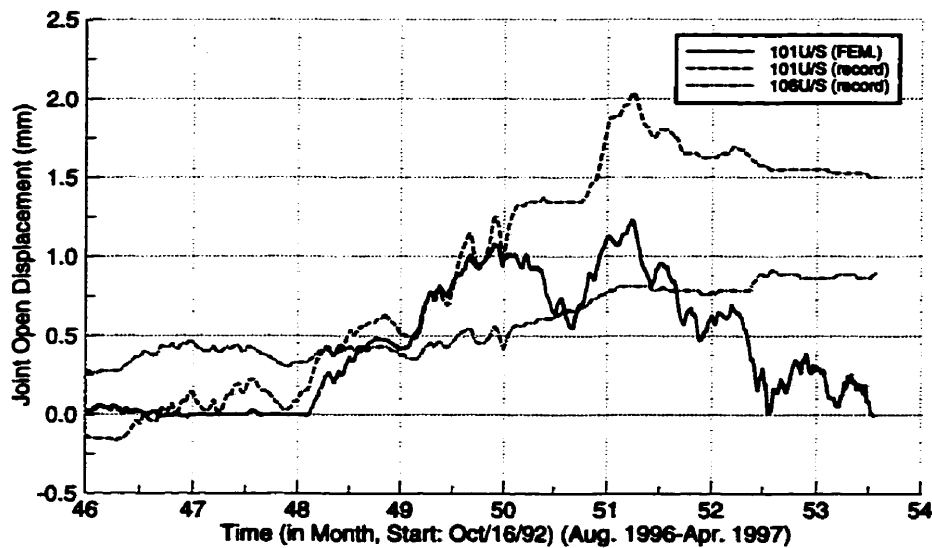


Figure 5.19 (e) *Joint Opening Displacement (upstream side)*

Figures 5.19(b) through 5.19(e) show that the displacements at 101U/S and 106U/S start to increase in each December. Around November and January, the displacement at location 101U/S reaches a stair. After the first stair of each year, there is quick jump to the highest peak around January or February. Then, the displacement begins to drop slowly until June or July. It is obviously that the slow drop of the displacement is caused by the jacking effect of the ice inside the cracked joint. In about June, the displacement has a sudden drop because of ice melting. There are a few more stairs in the curve of displacement at location 106U/S. The displacement does not go up and down so sharply as at location 101U/S. It has a sudden drop also in June, but a little bit later than at location 101U/S.

By the finite element analysis, the numerically predicted displacement curve starts to increase at the same time as that of record, i.e. in December. It follows the record

curve well until the end of November at location 101U/S. After that time, the finite element result still predicts the time of the peak displacement, but with much lower magnitude. Because no ice effect is included in the computational model, there is no stair in the displacement curve. The displacement drops to zero in March or April. It is noted that the finite element result does not coincide with the record at location 106U/S.

It can be seen that the displacement predicted by the finite element reaches about 50-60% of the recorded displacement peak value for every year. Ice accumulation inside the joint seems not to be responsible for this large discrepancy, because the gap water or ice fills must exist already. There must be another force to lift the upper block at the upstream side. The action of the ice sheet in the forebay is the only possibility for this force.

5.4.2 Displacements at Downstream Side Locations

The displacement curves of recorded data and finite element results for the downstream side are depicted in Figures 5.20(a) through 5.20(e). Similarly, the recorded displacements at location 103D/S and 104D/S start to increase in November. The displacement reaches its peak around January, i.e. at almost the same time for the upstream side. The ice inside the joint also makes some stairs in the displacement curve, but not so distinct as in the upstream side. The displacement drops to zero in June.

For this side, although the numerically predicted displacement curve also does not follow the record curve at location 104D/S, it coincides with the record at location 103D/S. The numerical curve can follow the record at location 103D/S even after its peak until February. This time the finite element result predicts a generally higher displacement magnitude than the recorded data. In considering the deformation pattern of

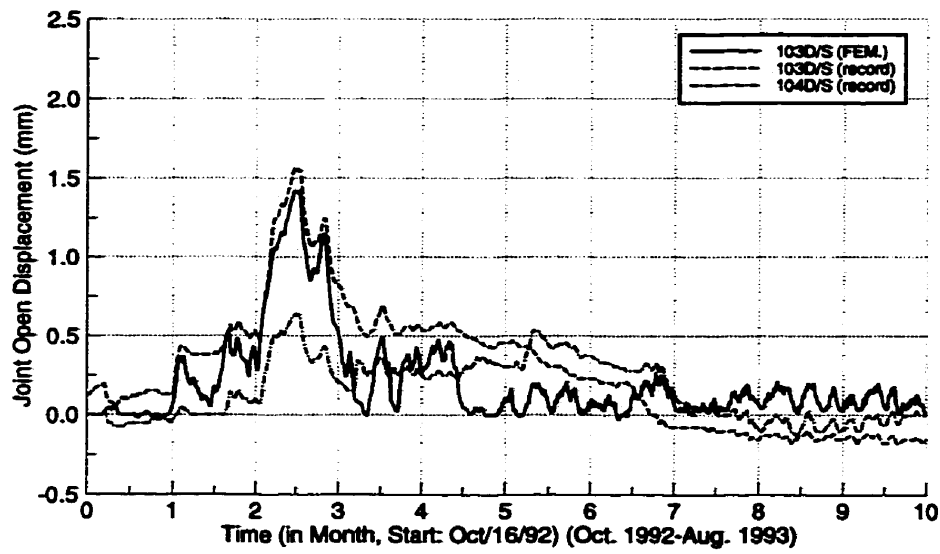


Figure 5.20 (a) Joint Opening Displacement (downstream side)

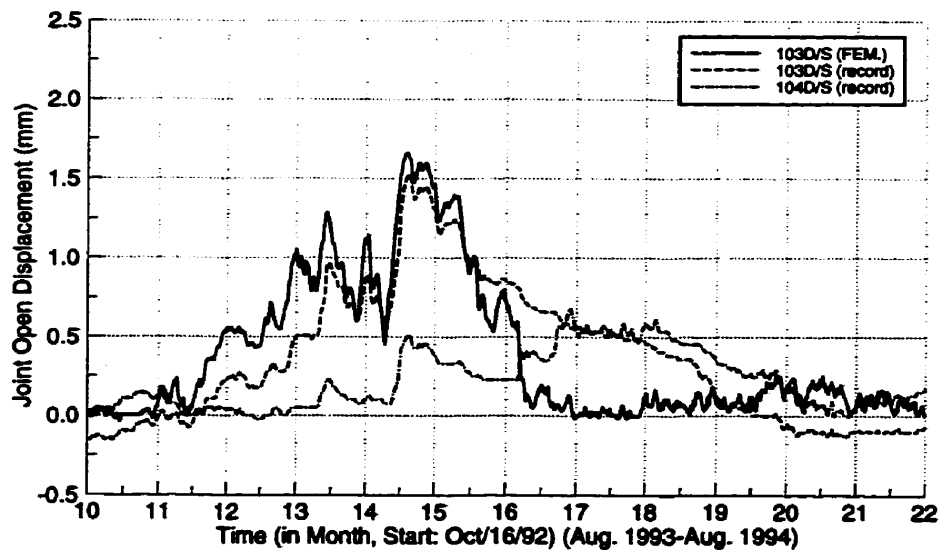


Figure 5.20 (b) Joint Opening Displacement (downstream side)

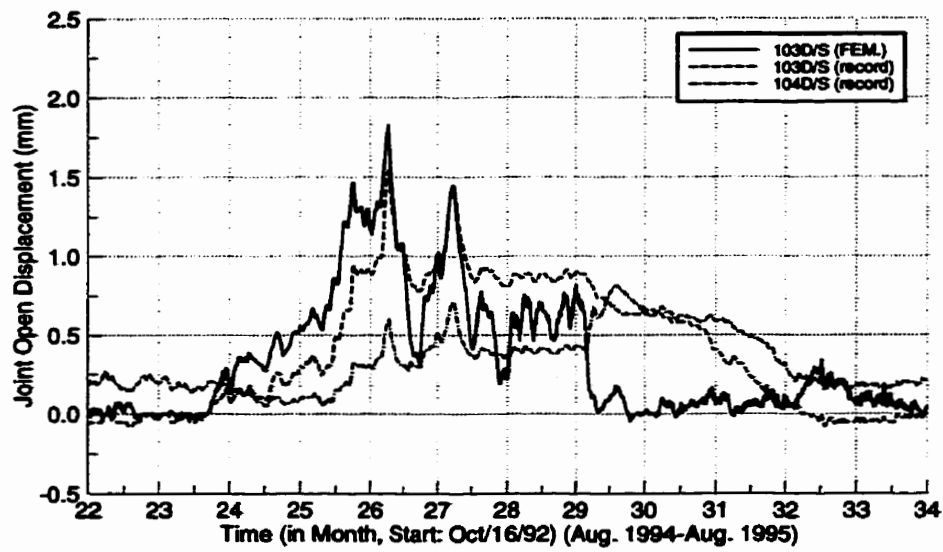


Figure 5.20 (c) Joint Opening Displacement (downstream side)

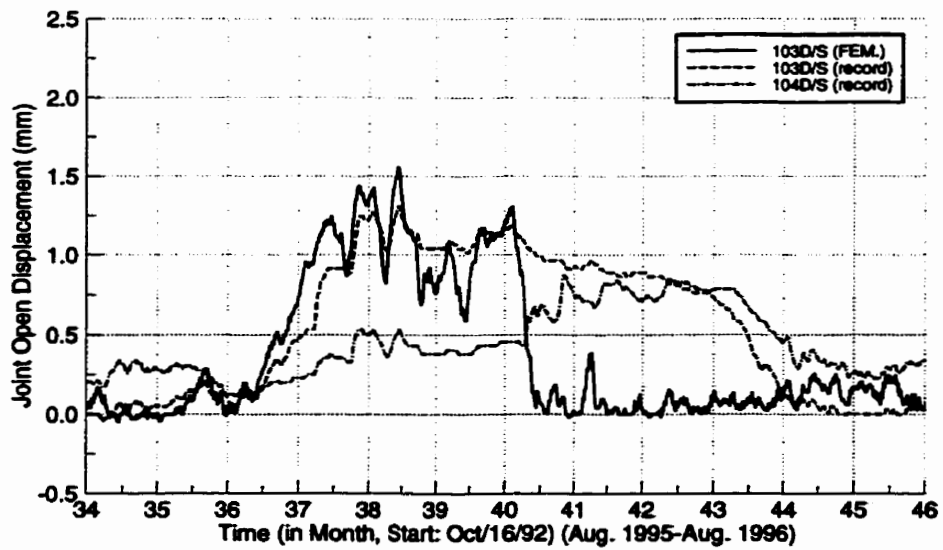


Figure 5.20 (d) Joint Opening Displacement (downstream side)

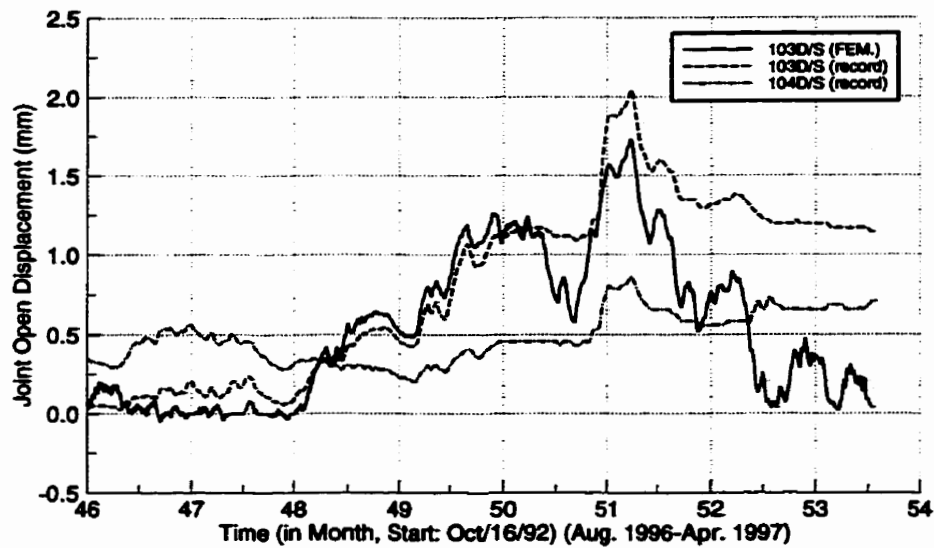


Figure 5.20 (e) *Joint Opening Displacement (downstream side)*

the upper block, we expect that once the ice force is introduced in the upstream side to increase the displacement at that end, the numerical result will become lower and fit the recorded data better.

5.4.3 Displacements in Dam Center Locations

The finite element results are compared with the recorded displacement at location 105C/L in Figure 5.21(a) through 5.21(d). According to the situation of the displacement at locations 101U/S and 103D/S, it is expected that the finite element results should coincide with the record at location 102C/L to some extent. Unfortunately, corresponding measured data is not available. Because location 105C/L is at the dam center, the displacement change is not so harsh as at other locations. The ice jack phenomenon is clearly shown in the displacement at location 105C/L. The displacement has a sudden jump between February and March. The gap created by the opening

displacements at the upstream and down stream end of the joint has reached its widest thickness and filled with ice at that time. Meanwhile, the upper block starts back to an uncurled deformation pattern. In the no ice model, the ice jacking effect is not considered. Therefore, the numerical result can not follow the recorded displacement at this location. In July and August, the finite element result coincides with the displacement, because the displacements at the upstream and downstream ends are almost zero (joint closed) and there is no ice jack effect during this period. The recorded displacement can be decomposed as two parts: thermal deformation and jacking displacement. The good agreement of the time for the displacement peaks to appear between the record and finite element curve reflects the rigid motion of the upper block of the dam structure.

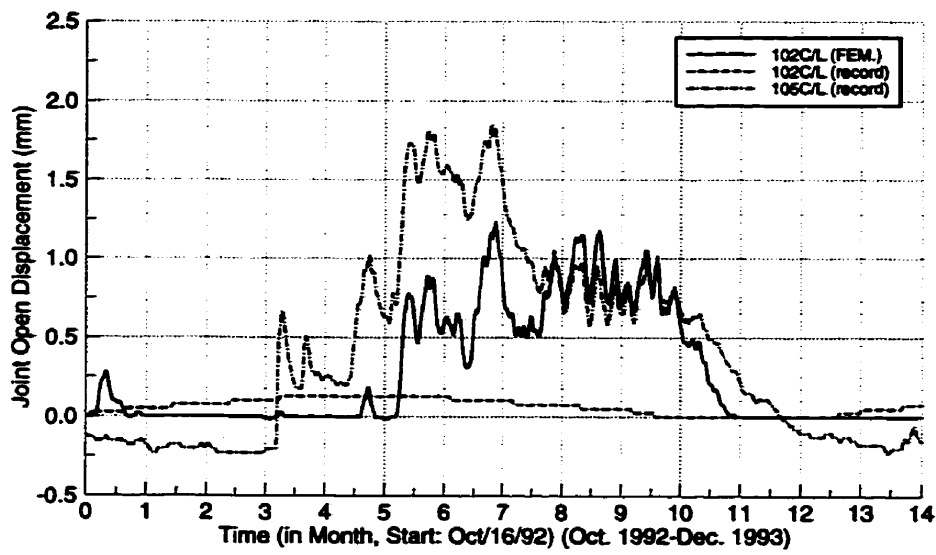


Figure 5.21 (a) *Joint Opening Displacement (dam center)*

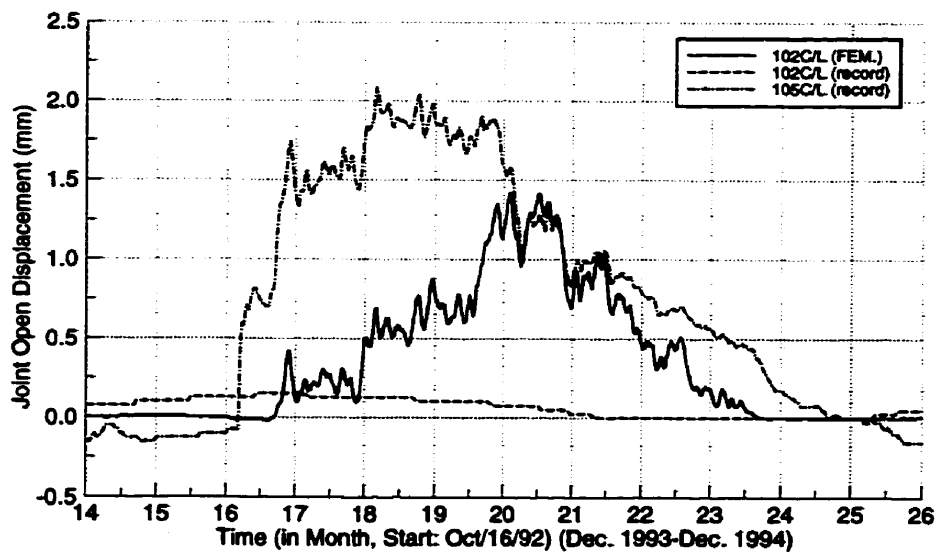


Figure 5.21 (b) *Joint Opening Displacement (dam center)*

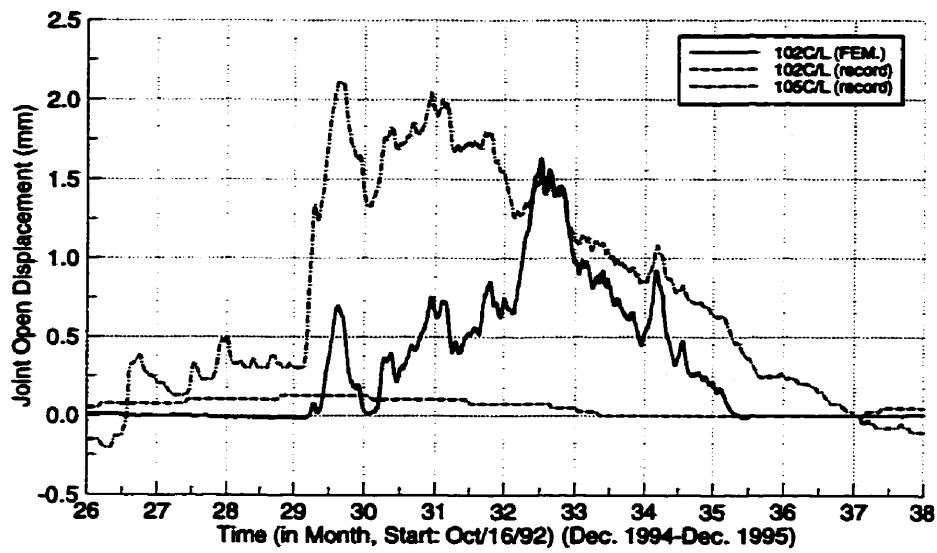


Figure 5.21 (c) *Joint Opening Displacement (dam center)*

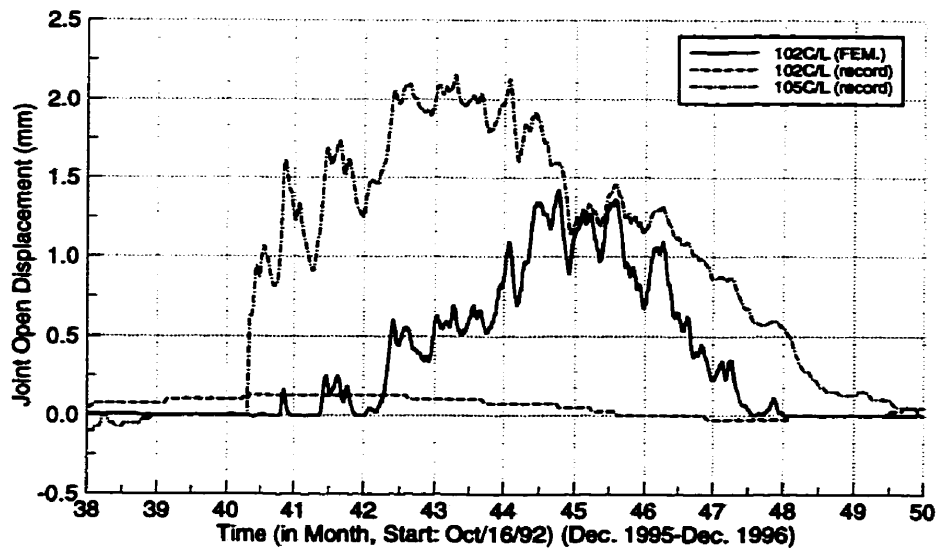


Figure 5.21 (d) *Joint Opening Displacement (dam center)*

5.5 Conclusions

Construction joints of dams are the weak parts of the structures. The joints in a high temperature gradient region are vulnerable to severe ambient temperature variations. During a long service time, the bond of a construction joint may be partially or totally broken. Debonded joints create cracks inside the dams and cause leakage. They are seriously harmful to the integrity and safety of the structure. Motion analysis of the dams with leaking joints is important for both safety assessment and rehabilitation. Because of the structural discontinuity incurred by the debonded joints or crack, nonlinearity has to be encountered in the motion analysis. An efficient algorithm must be developed to follow the long time motion history of the dam structure.

In this chapter, attention is focused on the motion analysis of the dam with through-thickness debonded joints. The main difficulty for this sort of problem is eventually the contact between two joint surfaces of the joints. Interface elements are adopted to deal with this problem numerically. The finite element formulation of an interface element is presented. The iteration process and the interfacial constitutive relations are specifically designed for this interface element. The evaluation of the interface stresses plays a crucial role in this analysis. The stress values are needed at the nodal point to determine the opening/closing state of the interface. The stress evaluation method developed in Chapter 3 finds its very application in this new interface element. Model test shows that very smooth stress and displacement distributions can be found by using the new interface element.

The motion analysis of dams with cracks involves some complicated interface properties. In the study of this chapter, only a simple interface configuration and

constitutive relations have been used. The algorithm developed in this chapter serves as a good base for the thermal induced motion analysis of cracked dam structures.

As a typical dam structure with failure, the thermal induced motion of Long Spruce South Transition Structure has been analyzed with the newly developed computational method. In order to study the thermal induced motion and the effect of ice on the structure separately, a model without any consideration of ice effect is adopted in the numerical analysis. Although the field measurement of joint opening displacement shows some three dimensional deformation pattern, a two dimensional plan strain model can still predict the displacement variation to some. It can be seen from the numerical results that the no ice model performs well in the time span when there is no ice.

A comparison between the displacement predicted by the present model and the recorded data shows the significant influence of ice on the motion of dam structure. It can be estimated that 50-60% the displacement peak value comes from the contribution of the thermally induced displacement. The after peak behavior of the structure is mainly determined by the ice's influence. Therefore, the study of ice loading and influence is the next important topic.

Chapter 6

Ice Loading on Dam Structures in Northern Region

6.1 Introduction

In the previous chapter, the computational method for the thermal induced motion analysis of dam structures with cracked construction joint has been developed. The thermally induced deformation of the Long Spruce South Transition has been analyzed without an ice load. Numerical results show that the finite element displacement curves are able to follow the measured displacement curves in certain time intervals, which cover the warm seasons. A large discrepancy starts between the finite element prediction and the measured opening displacement around the month of December when freezing occurs until the time when the ice melts. The peak displacement values predicted by the finite element analysis appear almost simultaneously as the recorded displacement peaks appear, but the amplitudes are much lower during this period. The record also shows that the crack's opening displacement maintains a high value and slowly drops down after the displacement peak, while the numerical analysis predicts a sharp closing movement for the crack. This happens in both the upstream and downstream sides. The appearance of ice much amplifies the amplitude of the displacements. The discrepancies demonstrate the effect of ice on the movement of the structure.

Ice can affect the displacements of the structure in two different ways. First, water inside the crack turns to ice when the ambient air temperature is lower than zero. The ice layer forms between the crack surfaces and its thickness is the same size as the crack opening. Ice can stand a certain amount of tensile stress and a much higher compressive

stress. For the ice layer inside the crack, it can bond the crack surfaces when the opening driving force is lower than the tensile strength. This phenomenon can be seen from the displacement records. Several short plateaus are followed by sharp jumps in the displacement curves. The jumps indicate the breaking of the ice bond. The jacking effect of the ice is responsible for the post-peak slow drop of the opening displacement when the ice layer is under pressure. The thickness of the ice layer inside the crack depends on the opening of the crack surfaces which changes slowly. It is unlikely that the large discrepancy between the peak of the numerical predictions and the record curve is caused by the inside ice layer. Therefore, we must turn to the second ice effect on the structure i.e., the ice loading from the forebay ice sheet.

In winter, an ice sheet forms on the surface of the forebay. This ice sheet freezes and attaches to the dam structure's upstream side. It can be 2 *meter* thick and can move up and down when the water level changes. It can also be dragged by water waves and wind to move horizontally. Strong forces are exerted on the structure by the ice sheet. The ice loading is believed to have a significant influence on the structural degradation of the dam. In this chapter, a study of the ice loading mechanism is performed. Models are proposed for the estimation of the ice sheet thickness and an analysis of the breakings of an ice sheet. The ice loading forces on the structure are the main variables that need to be identified.

Several models have been tried to study the ice sheet. Recorded data are used to calibrate these models. The vertical and horizontal loads caused by the ice sheet are set up as functions of the ambient air temperature. The dam's response to both the ice loading and thermal loading is analyzed next for the purpose of identifying the factors responsible for the dam structure's degradation and damage.

6.2 Forebay Ice Sheet Formation

The ice sheet forms in the forebay in winters, as sketched in Figure 6.1. We can consider it a half plane with one end on the dam body and the other end extending to infinity.

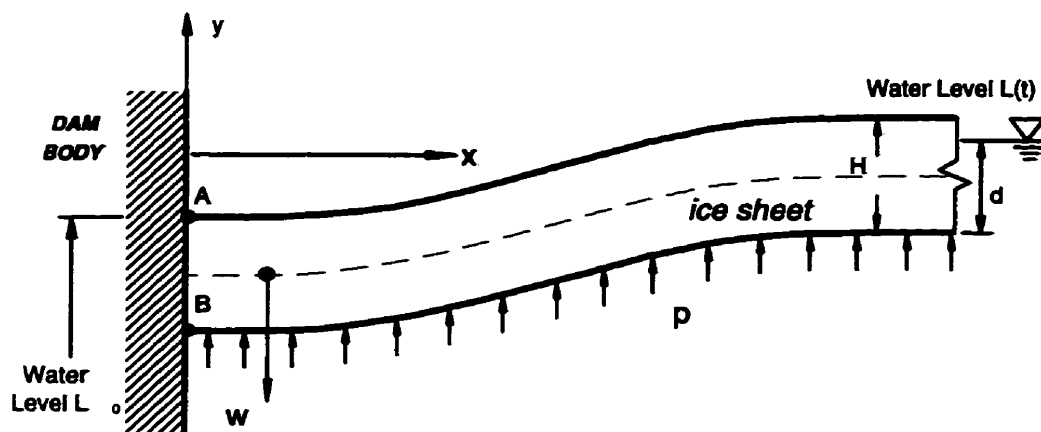


Figure 6.1 *Sketch of the Ice Sheet*

In the beginning of a freezing season, a very thin flat ice sheet appears and freezes itself to point A in Figure 6.1. The height of point A on the dam structure is the same as the initial water level L_0 at that time. In the lower side of the ice sheet, more and more ice grows and the ice thickness H increases with the ambient temperature dropping as a function of time, t . The ice sheet is supported by the water and the dam structure. A water level fluctuation, water flow and wind drag cause the ice sheet to deform elastically or plastically and even crack. Very complicated phenomena can be encountered with the behavior of the ice sheet.

For the purpose of identifying the ice loading, the formation process of the ice sheet need be analyzed first in terms of thickness growth. In the case where the yearly ice thickness record is not available, the thickness has to be predicted theoretically. Several researches were made in the study of the relation between ice formation and

environmental factors (See G.D. Ashton 1986 and T. M. Miles 1993.) The heat loss of water is the main factor that influences formation of the ice. Several mainly empirical relations have been suggested to calculate the heat loss in terms of the ambient air temperature, wind speed, solar radiation, cloud and snow cover, water flow speed, etc. Much meteorological data is needed to properly establish the ice freezing process.

Usually, only the ambient temperature is generally available. Therefore, the *Stefan equation* (see Ashton 1986) is widely accepted as a simple estimation of the ice sheet thickness in lakes in accordance with the ambient air temperature. In this model, water loses its heat to air through the ice-water interface. The heat exchange between the ice and water is neglected. It is also supposed that heat does not transfer in the direction parallel to the ice sheet. Therefore, the increment of the ice thickness dh satisfies:

$$\phi_i dt = \rho L dh \quad (6.1)$$

where h is the ice thickness; ϕ_i is heat flux from the ice to air; ρ is the density of the ice; L is the latent heat of ice fusion and t is time. The ice's temperature is assumed to vary linearly along its thickness. The end temperature of ice is taken to be the air temperature T_a and water temperature T_w , respectively. Actually, the water-ice interface temperature equals zero. From the Fourier's heat transform law, we have

$$\phi_i = k_i \frac{T_w - T_a}{h} \quad (6.2)$$

where k_i is the heat conductivity of ice. Substituting equation (6.2) into (6.1), we obtain:

$$h dh = \left(\frac{k_i}{\rho L} \right) (T_w - T_a) dt \quad (6.3)$$

Thereafter, we have:

$$h = \sqrt{\left(\frac{2k_i}{\rho L} \right) S} \quad (6.4)$$

where S is the number of *degree-days of freezing* defined as:

$$S = \int (T_w - T_a) dt \quad (6.5)$$

When appropriate values are inserted in equation (6.4) and the coefficient is adjusted empirically, the following formula is obtained:

$$h = 3.5 \times 10^{-2} \alpha S^{1/2} \quad (6.6)$$

where h is in meters and S is in degree-day. A correction factor α is included in the above formula to account for the influence of snowfall. The integration in formula (6.5) can be done numerically with the ambient air temperature record. In this way, the ice thickness is determined as a function of time.

The ice thickness of the Long Spruce forebay is set up as a function of time by means of formula (6.6) and the results are depicted in Figure 6.2. In the practical calculation, the degree-day number S is first set to zero. When the ambient air temperature is lower than zero degree, we start the integration (6.5). The accumulation of the degree-day number S reaches a peak value every year. When the ambient temperature goes up above zero degree, S starts to drop, which corresponds to the ice melting.

The ice melting process is even harder to simulate. Ice sheet breaks into pieces, and the pieces float away with the water. To estimate the date for the ice sheet breaking, a new degree-day number S_m is defined as

$$S_m = \int -T_a dt \quad (6.7)$$

when $T_a > 0$. Once the value of S_m reaches a certain level S_{train} , the accumulation of S stops, and the ice thickness is truncated to zero. Several numerical tests have been done to try to determine the value of S_{train} . It is found that S_{train} can be set in a range of

$$200 \text{ degree-day} < S_{train} < 400 \text{ degree-day} \quad (6.8)$$

In our calculation, S_{train} is chosen as 250 degree-day.

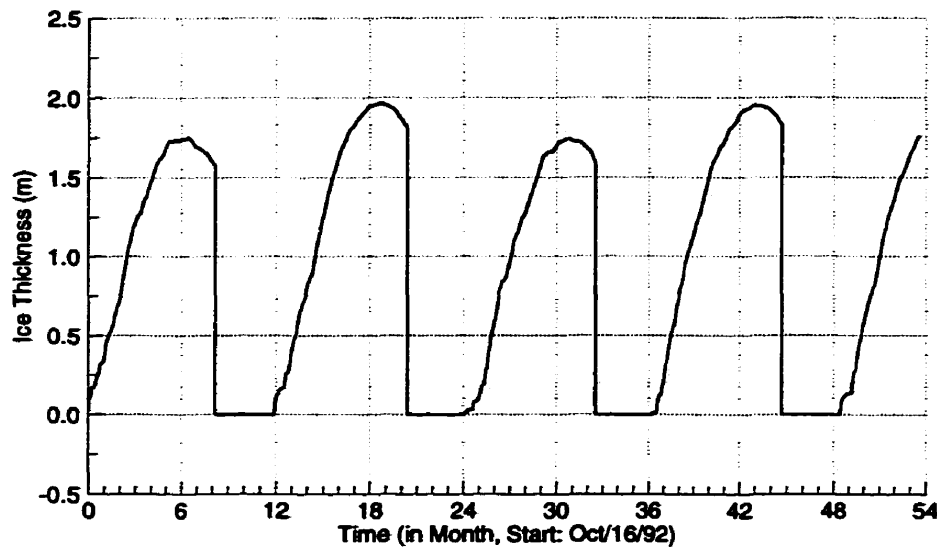


Figure 6.2 *Predicted Ice Sheet Thickness of the Long Spruce Forebay*

Field measurement of ice thickness was done for the Long Spruce forebay in March 27th, 1997. Two holes were drilled in the ice sheet. One was 1.5 feet away from the upstream surface, where the measured ice thickness was *1.4m*. The other was 21 feet away from the ice sheet root with a thickness of *1.88m*. By the freezing degree day method, the ice thickness was predicted as *1.85m* with the factor $\alpha=1$.

6.3 Ice Properties

The physical properties of ice need be addressed before we can analyze the ice sheet deformation. The strength under different loading patterns, Young's modulus and Poisson's ratio are most important to our analyses. Ice properties show a certain diversity; they strongly depend on the experimental conditions under which they are measured. Special experiments are necessary to yield proper data for various problems. General experimental results (from Ashton 1986) are listed below as a reference for further studies.

Mechanical Properties of Ice

Table 6.1

Property	Value range	Value used in this Study
Young's Modulus	8.35 - 9.48 GPa	9.0 GPa
Poisson's Ratio	0.314 - 0.35	0.35
Shear Strength	0.3 - 3.1 Mpa	0.7 MPa
Tensile Strength	1.8 - 2.2 Mpa	2.0 MPa
Compressive Strength	2 - 20 MPa	not used
Density	900 - 917 kg/m ³	900 kg/m ³

The strength of ice is sensitive to the loading speed and loading direction. For our problem, the loading speed is considered to be slow enough. Therefore, lower or middle values for the strength are acceptable. More information about ice properties can be found in (Ashton 1986).

6.4 Vertical Ice Sheet Load

6.4.1 Elastic Plate Model of the Ice Sheet

In the early winter season, the ice sheet is very thin and floats on the water surface. Once the thickness reaches a certain value H_0 , the ice sheet freezes in the dam surface at the current water level L_0 . The thickness of the ice sheet grows progressively underneath with the air temperature dropping. The ice sheet fluctuates with the water level variation. At points far from the dam structure, the ice sheet goes freely up and down with the water level. However, since the end of the ice sheet in the vertical upstream wall of the dam is not able to move freely, it can only move with the dam structure. Therefore, a vertical load as well as a bending moment is exerted on the upstream side of the dam.

The thickness is not the same over all the ice sheet. For simplicity, it is generally supposed that the ice sheet has a uniform thickness. At time t , the ice thickness is H and the deflection of its middle plane is w , as shown in Figure 6.1. Far from the dam, the deflection of the ice sheet is set to zero. The ice sheet can be simulated as an elastic plate and the deflection satisfies the following equation (Kerr 1985 and Christensen 1987):

$$D\nabla^4 w + p = 0 \quad (6.9)$$

where $D = \frac{EH^3}{12(1-\nu^2)}$, H is the plate thickness; E is *Young's* modulus of ice; ν is Poisson's

ratio of ice; p is the uplift distributed load on the ice sheet;

$$\nabla^2 = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial z^2},$$

z is the coordinate along the dam span direction. The relation between the ice sheet's deformation and the forces exerted on the dam body can be established. The uplift distributed load p , as a function of the deflection w , is

$$p = wg\rho_{\text{water}} \quad (6.10)$$

after the weight of the ice sheet and the buoyancy cancel each other. Substituting equation (6.10) into (6.9), and letting

$$4k^4 = \frac{\rho_{\text{water}}g}{D}$$

or

$$k = \left(\frac{3(1-\nu^2)Hg\rho_{\text{water}}}{E} \right)^{1/4} \frac{1}{H} \quad (6.11)$$

we obtain

$$\nabla^4 w + 4k^4 w = 0 \quad (6.12)$$

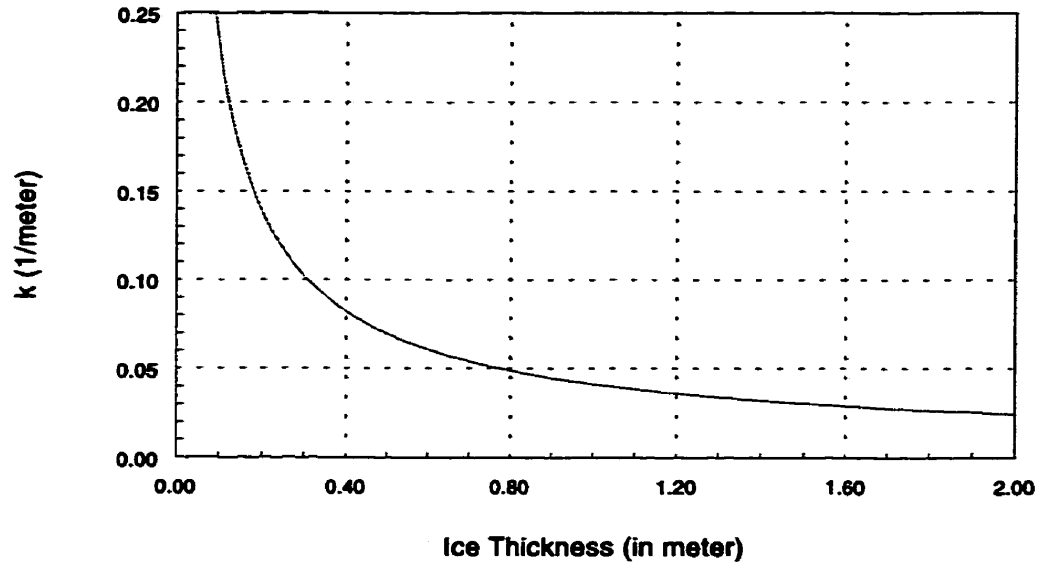


Figure 6.3 Variation of the Parameter k against Ice Thickness

The parameter k controls the variation of the stress state in the x direction. The value of k is depicted against the ice thickness in Figure 6.3. The material properties presented in Table 6.1 are used. It is noted that the response of parameter k to the variation of ice thickness becomes relatively slow after $H > 0.5$ m.

The deformation is considered to be homogeneous in the dam's span direction. Therefore,

$$\nabla^4 = \frac{d^4}{dx^4} \quad (6.13)$$

with the condition when

$$x \rightarrow \infty, w \rightarrow 0 \quad (6.14)$$

The closed form solution of equation (6.12) is reduced to

$$w = e^{-kx} (c_1 \sin(kx) + c_2 \cos(kx)) \quad (6.15)$$

where the arbitrary constants c_1 and c_2 are to be determined by the boundary conditions, i.e., the displacement of the dam. If the end deflection of the ice sheet is w_0 with respect

to the ice sheet at infinity and the angle between the tangential line across point A after deformation and vertical direction is $\Delta\theta$, as defined in Figure 6.4, then, the end displacement can be expressed as:

$$w(0) = w_0 \quad \text{and} \quad \left. \frac{dw}{dx} \right|_{x=0} = -\Delta\theta \quad (6.16)$$

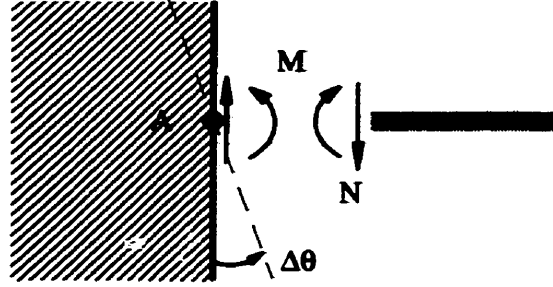


Figure 6.4 *Convention of Symbols at the End of the Ice Sheet*

With conditions (6.16), the arbitrary constants in solution (6.15) can be determined, and we obtain

$$w = \left[\left(w_0 - \frac{\Delta\theta}{k} \right) \sin(kx) + w_0 \cos(kx) \right] e^{-kx} \quad (6.17)$$

The vertical force and moment distribution in the x direction are derived from the deflection w by using:

$$M = -D \frac{d^2 w}{dx^2} = 2Dk^2 e^{-kx} \left[w_0 (\cos(kx) - \sin(kx)) - \frac{\Delta\theta}{k} \cos(kx) \right] \quad (6.18)$$

and

$$N = D \frac{d^3 w}{dx^3} = 2Dk^3 e^{-kx} \left[2w_0 \cos(kx) - \frac{\Delta\theta}{k} (\sin(kx) + \cos(kx)) \right] \quad (6.19)$$

Then, the end force and moment are related to the end displacement and rotation by means of equation (6.18) and (6.19) with $x = 0$, i.e.:

$$\begin{Bmatrix} M \\ N \end{Bmatrix} = D \begin{Bmatrix} 2k^2 \\ 4k^3 \end{Bmatrix} w_0 + D \begin{Bmatrix} -2k^2 \\ -2k^3 \end{Bmatrix} \left(\frac{\Delta\theta}{k} \right) \quad (6.20)$$

It should be noted that the force and moment in the above equation are acting on a unit length in the z (dam span) direction.

Equation (6.20) indicates that the vertical force and moment depend on the structure's deformation itself. Numerical tests show that the force and moment caused by the rotation $\Delta\theta$ are negligible with respect to the first term in equation (6.20). The ice sheet's end deflection w_0 eventually decides the load level. The deflection w_0 equals the height difference between the upper surface of the ice sheet at infinity and point A, which is

$$w_0 = \Delta L + H\left(1 - \frac{\rho_{ice}}{\rho_{water}}\right) - \Delta v_A \quad (6.21)$$

where ΔL is the relative water level, Δv_A is the difference between the current vertical displacement and the displacement when freezing occurs at point A. The relative water level for every year is defined as the difference between the current water level and the water level when freezing occurs, i.e.

$$\Delta L = L(t) - L_0 \quad (6.22)$$

In equation (6.21), Δv_A makes the load coupled with the structure's deformation. However, in most cases, this term is very small with respect to the first two terms and can be neglected. Therefore, we obtain

$$w_0 = L(t) - L_0 + H\left(1 - \frac{\rho_{ice}}{\rho_{water}}\right) \quad (6.23)$$

which is a function of time and independent of the deformation of the structure.

The yearly water level change recorded in the Long Spruce forebay is depicted in Figure 6.5. The water level is increased in winter in order to produce more power. Of course, this is the time when the ice cover forms in the forebay. The higher water level is maintained until the ice cover begins to melt.

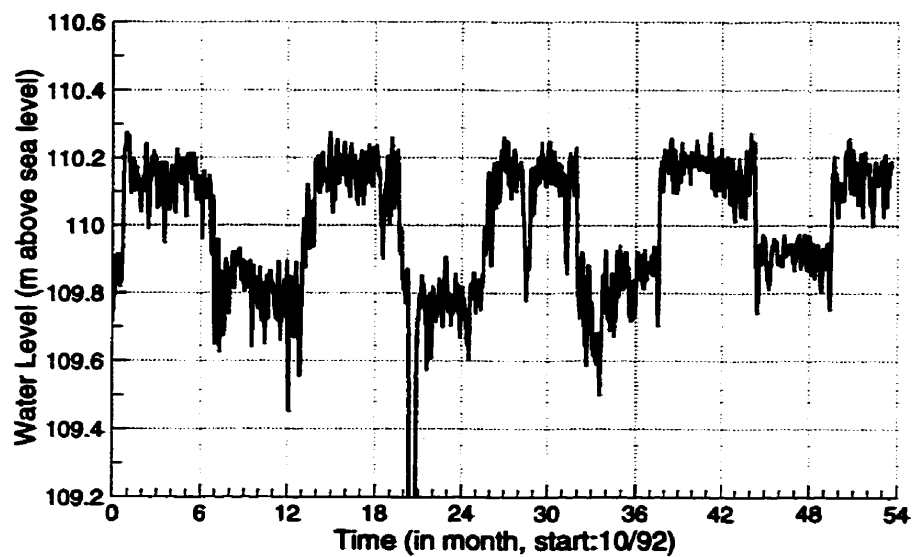


Figure 6.5 *Time History of Water Level of the Long Spruce Forebay*

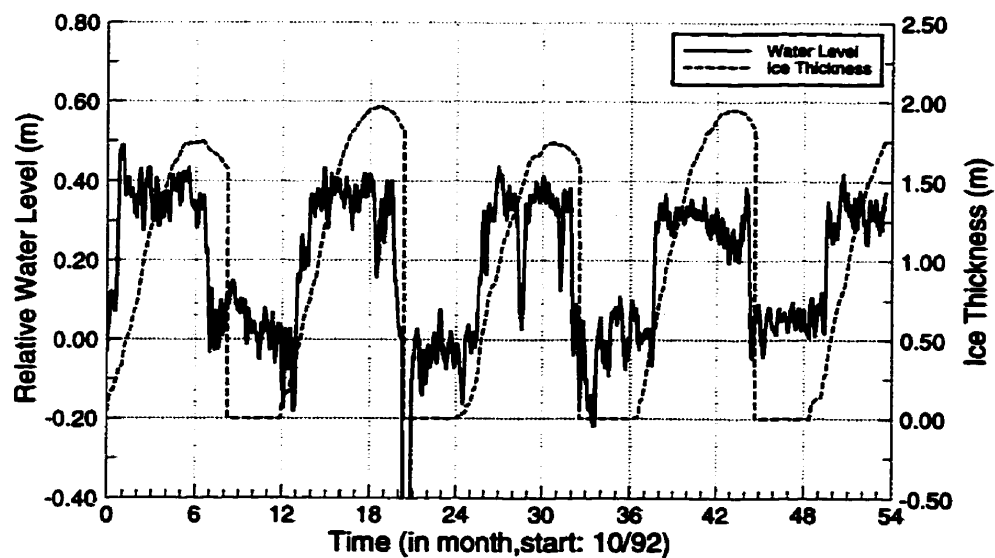


Figure 6.6 *Relative Water Level of Long Spruce Forebay*

To obtain the relative water level ΔL , the ice thickness data shown in Figure 6.2 and the water level data given in Figure 6.5 are analyzed. The water level at the time when the ice thickness is about 1 to 2 cm is chosen as the yearly initial water level. It is found that we can choose the level $L_0 = 109.8 \text{ m}$ for every year from 1992 to 1997. The relative water level determined in this way is shown in Figure 6.6. It can be seen that the water level increases just after the increment of ice sheet thickness, and goes down before the ice sheet breaks.

6.4.2 Cracking of the Ice Sheet

Equation (6.20) shows that the bending moment at the ice sheet root is:

$$M_1 = 2Dk^2 w_0 \quad (6.24)$$

and the maximum bending moment that the ice sheet can withstand is:

$$M_s = \frac{\sigma_t H^2}{6} \quad (6.25)$$

where σ_t is the tensile strength of ice. In the early stage of ice sheet formation, this limit is quite easy to reach and to be exceeded by the bending moment M_1 at the root, because of the water level fluctuation. The bending moment causes the ice sheet to break. Therefore, a hinge appears at position I in the ice sheet, as shown in Figure 6.7

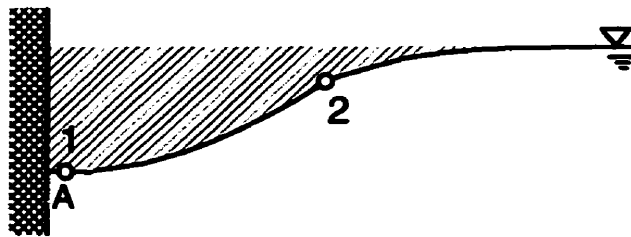


Figure 6.7 Crack Position in Ice Sheet

This hinge releases the end bending moment and much of the stress inside the ice sheet.

In this case, the ice deflection solution becomes:

$$w = w_0 e^{-kx} \cos(kx) \quad (6.26)$$

Thereafter, the vertical shear force and bending moment in the ice sheet become:

$$N = 2Dk^3 w_0 (\cos(kx) - \sin(kx)) e^{-kx} \quad (6.27)$$

and

$$M = -2Dk^2 e^{-kx} \sin(kx) \cdot w_0, \quad (6.28)$$

respectively. The vertical force then drops to half the load level as in the case without the root crack. The position which corresponds to $N=0$ can be easily found to be:

$$x = \frac{\pi}{4k} \quad (6.29)$$

This position is depicted as point 2 in Figure 6.7. At this position, the magnitude of the moment in equation (6.28) achieves its highest value:

$$M_2 = \sqrt{2} e^{-\frac{\pi}{4}} Dk^2 \cdot w_0 = 0.64 Dk^2 \cdot w_0 \quad (6.30)$$

It is found from Figure 6.8 that if the tensile strength of ice σ_t is between 2.0 ~ 2.2 MPa, the value of M_2 is quite close to the limit moment value M_j . This indicates that the ice sheet is on the verge of cracking for a second time.

Because of water flooding of the crack, this amount of bending moment maintains in the ice sheet. Therefore, a vertical load of

$$N = 2Dk^3 \cdot w_0 \quad (6.31)$$

practically acts on the upstream surface of the dam structure. The vertical load is depicted in Figure 6.9 as a function of time.

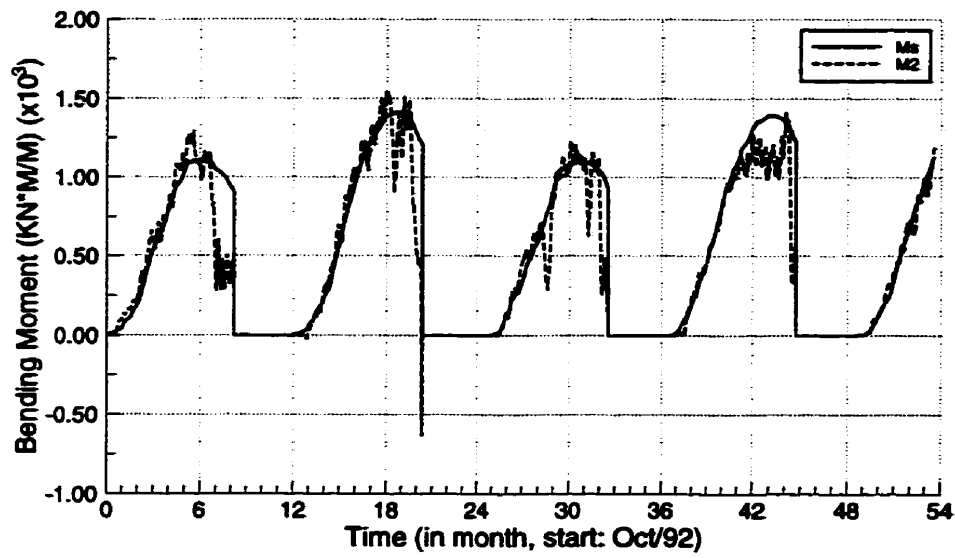


Figure 6.8 Moment M_2 versus M_s

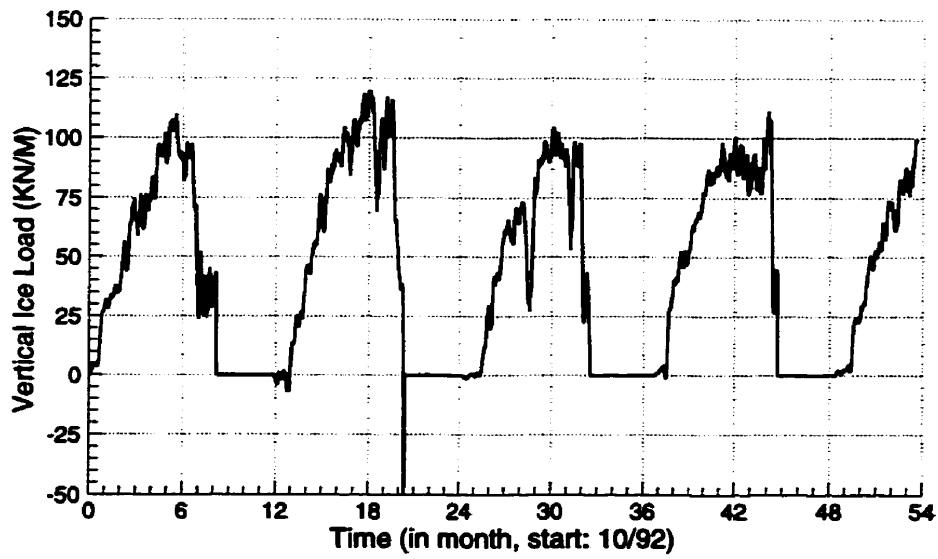


Figure 6.9 Vertical Load on the Dam Structure

From Figure 6.9, it can be seen that the peak value of the vertical load is about 100 KN/m . In practically numerical analysis, it is observed that this load significantly changes the opening displacement of the construction joint during the period when the load is lower than 75 KN/m . The time when the load reaches its peak corresponds to the time for the joint to close.

6.5 Horizontal Ice Sheet Load

The horizontal force caused by ice is usually considered as one of the major loads acting on dams in cold regions. The horizontal force is mainly produced by thermal expansion in the ice sheet and by wind drag. Furthermore, the thrust exerted by expanding ice depends on the thickness of the sheet, the rate of temperature rise in the ice, fluctuations in the water surface, the character of the reservoir shores etc. The rate of temperature rise in the ice is a function of the rate of the air temperature rise and the amount of snow cover on the ice. Lateral restraint of the ice sheet depends on the character of the forebay shores. For the present studies, a one-dimensional model is used to evaluate the ice temperature in terms of the ambient air temperature, and the ice stresses are subsequently accounted by a rheological model.

6.5.1 Ice Temperature Analysis

Many field measurements have been conducted for the temperature distribution inside an ice sheet. It is found that the ice temperature is strongly environmental dependent. The data measured in one place can't be used for another place. The ice temperature is seldom available to estimate the ice load. In view of the fact that an air temperature variation is the main cause of an ice temperature change, a one dimensional

heat transfer process inside the ice can be solved to find the dependence of the ice temperature on the air temperature.

In winter, once the ice sheet is of a certain thickness, snow accumulates on its top to form a snow layer. This snow layer insulates the ice from air even if it is quite thin. For the ice temperature analysis, heat transfer in both ice and snow cover need to be considered. The boundary conditions are shown in Figure 6.10.

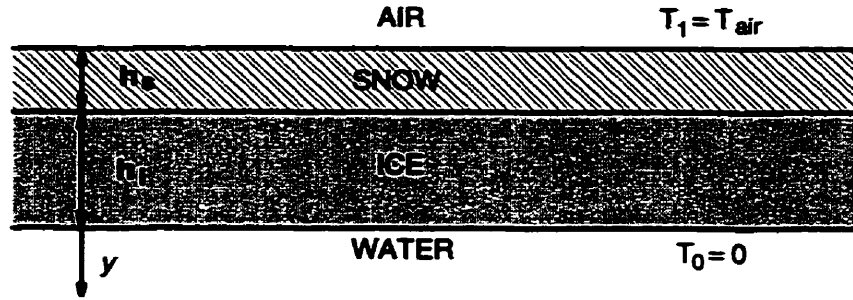


Figure 6.10 Snow and Ice Layer

On the water-ice interface (the bottom of the ice layer), the ice temperature is always set to 0°C . On the top, the snow temperature is set to the air temperature. Heat transfer processes inside the snow and ice satisfy:

$$\left(\frac{\rho c}{k}\right)_s \frac{\partial T}{\partial t} - \frac{\partial^2 T}{\partial y^2} = 0 \quad (6.32a)$$

and

$$\left(\frac{\rho c}{k}\right)_i \frac{\partial T}{\partial t} - \frac{\partial^2 T}{\partial y^2} = 0 \quad (6.32b)$$

where ρ is the density, c is the specific heat and k is the heat conductivity and index s and i stand for the snow and ice, respectively. Considering that the thickness of ice and snow is variable, the x -coordinate can be normalized by using

$$\xi = \frac{y}{h_s} \quad \text{and} \quad \eta = \frac{y}{h_i} \quad (6.33)$$

respectively. Therefore, equation (6.32a) and (6.32b) are normalized as:

$$\left(\frac{\rho c h_s^2}{k} \right)_s \frac{\partial T}{\partial t} - \frac{\partial^2 T}{\partial \xi^2} = 0 \quad 0 \leq \xi \leq 1 \quad (6.34a)$$

and

$$\left(\frac{\rho c h_i^2}{k} \right)_i \frac{\partial T}{\partial t} - \frac{\partial^2 T}{\partial \eta^2} = 0 \quad 0 \leq \eta \leq 1 \quad (6.34b)$$

Equation (6.34a) and (6.34b) are solved by applying finite difference method with the air and water boundary conditions and the conductive condition on the ice-snow interface.

The ice thickness h_i has already been set as a function of time in section 6.2 by the freezing degree day method. Meteorological records are needed for the snow thickness. At the Long Spruce Generation Station, the snow record from September 1996 to February 1997 is listed in Table 6.2.

Numerical tests have been done to analyze the influence of snow thickness variation. It is found that once there is a snow cover which is thicker than 20 cm, different snow thickness causes little change in the ice temperature. Therefore, a snow thickness template can be set for all the years according to the data presented in Table 6.2 and shown in Figure 6.11

Snow Record from September 1996 to February 1997

in Long Spruce Station

Table 6.2

Mon/Yr	Sept./96	Oct./96	Nov./96	Dec./96	Jan./97	Feb./97
Snow Thickness	0.0 cm	52 cm	57 cm	51 cm	58 cm	72 cm

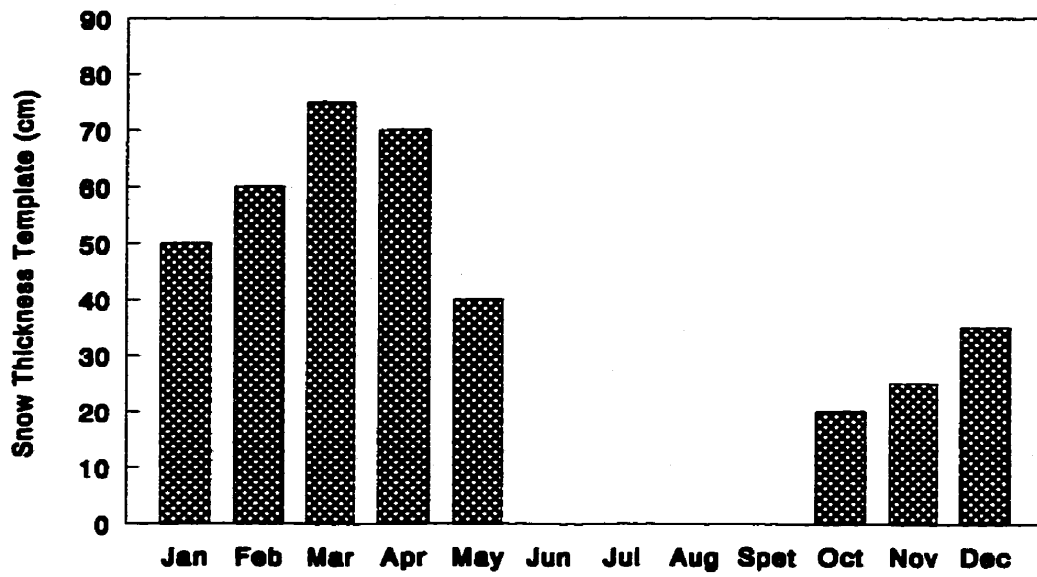


Figure 6.11 *Snow Thickness Template*

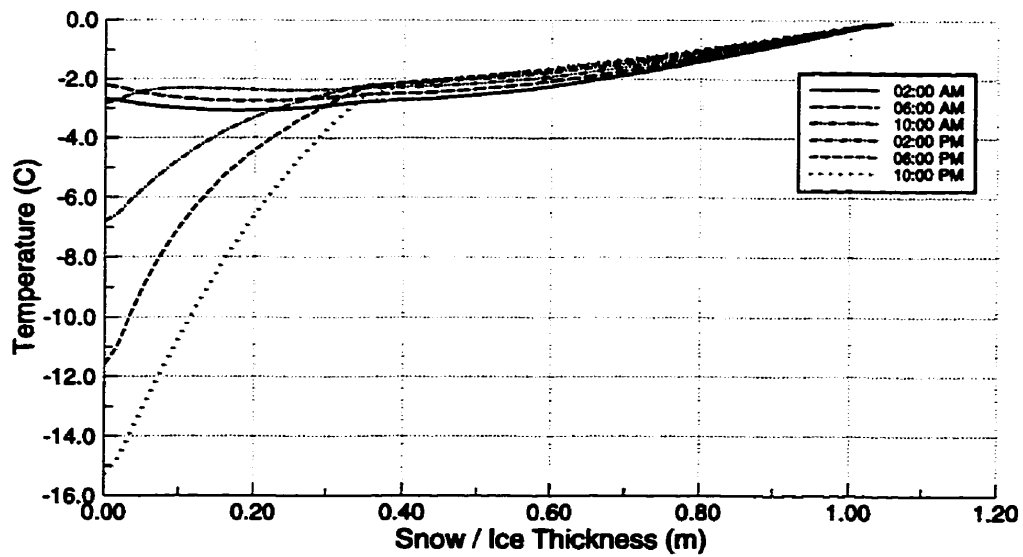


Figure 6.12(a) *Snow and Ice Temperature (Dec. 1st, 1993)*

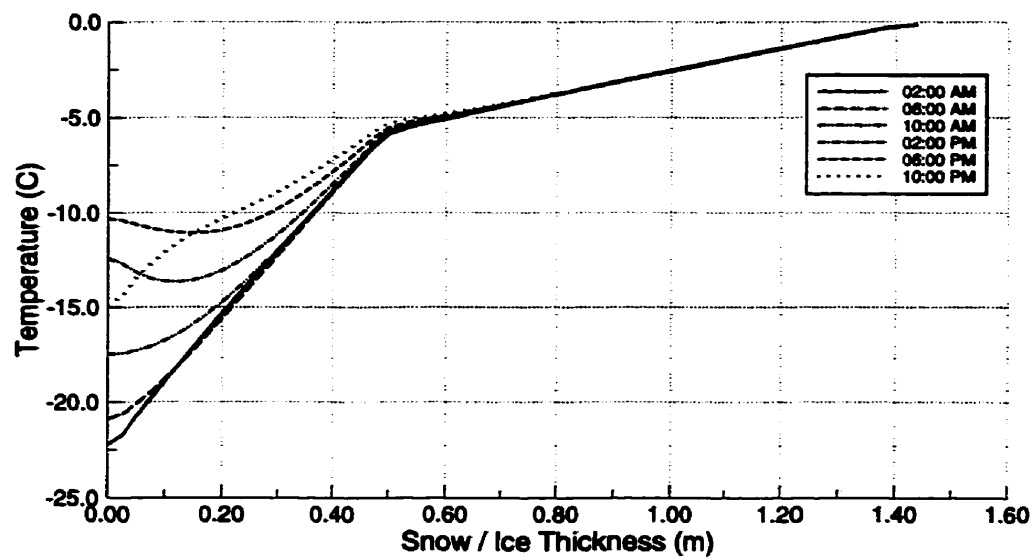


Figure 6.12(b) *Snow and Ice Temperature (Jan. 1st, 1995)*

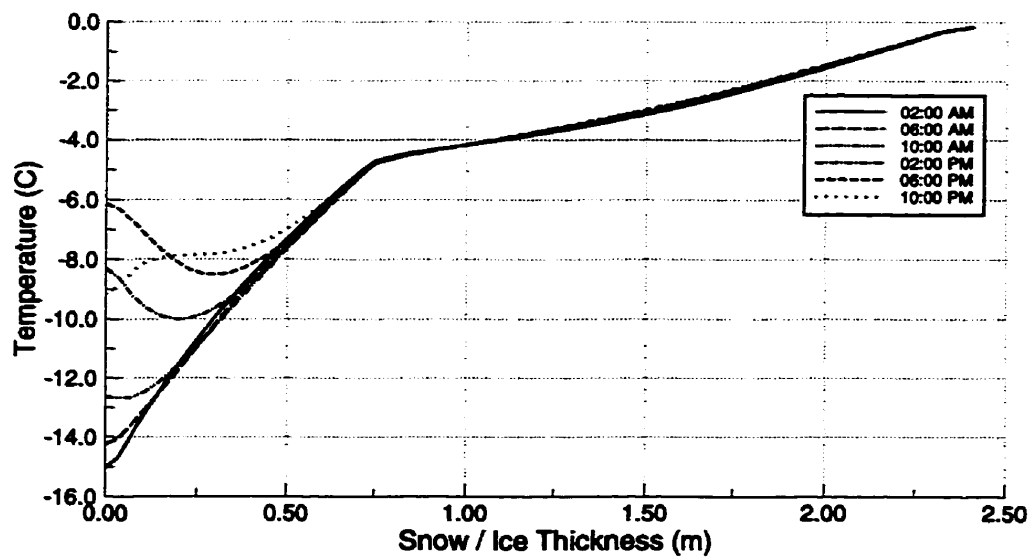


Figure 6.12(c) *Snow and Ice Temperature (Mar. 1st, 1995)*

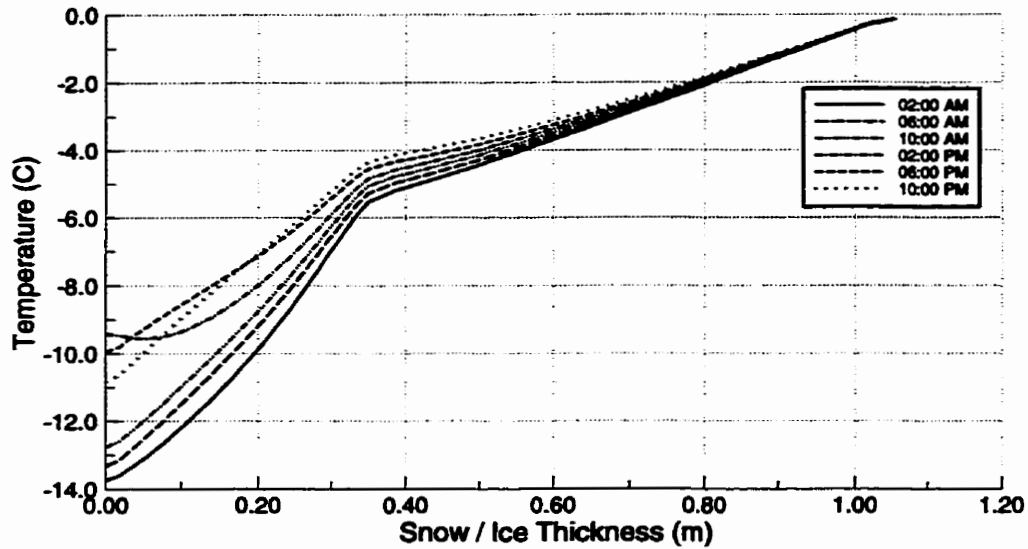


Figure 6.12(d) Snow and Ice Temperature (Dec. 1st, 1996)

Typical temperature distributions inside the snow and ice are given in Figure 6.12(a) through 6.12(d). It can be seen that, with the snow insulation, the daily ice temperature variation is quite small. Generally the lowest ice temperature is around -10°C .

6.5.2 Rheological Model for Ice Pressure

For estimating the ice pressure, the ice sheet is considered to be a constrained visco-elastic body. The ice strain rate is assumed to have the linear decomposition form:

$$\dot{\epsilon} = \dot{\epsilon}_e + \dot{\epsilon}_c + \dot{\epsilon}_T \quad (6.35)$$

where $\dot{\epsilon}_e$ stands for the elastic strain rate, $\dot{\epsilon}_c$ for the creep strain rate and $\dot{\epsilon}_T$ for the thermal strain rate. If the stress inside the ice sheet is σ , the elastic strain rate is related to it by:

$$\dot{\sigma} = E\dot{\epsilon}_e \quad (6.36)$$

The thermal strain rate depends on temperature via

$$\dot{\epsilon}_T = \alpha \dot{T} \quad (6.37)$$

where α is the thermal expansion coefficient of the ice. Only the creep strain rate now need be determined. Several rheological models have been proposed to calculate the thermal ice creep (Cox 1984, Ashton 1986 and Lewis 1993). The model introduced by Cox (1984) includes fewer parameters than others. The creep strain rate was set as:

$$\dot{\epsilon}_c = A(T) \left(\frac{\sigma}{\sigma^*} \right)^n \quad (6.38)$$

where σ^* is a unit stress in Pa , $A(T)$ is a function of the ice temperature T to describe the temperature dependence of the creep rate. The temperature dependence factor $A(T)$ is calibrated by the experiment conducted by Mellor and Cole (1982). The experimental data is listed in Table 6.3:

<i>Parameters of Ice Rheology Model</i> Table 6.3	
Temperature ($^{\circ}C$)	$A(T)$
-1	2.42×10^{-29}
-10	3.11×10^{-31}
-30	3.42×10^{-32}

Cox used a power function of

$$A(T) = B \left(\frac{T}{T^*} \right)^m$$

to fit the data in Table 6.3. An exponential function of the form

$$A(T) = \exp(a + bT)$$

is adopted for this study to fit the experiment data. The function $A(T)$ turns out to be

$$A(T) = \exp(1.366 + 0.4822T) \times 10^{-29} \quad (6.39)$$

If we consider the ice sheet is well constrained on its boundary, the total strain rate in equation (6.35) can be taken as approximately zero. Let the ice pressure p be equal to $-\sigma$, we obtain that

$$\dot{p} = E \left(\alpha \dot{T} - A(t) \left(\frac{p}{\sigma^*} \right)^n \right) \quad (6.40)$$

The nonlinear equation (6.40) can be integrated numerically once the time history of the ice temperature is available. In this rheological model, a low level tensile stress may be allowed. Micro-cracks are supposed to appear inside the ice sheet when the tensile stress reaches a certain level inside the ice sheet. The micro-cracks are flooded immediately by water to form a new ice mass and the ice strain is released.

In practical calculations, an effective modulus is used for E instead of the elastic modulus (Cox 1984, Lewis 1993). The parameters used for the rheological model are:

<i>Parameters of Cox Rheological Model</i>		<i>Table 6.4</i>
E	α	n
4.0 GPa	$(54 + 0.18T) \times 10^{-6}$	3.651

Some typical ice pressure distributions are depicted in Figures 6.13(a) through 6.13(d). Field measurement shows that the ice pressure is mainly concentrated in the top one third of the ice thickness (Xu 1981). The ice temperature near the ice sheet bottom changes little. A large variation takes place in the top part. Therefore, the stress in that part is relatively higher. The pressure over the ice thickness is integrated to yield the resultant horizontal load P on the dam structure:

$$P = \int_0^{h_i} p \, dy \quad (6.41)$$

The yearly horizontal load of the Long Spruce Dam is shown in Figure 6.14.

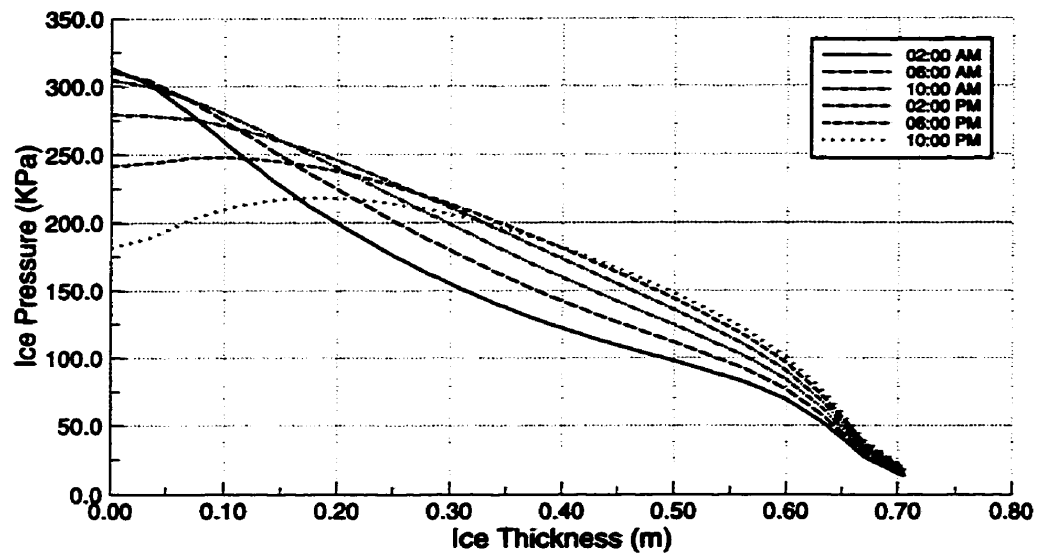


Figure 6.13(a) Ice Pressure (Dec. 1st, 1993)

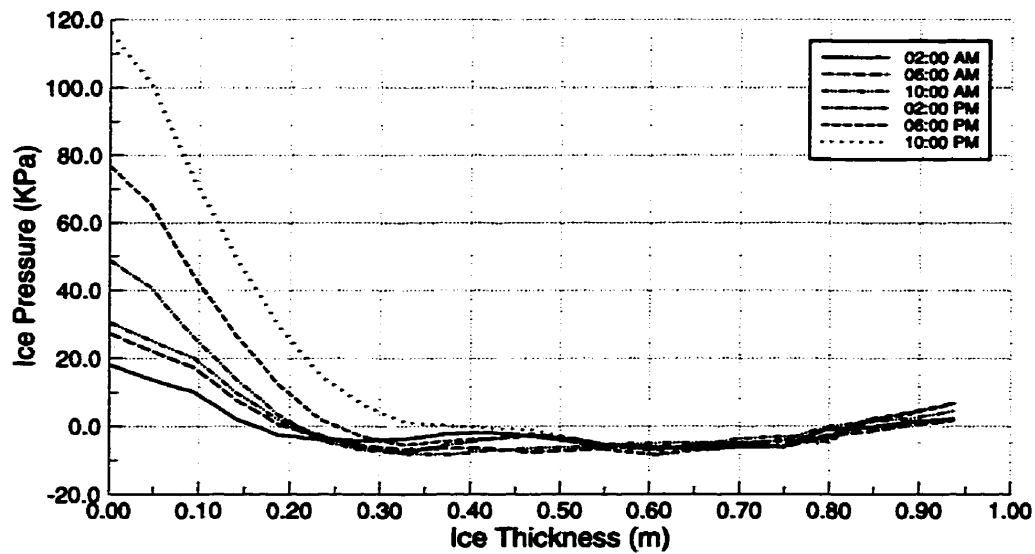


Figure 6.13(b) Ice Pressure (Jan. 1st, 1995)

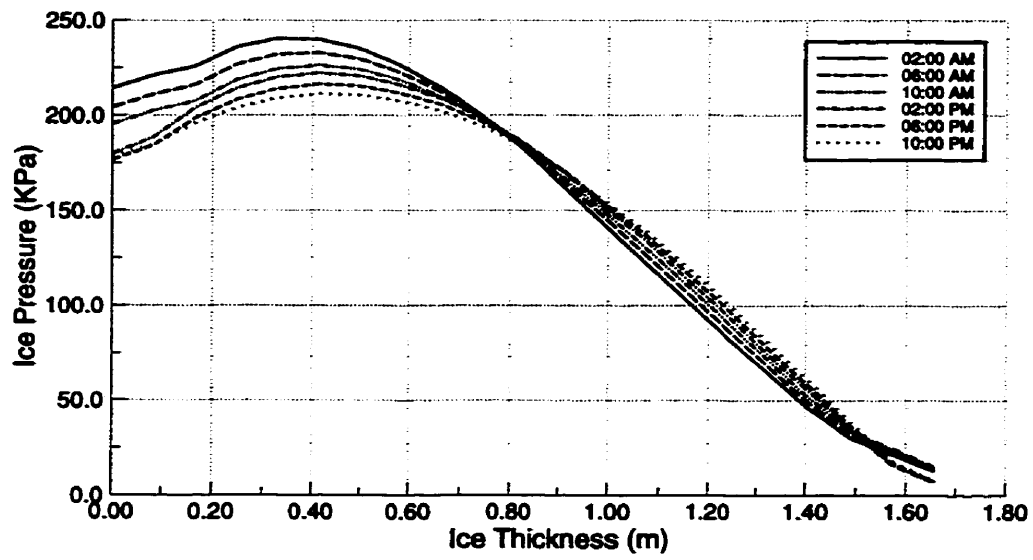


Figure 6.13(c) Ice Pressure (Mar. 1st, 1995)

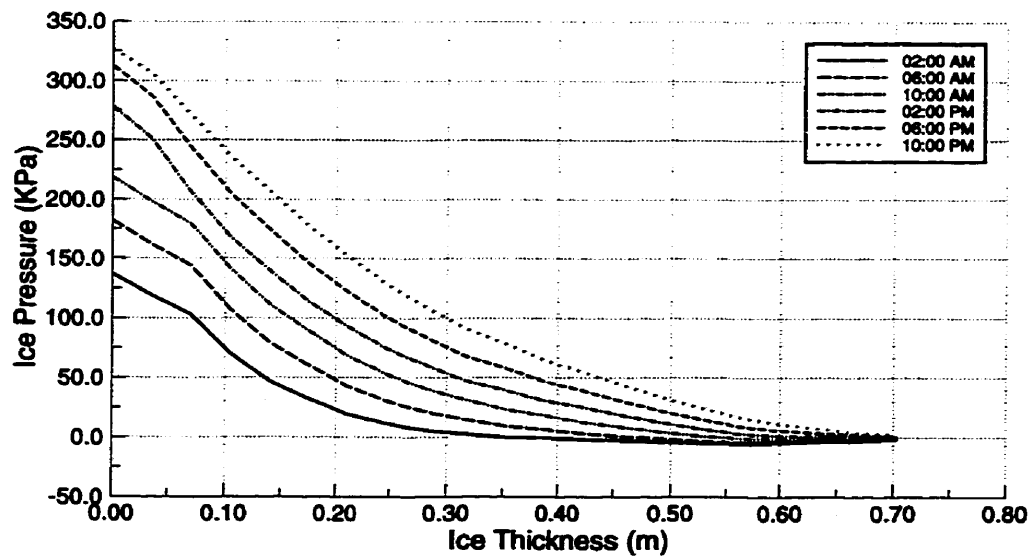


Figure 6.13(d) Ice Pressure (Dec. 1st, 1996)

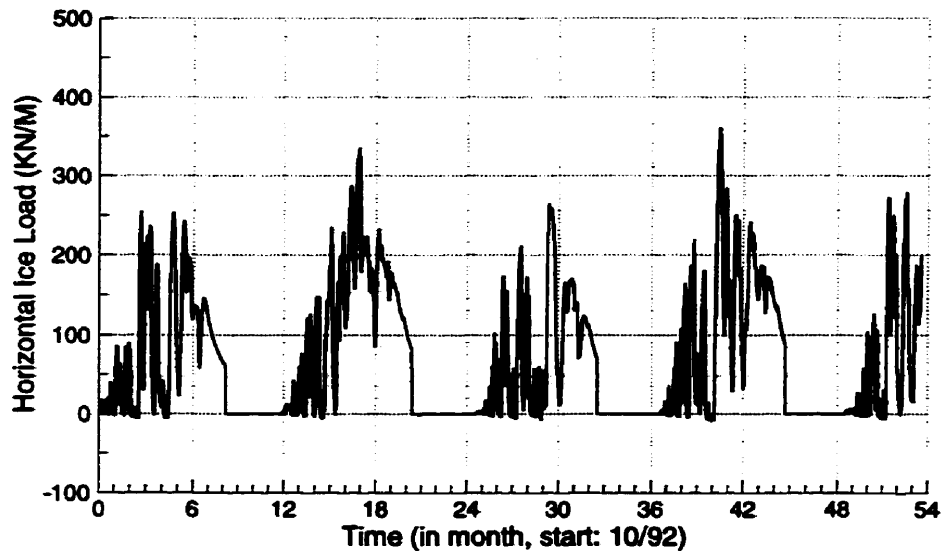


Figure 6.14 *Horizontal Load on the Dam Structure*

6.6 Motion Analysis of Long Spruce South Transition Structure under Ice Loads

After the vertical and horizontal ice loads are determined by the previous analyses, we can apply them on the dam structure. Using the finite element model of Chapter 5, the motion of the Long Spruce South Transition Structure is analyzed again. The vertical load (V. Load) and horizontal load (H. Load) are applied to the structure as shown in Figure 6.15. There are two ice load acting points. Point 1 is $1.1m$ above the leaking joint. The horizontal load and half of the vertical load act at this point. Another half of the vertical load acts at point 2, which is $0.26m$ above the leaking joint. The position of point 1 is close to the top of the ice sheet, where the horizontal stress concentrates.

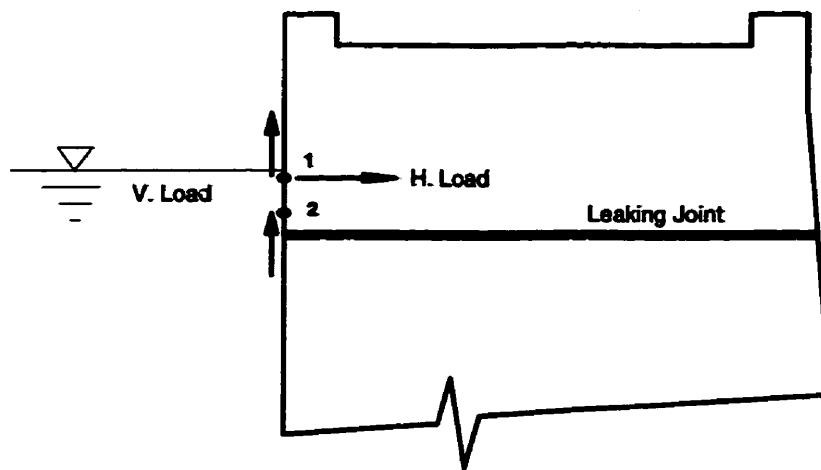


Figure 6.15 Ice Loading Points

6.6.1 Joint Opening Displacement

The structure is first loaded by vertical load only, and then, both vertical and horizontal loads are applied on the structure, in order to analyze their individual influences. With the ice loads, the finite element prediction of the joint opening displacement is significantly improved.

For the upstream side, the finite element results are compared with the recorded displacement at location 101U/S. Figures 6.16(a) through 6.16(e) present the comparisons for five years from 1992 to 1997. Except for the first year in which the temperature analysis is not totally convergent to the real situation, the finite element curve starts to increase at almost the same time for the record curve (December) and follows the record curve until the peak value very well. The predicted displacement peak value now has no distinct difference with the record. The post-peak behavior is not much improved because the ice inside the joint is not taken into account in the computation model.

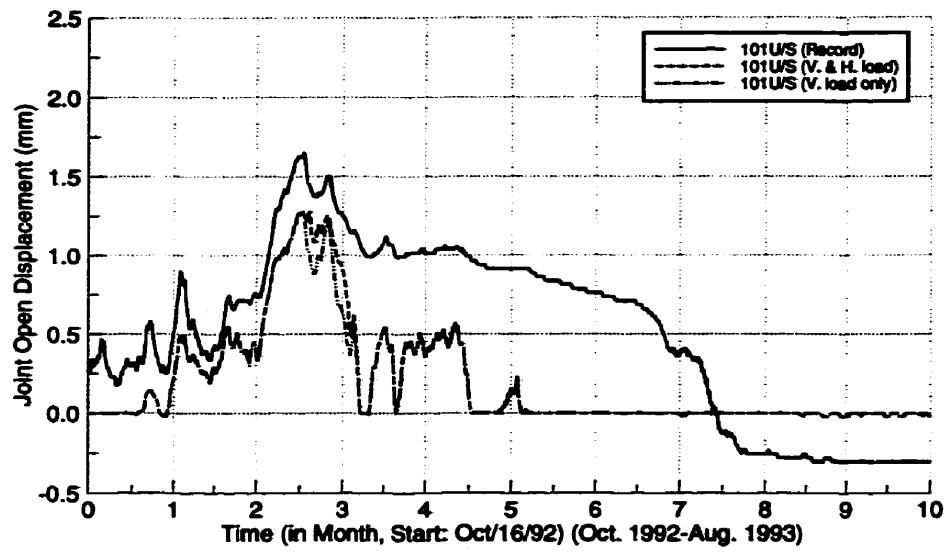


Figure 6.16(a) Joint Opening Displacement at Upstream Location

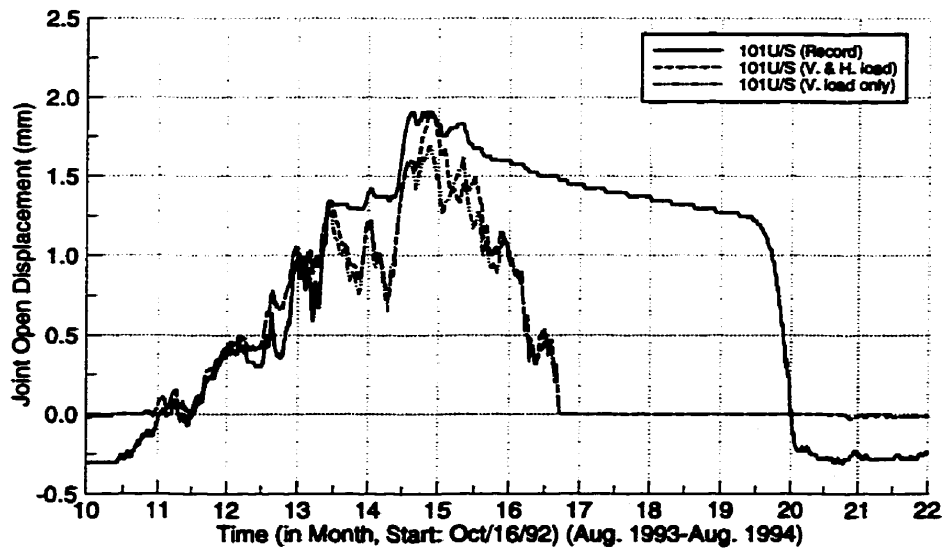


Figure 6.16(b) Joint Opening Displacement at Upstream Location

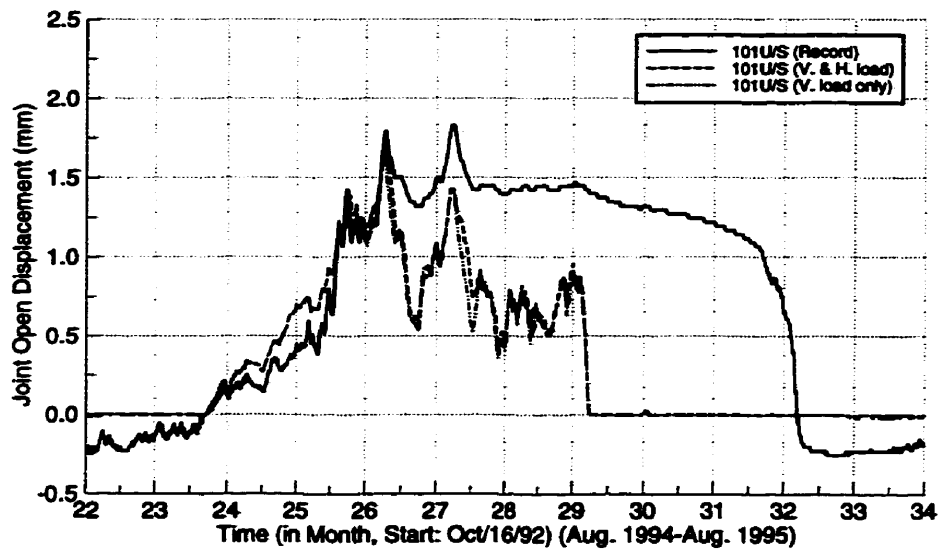


Figure 6.16(c) Joint Opening Displacement at Upstream Location

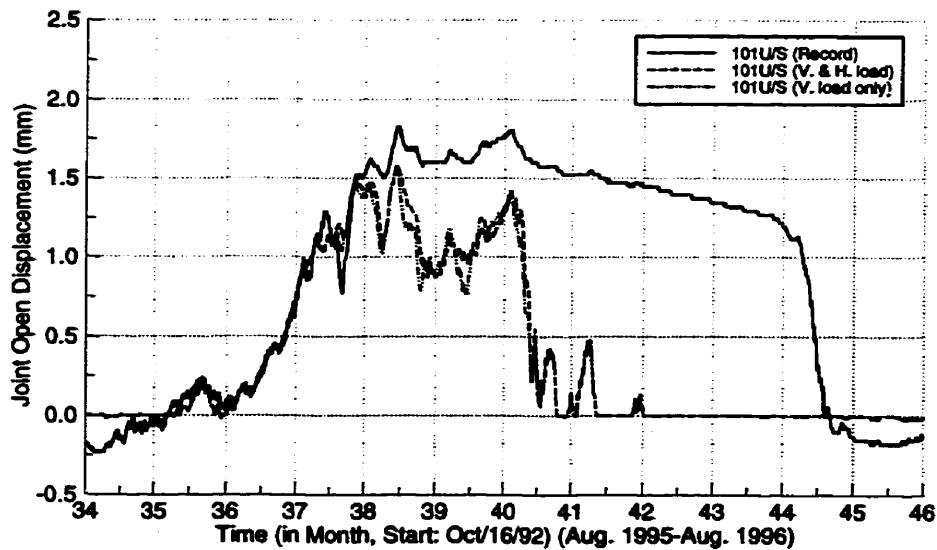


Figure 6.16(d) Joint Opening Displacement at Upstream Location

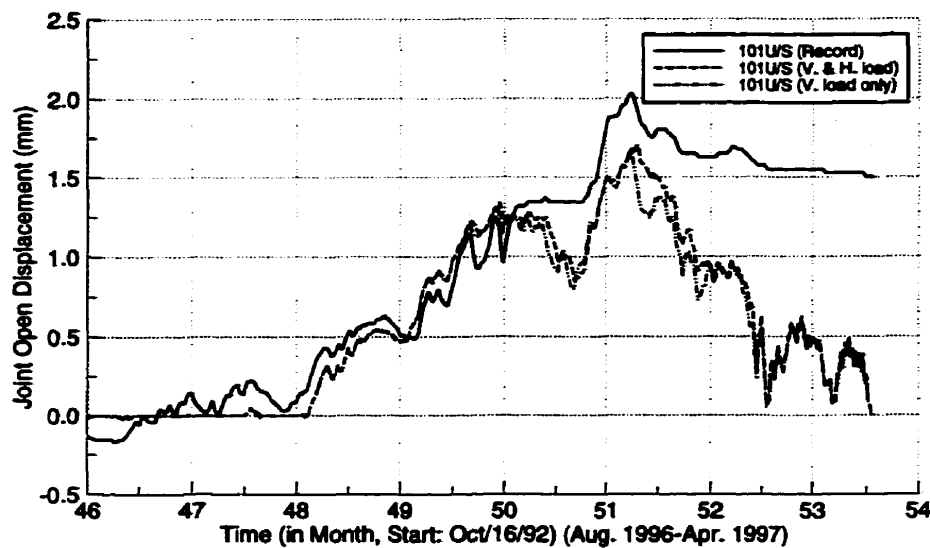


Figure 6.16(e) Joint Opening Displacement at Upstream Location

In comparison with the results of the no-ice model presented in Chapter 5, the ice loads brings the numerical curve up to reach the recorded peak displacement values. Therefore, the ice load caused joint opening displacement makes up the gap between the recorded peak value and the prediction by the no-ice model, which is about 40% of the total displacement.

As expected in the previous chapter, the joint opening displacement at the downstream side location is brought down with the application of the ice load. The upper block of the dam curls in winters. If one side (upstream side) goes up, the other side (downstream side) must go down. The finite element results provide a better fit of the record in this side. The comparisons between numerical results and the record at location 103 D/S are given in Figures 6.17(a) through 6.17(d). Even for the first year, the numerical results coincide well with the record data. The exception is in January 1997. A

large difference appears at that time. The operation of the heating tunnel on the downstream surface is believed to be responsible for it.

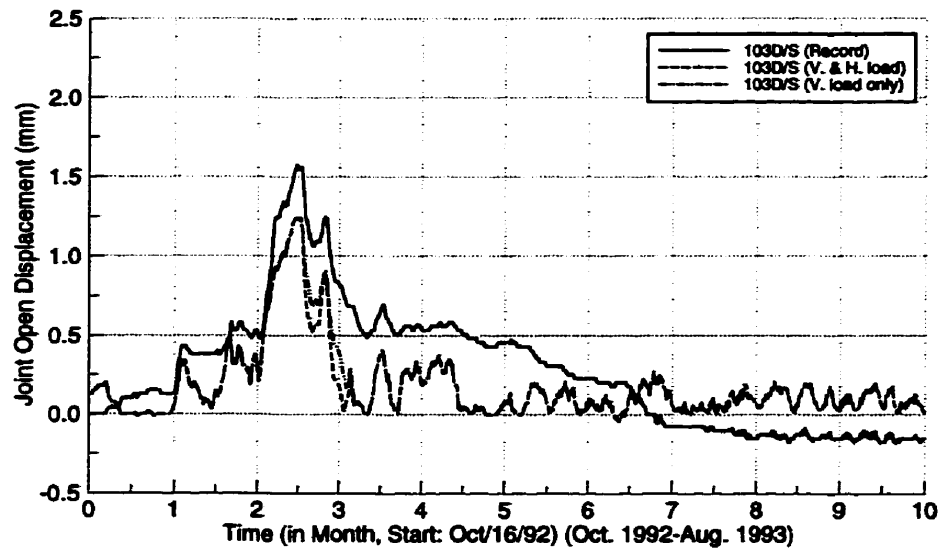


Figure 6.17(a) Joint Opening Displacement at Downstream Location

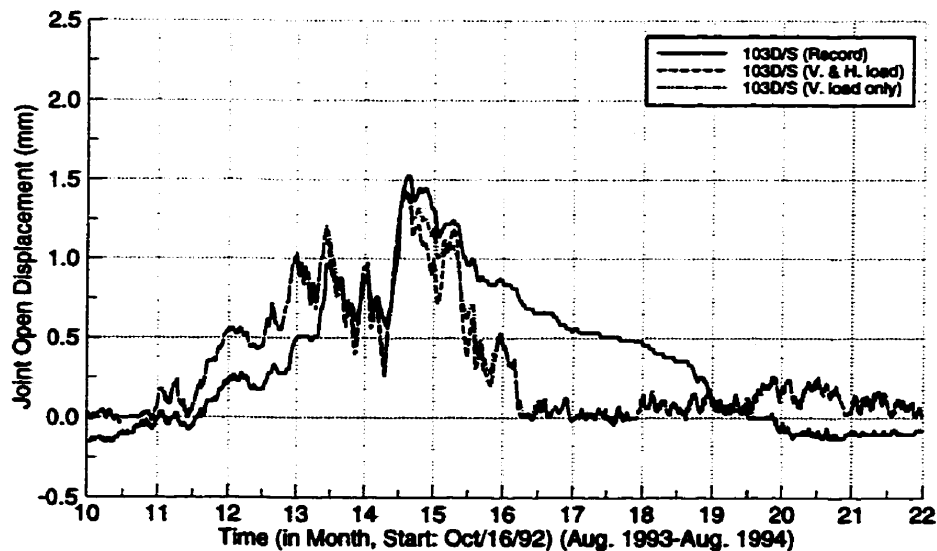


Figure 6.17(b) Joint Opening Displacement at Downstream Location

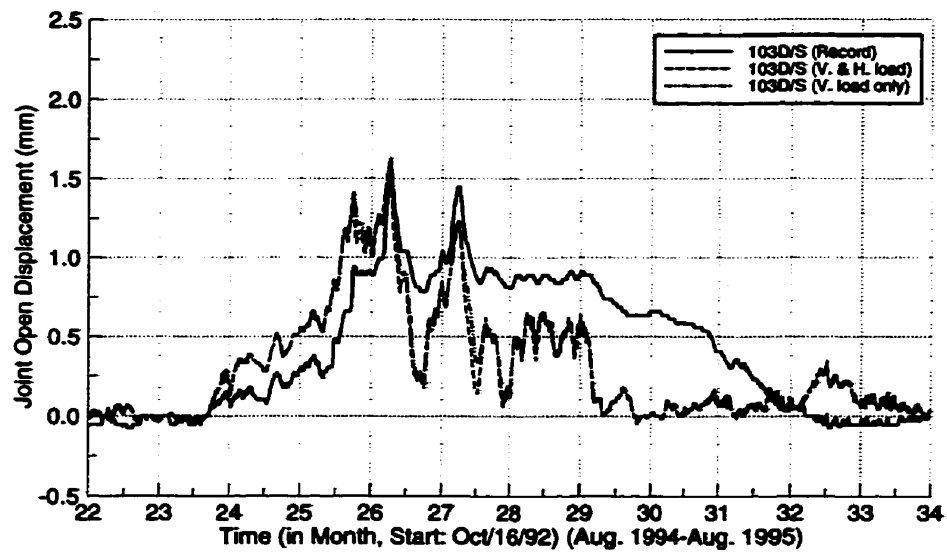


Figure 6.17(c) Joint Opening Displacement at Downstream Location

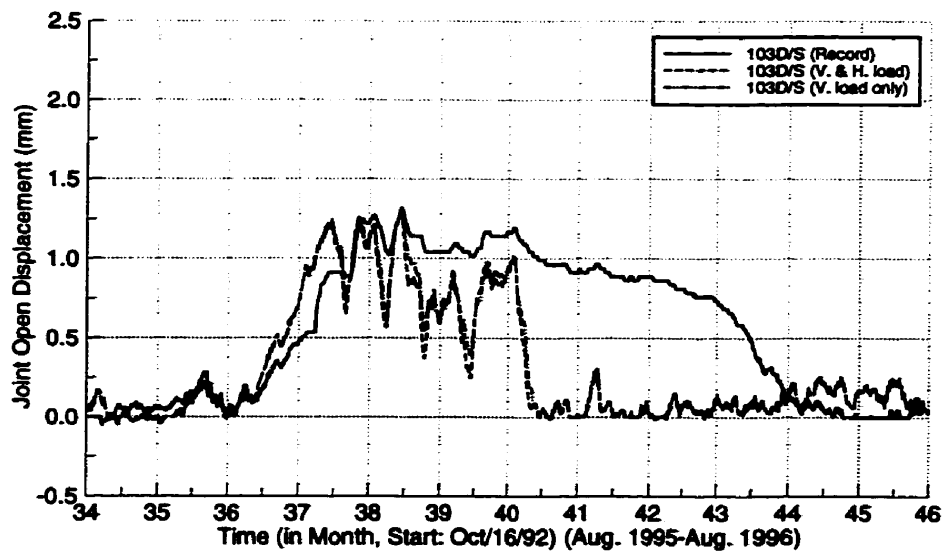


Figure 6.17(d) Joint Opening Displacement at Downstream Location

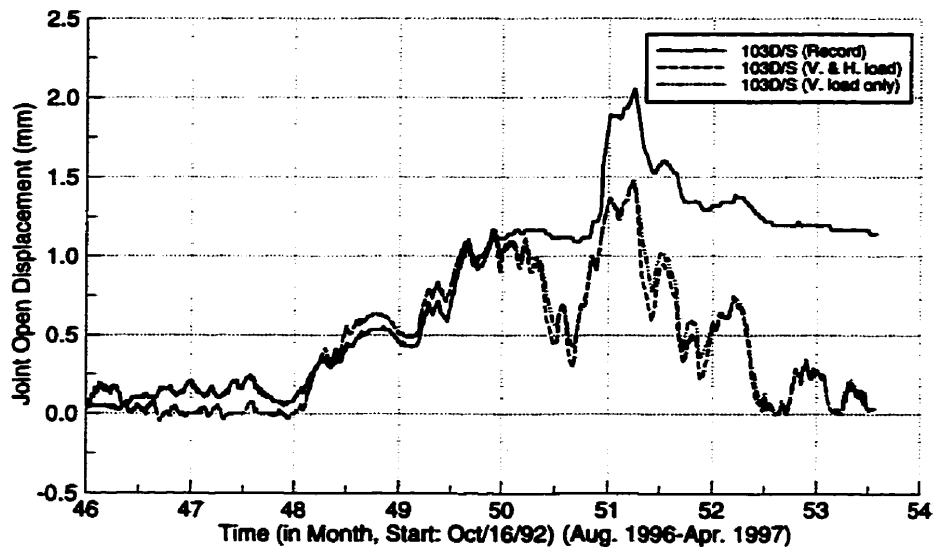


Figure 6.17(e) Joint Opening Displacement at Downstream Location

For the joint opening displacement at the upstream and downstream locations, Figures 6.16(a) through (e) and Figures 6.17(a) through (e) show that a very small difference exists between the curves corresponding to vertical loading only and vertical-horizontal loading cases. This indicates that the vertical load component of ice sheet is responsible for the joint opening displacement and the contribution of the horizontal load is negligible

6.6.2 Interface Stress in the Construction Joint

For the purpose of dam rehabilitation, the interface stress in the construction joint is studied. The construction joint is supposed to be intact. This might be the case either in the stage without damage or after the joint is fixed with grout. The loads are still the thermal load and the ice loads, as well as the gravity and water pressure. The interface stress evaluation method developed in Chapter 4 is used for this analysis.

The stresses at three groups of points in the joint are examined. The locations of the stress output are shown in Figure 6.18.

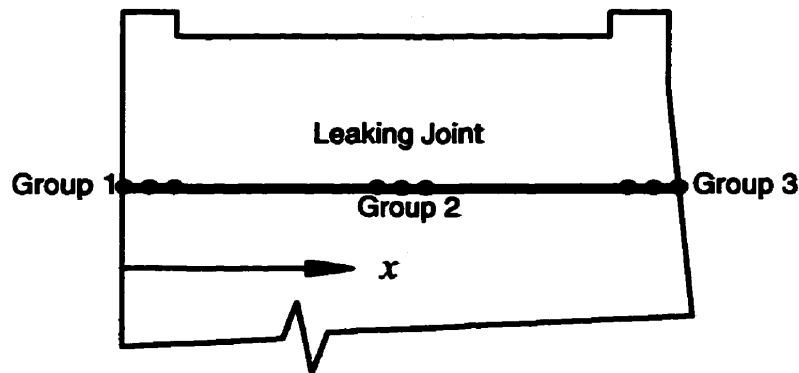


Figure 6.18 Stress Output Locations

The first group of points are in the upstream side with one on the upstream surface and the other two 0.46m apart inside the dam. The second group are in the middle of the dam. The points are 0.46m equally spaced and the central point is right in the middle of the joint. The third group are laid out on the downstream side in a similar fashion.

The normal stresses at group 1 are shown in Figures 6.19(a) through (e) and the shear stresses are shown in Figure 6.20. The normal stress reaches its yearly highest level around December and January and maintains a high level in February and even in March. On the boundary (upstream surface) the normal stress can be 4 MPa. The stress level drops quickly toward the center of the dam center. At the point 0.46m away from the boundary, the highest stress level is about 2 MPa. The stress changes its sign twice a year in October and April. A low stress level exists on the interface during these months. It can be observed that the shear stress level is 0.5 MPa, which is much lower than that of normal stress. Therefore, the damage of the joint has less relation with the shear stress.

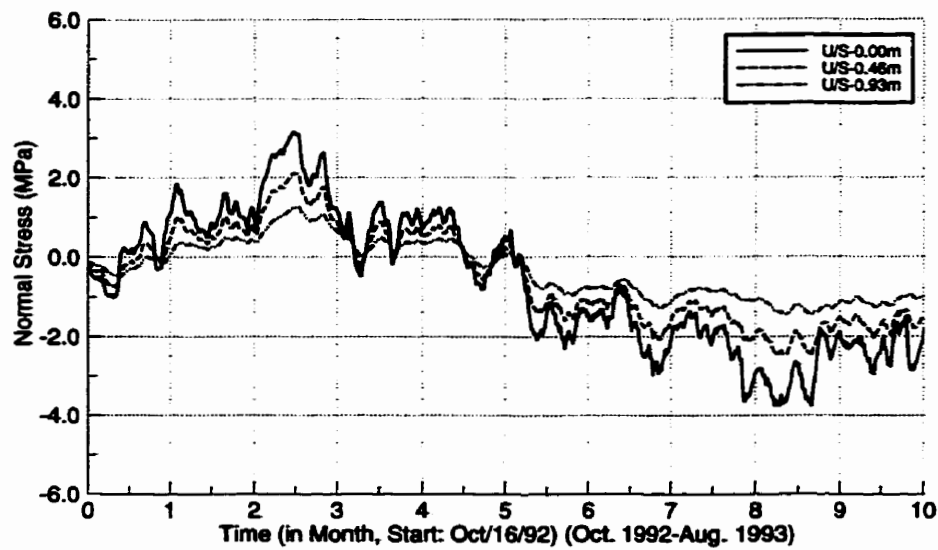


Figure 6.19(a) *Interface Normal Stress in the Upstream Side*

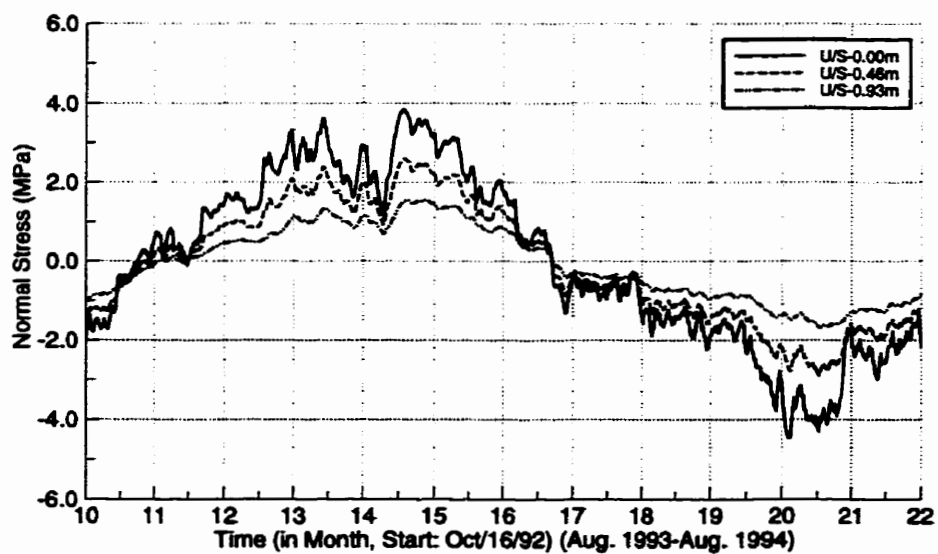


Figure 6.19(b) *Interface Normal Stress in the Upstream Side*

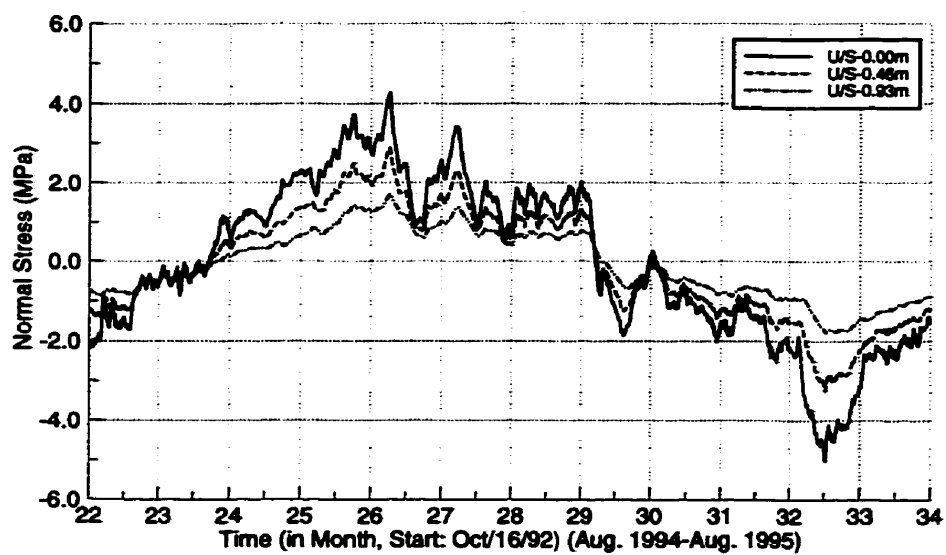


Figure 6.19(c) Interface Normal Stress in the Upstream Side

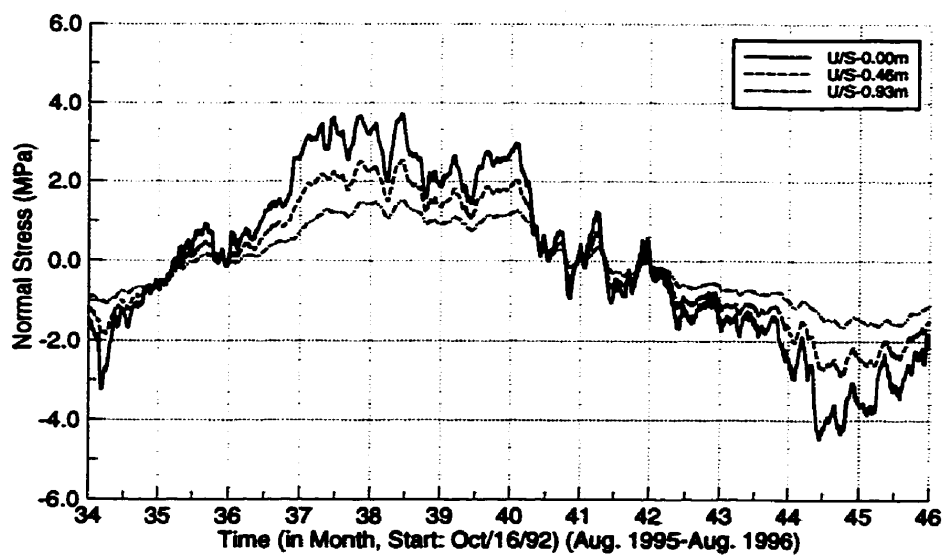


Figure 6.19(d) Interface Normal Stress in the Upstream Side

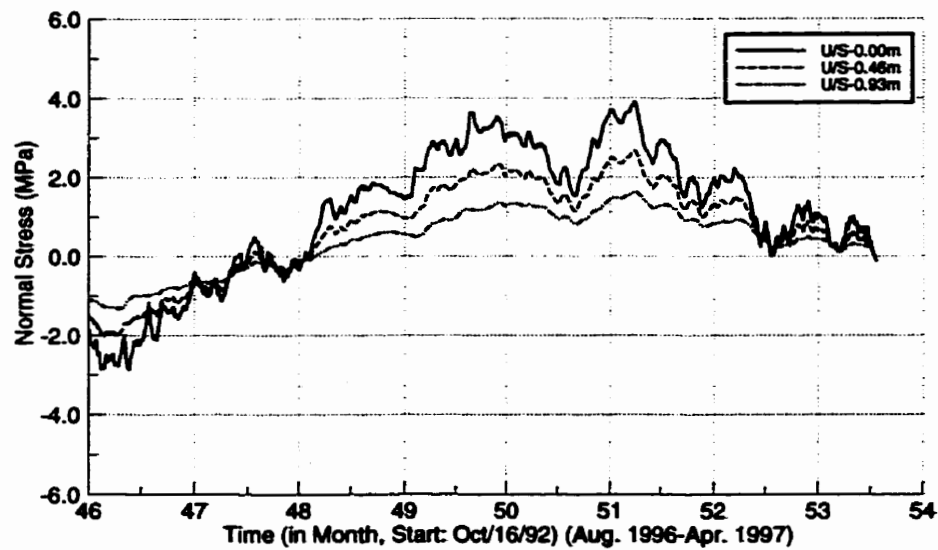


Figure 6.19(e) Interface Normal Stress in the Upstream Side

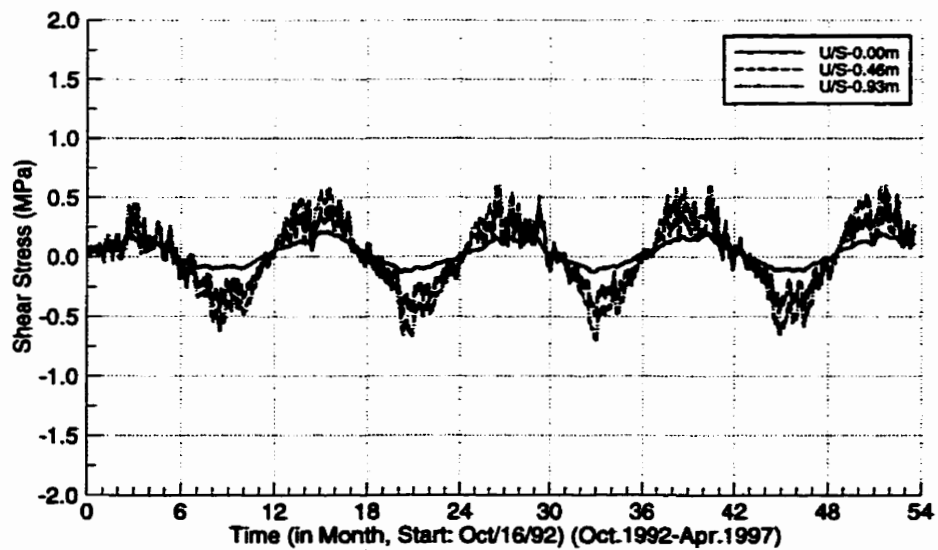


Figure 6.20 Interface Shear Stress in the Upstream Side

The normal stresses in the downstream side are given in Figures 6.21(a) through (e). The shear stresses are shown in Figure 6.22. The situation is quite similar to the upstream side. The normal stress reaches its peak value at the same time as in the

upstream side. Because the downstream side is exposed to the air, the temperature variation is even severe in this side. The yearly stress variation is harsh. A higher stress level of 6 MPa ~ 7 MPa appears every year on the boundary. At the point 0.46m inside the dam, the stress can still be as high as $3\sim 4\text{ MPa}$. The shear stress level is $0.6\sim 0.7\text{ MPa}$, which is also higher than that in the upstream side, but relatively low with respect to the normal stress.

The stress state variation in the dam center is slow. The stress level is lower than that near the dam surfaces. The normal stress is less than 2 MPa and shear stress is less than 0.4 MPa . Figures 6.23 and 6.24 show the normal and shear stresses in the dam center.

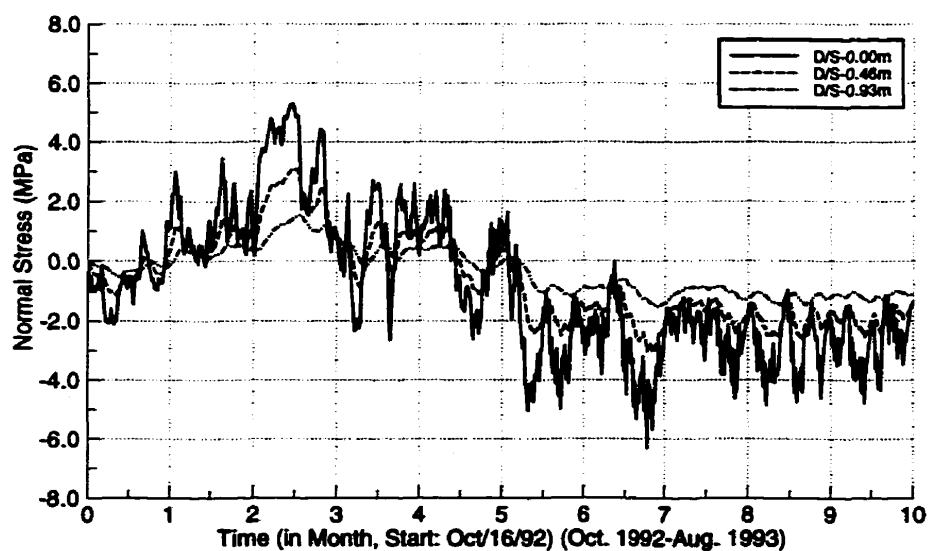


Figure 6.21(a) *Interface Normal Stress in the Downstream Side*

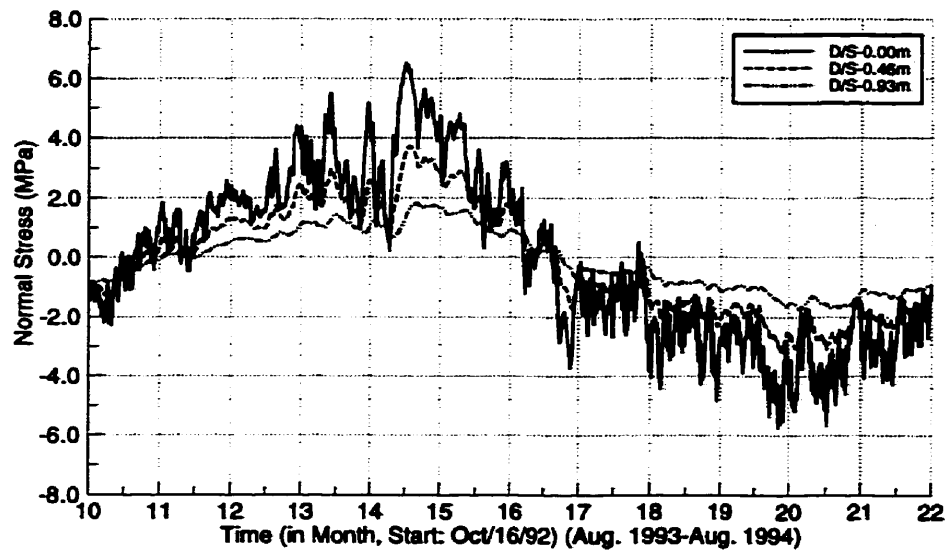


Figure 6.21(b) *Interface Normal Stress in the Downstream Side*

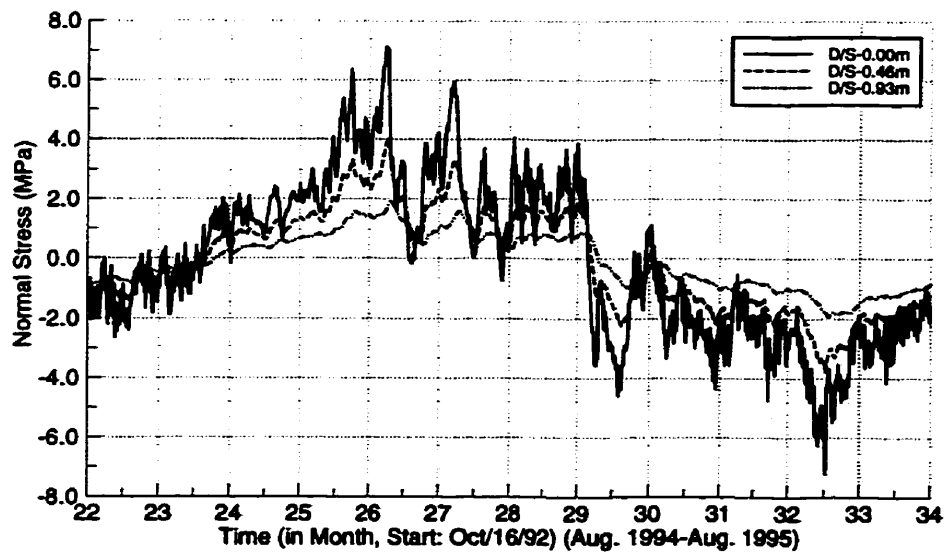


Figure 6.21(c) *Interface Normal Stress in the Downstream Side*

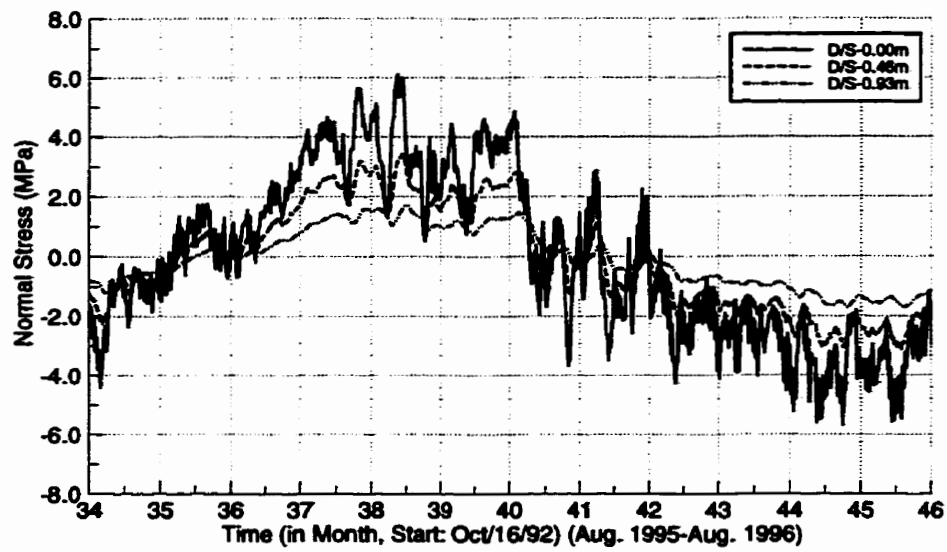


Figure 6.21(d) *Interface Normal Stress in the Downstream Side*

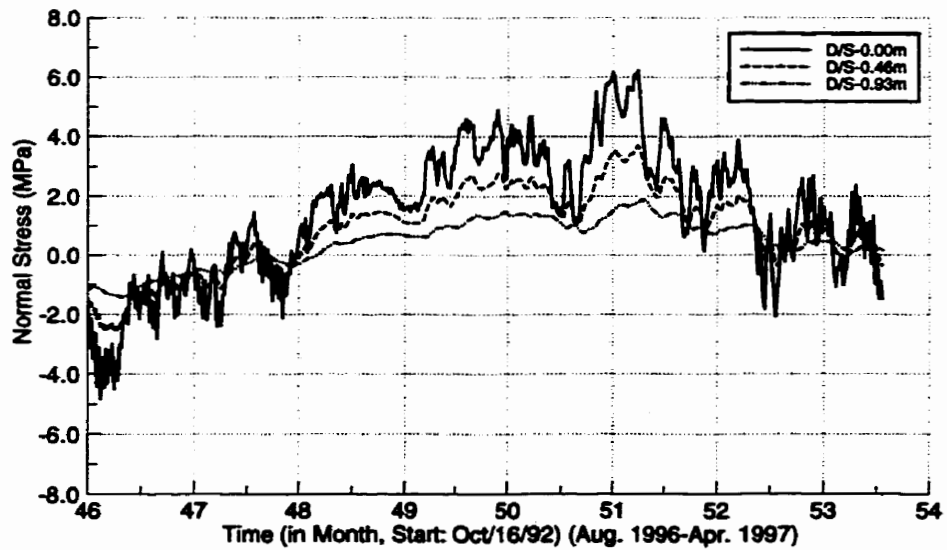


Figure 6.21(e) *Interface Normal Stress in the Downstream Side*

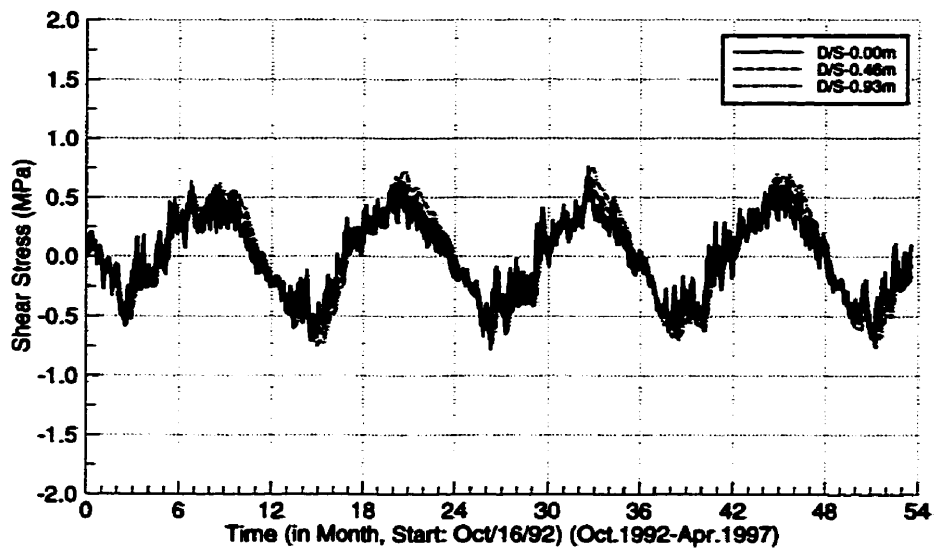


Figure 6.22 *Interface Shear Stress in the Downstream Side*

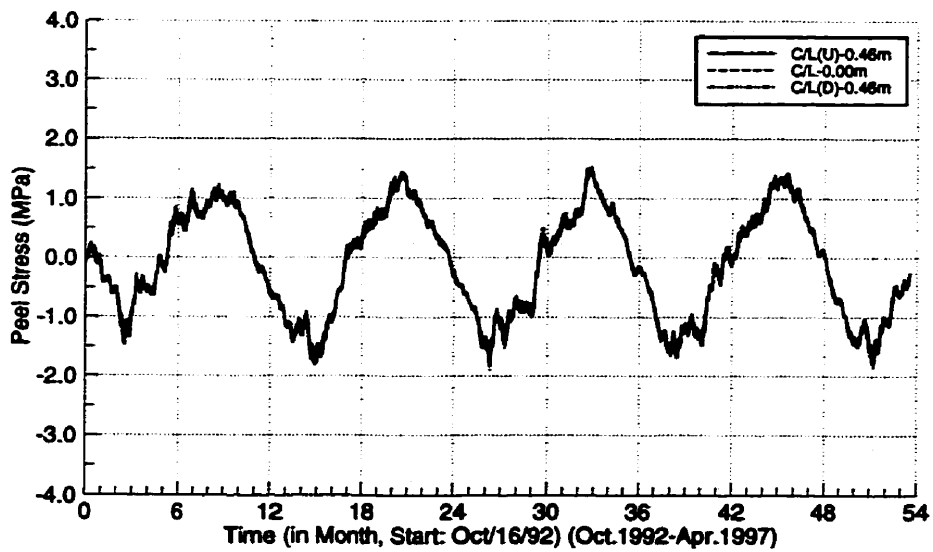


Figure 6.23 *Interface Normal Stress in the Dam Center*

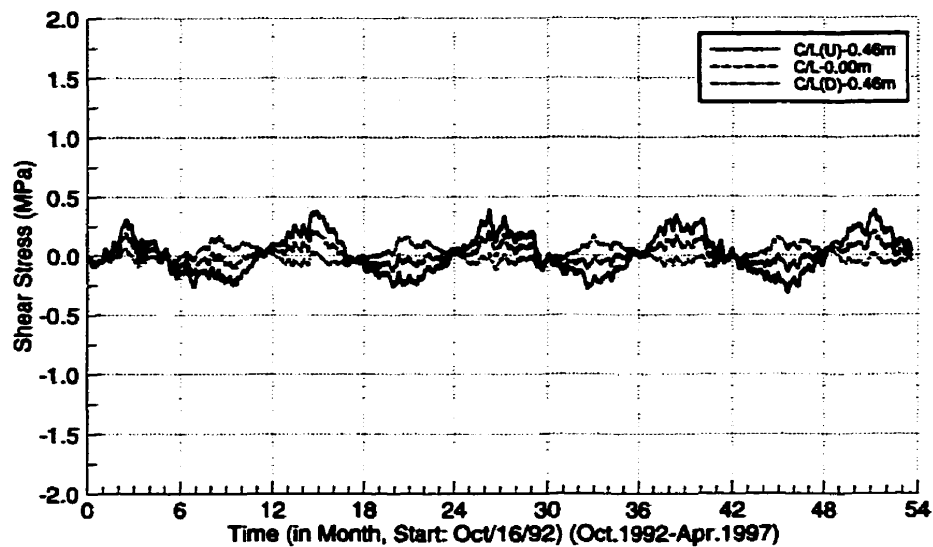


Figure 6.24 *Interface Shear Stress in the Dam Center*

Typical interface stress distributions in winter and summer are shown in Figure 6.25 and 6.26, respectively. The origin of the coordinate X is at the intersection of the construction joint and the upstream surface, as shown in Figure 6.15. The length of the joint is $9.72m$. Very smooth interface stress distributions are obtained. From Figure 6.25, it can be observed that, over a distance of two meters, the normal stress is above zero near the upstream side. The tensile normal stress in the downstream side is higher and drops faster. Over a distance of about $1.5m$ it changes to a compressive state. In these two high tensile stress zones, it is quite easy for the interface to crack. In the summer season, a tensile state takes place in the center of the dam. This tensile zone is about $6m$ long, but the level is relatively low (See Figure 6.26.) The shear stress state in the winter and summer is quite similar. Its level is almost the same, only the direction changes. It is worth noting that the shear stress is close to zero at the origin, because the joint is perpendicular to the upstream surface.

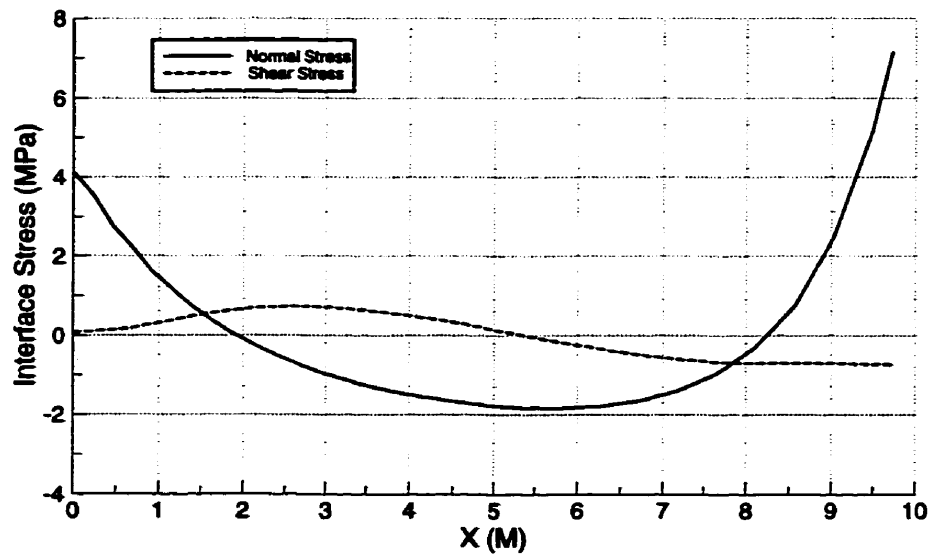


Figure 6.25 *Interface Stress Distribution in Winter (Dec. 12th, 94)*

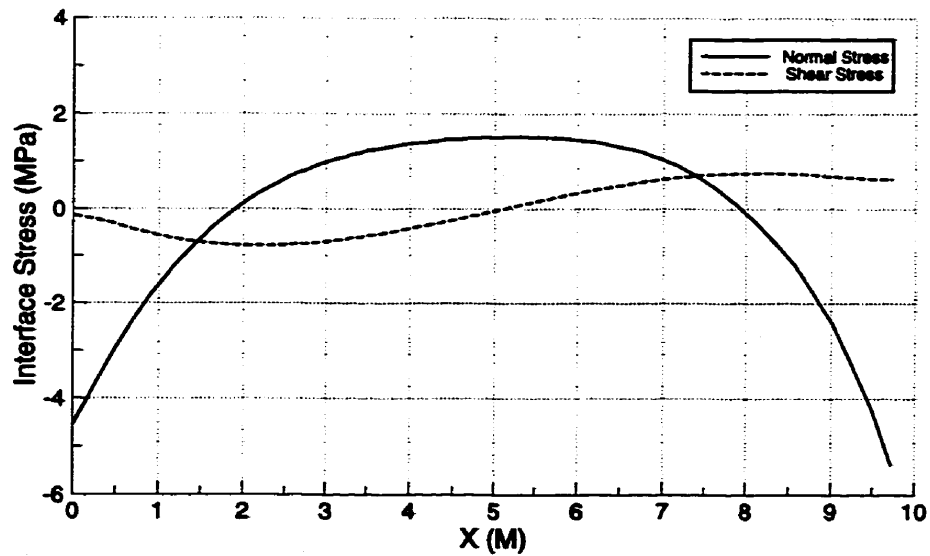


Figure 6.26 *Interface Stress Distribution in Summer (June 6th, 95)*

6.7 Summary and Conclusions

Ice loads are believed to be one of the major factors to cause damage in dams. To obtain accurate ice loads on the dam structure is an extremely difficult task. Certain load data can be obtained from field measurements. However, ice loads are strongly environmental dependent. The measurements obtained from one place are not usable for other places. Although many environmental factors, such as the air temperature, snow cover, solar radiation, wind drag, water flow and wave influence the ice formation and loads, the air temperature variation plays the most important role.

In this chapter, an attempt is made to develop several models to relate the ice thickness, vertical load and horizontal load with ambient air temperature. The widely accepted freezing degree-day method is used to estimate the change in ice thickness. The predicted ice thickness coincides with the available field measurements. It forms the base for the ice load estimation model.

An elastic plate model is used to analyze the vertical ice load on the structure. The vertical deformation and bending stress state of an ice sheet are analyzed by this model. Cracks are predicted to appear because of the high bending moment. A crack near the ice sheet root appears in the early stage of ice freezing and acts as a hinge during the winter season. This crack changes the ice sheet boundary conditions and releases much of the vertical ice load acting on the structure. The bending moment causes a second crack in the ice sheet at a certain distance from the dam structure. According to the ice thickness and water wave data for the Long Spruce forebay, the maximum bending moment inside the ice sheet after the appearance of the root crack is quite close to the bending strength of ice plate. The ice sheet is on the verge of having a second crack. In consideration of water flooding of this crack, the ice sheet is supposed not to have this

crack. The vertical load is calculated by a half infinite elastic plate with a simply supported boundary. This assumption may be not true if the ice thickness and water wave data are different.

More unknown factors exist for the analyses of the horizontal and vertical loads. The zero total strain assumption is adopted to avoid the complicated boundary conditions of the forebay bank. Only two major factors, the ice temperature and the creep behavior, are taken into account. The temperature change rate has its immediate effect on the ice stress variation and the rheological character of ice causes the stress relaxation. Generally the ice sheet is insulated by a snow cover. One dimensional heat transfer processes have been solved to relate the temperature of the snow covered ice with the air temperature. The horizontal stress inside the ice sheet is obtained by numerical integration of a nonlinear rheological equation. Resultant of distributed ice stress over the ice thickness forms the horizontal ice load. Because a rigid boundary condition of ice sheet has been used, the horizontal load by this model is overestimated.

The theoretically estimated ice loads are applied to the Long Spruce structure. The numerical results with these loads turn out to be much better than those by the no-ice model. The pre-peak behavior of the construction joint opening displacements at upstream and downstream side locations can be predicted well by the finite element method. The predicted peak values of the opening displacement are quite close to the recorded values in every year of analysis. The no ice model predicts only about 60% of the peak values. The ice loads bring the numerical peak values close to the recorded values. This verifies that the ice loads cause about 40 % of the peak values. Therefore, the ice loads are a must-be-considered factor for any stress and rehabilitation analysis of this kind of dam structures.

The individual influences of vertical and horizontal ice loads are examined. The results show that the numerical joint opening displacement curves, obtained by applying the vertical load only and by applying both of them simultaneously, are very close. With the application of even an overestimated horizontal load, the displacements do not change significantly. Actually, the effect of the horizontal load on the joint opening displacement can be negligible in comparison with the effect of the vertical ice load. The vertical load acts at the position very close to the construction joint. It causes a tensile or normal stress on the joint interface; meanwhile the horizontal load causes more shear stress on the interface. Along with the thermally induced stress, the vertical load is much more harmful to the joint interface.

An ice jacking effect influences the post-peak behavior of the opening displacement. It may keep the joint open in later winter when the joint is going to close. It doesn't cause any joint opening displacement as the forebay ice load or the thermal load does. Therefore, the ice jacking is considered to be less important than the winter deformation analysis of the dam structure.

For the purpose of an initial joint damage analysis and rehabilitation analysis, the dam is supposed to form a perfect joint under the action of all kind of loads. The interface stress is calculated. It is found that high tensile stress zones exist near the upstream side and downstream side, on the construction joint with a $1.5\text{--}2\text{m}$ range in winters. The highest tensile stress in the downstream side is as high as 7 MPa , and in upstream side 4 MPa . For this tensile stress level, it is quite easy for the interface to crack. In summer, a tensile zone is about $2/3$ of the joint length in the dam center, but the stress level is lower than 2 MPa . The shear stress is always low as $0.4\text{--}0.6\text{ MPa}$ during winter and summer.

If a leaking joint is to be fixed, the interface must be strong enough to withstand the above stress level. With the stress data known, it will be easier to choose a feasible rehabilitation strategy.

Chapter 7

Studies on the Repair of Long Spruce South Transition

7.1 Introduction

The failure mechanism of the construction joint in the Long Spruce South Transition has been studied. Thermally induced stress and ice loads are found to be mainly responsible for the joint debonding and structural degradation. The slow thermal response of a dam structure to the ambient air temperature variation maintains a larger temperature difference between the dam top and lower part. This temperature difference makes the top construction block bend upward (in winter) and downward (in summer). A very high level tensile stress zone appears near the upstream and downstream sides of the structure, and the forebay ice exerts a horizontal and a vertical load on the upper construction block. Ice loading creates a very large local stress concentration, whose location is close to the ends of the construction joint. They form a worse load combination to the dam structure. This load combination causes partial and even total failure of the construction joint. From the situation of the Long Spruce South Transition structure, it can be seen that even for the newly built dam structure, cracks in construction joints occur only after a few years of service.

The Long Spruce South Transition was repaired by grouting the upper block joint. However, the grout only stood for one or two years. As an interface, high level stresses usually appear at the ends of construction joints where the thermal and ice loads act. A theoretical motion analysis of the structures with or without cracked construction joints is

expected to be instructive for the rehabilitation work. Analysis work has been done to simulate the transient thermal response of the structure, the behavior under thermal load and ice load actions, especially the interface displacements and stresses on the cracked/uncracked construction joint. Through the thermal analysis, motion analysis and ice-loading model, we are able to theoretically predict the opening displacement of the cracked construction joint to a certain extent. The interface stress is also made clear when the joint has a perfect bond. Based on these results, several suggestions can be made for the dam repair.

7.2 Mechanism of Joint Failure

Construction joints of dams are the weak parts of the structure. The joints in high temperature gradient regions are vulnerable to severe ambient air temperature variations. During long service, the bond of a construction joint may be partially or totally broken. The thermally induced motion analysis, coupled with the forebay ice loads, has been carried out to determine the thermal and ice force driving the crack opening displacement and the interface stress needed to keep the crack closed.

In order to check the separate effects of ice and ambient thermal conditions, firstly the motion of the structure was analyzed with thermal load only. Secondly, the analysis was performed with both thermal and ice loads. The no ice model predicts only about 60% of the peak values of the opening displacement in the joint. After including the ice loads, the numerical prediction is found to be close to the in-situ recorded value. This verifies that the ice loads cause about 40% of the peak values, and therefore must be accounted for any stress and rehabilitation analysis. The role of horizontal and vertical ice forces on the joint motion was also examined separately. It turns out that the vertical ice

load is much more significant than the horizontal load for the opening displacement of the joint. The vertical load acts at the position very close to the construction joint. It causes a tensile stress on the joint interface; meanwhile the horizontal load mainly causes a shear stress in the interface. Along with the thermally induced stress, the vertical load is much more harmful to the joint interface.

To examine the interface stress needed for keeping the joint closed, the dam is supposed to have a perfect joint under the action of all loads. The interface stress is calculated for this case. It is found that high tensile stress zones exist near the upstream side and downstream side, which cover a distance of about $1.5\sim 2m$ in each side in winters. The highest tensile stress in the downstream side can be as high as 7 MPa , and in the upstream side up to 4 MPa . In summer, a tensile zone is in the dam's center, which is about $2/3$ of the joint length, but the stress level is lower than 2 MPa . The shear stress is always low in winter and summer ($0.4\sim 0.6\text{ MPa}$). Typical normal stress distributions are shown in Figures 7.1 and 7.2 for winter and summer time respectively. If the joint is repaired with a grout layer, the stress level would be at such a level.

For a newly built dam structure, the residual stress caused by concrete hydration is another important factor to be considered for joint strength. Analyses have been done and reported by Radovanovic (1997). The distribution of the residual normal stress along the construction joint, which can be seen in Figure 7.3, has a similar pattern to the stress distribution caused by the thermal and ice loads during winter. In worse cases, the maximum normal tensile stress level is about 2 MPa . The superposition of the thermal and ice load caused stress and the residual stress makes the total tensile stress level high up to $5\sim 6\text{ MPa}$ locally, near the joint end in upstream side and even higher in the downstream side.

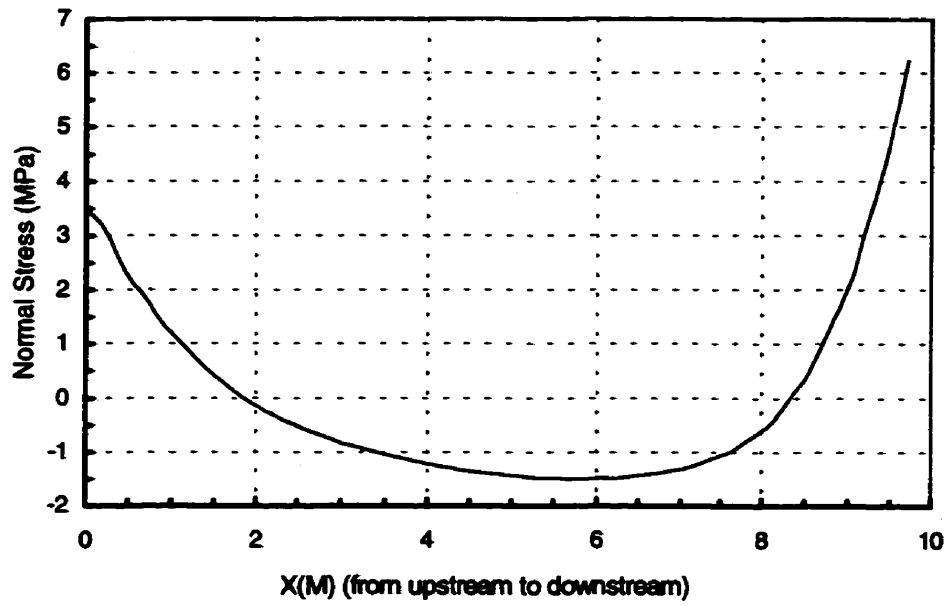


Figure 7.1 Normal Stress Distribution along Joint (winter time)

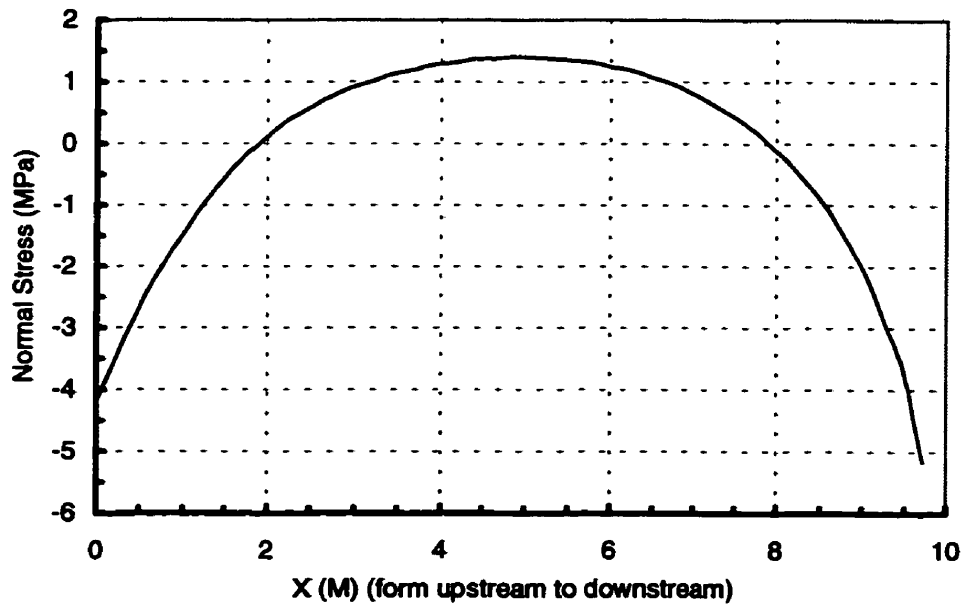


Figure 7.2 Normal Stress Distribution along Joint (summer time)

Such high level tensile stress results in a crack initiation. After crack initiation, some of deformation energy inside the dam structure will be released. However, the singular stress around the crack tip will keep driving the crack to propagate inside the core of the dam.

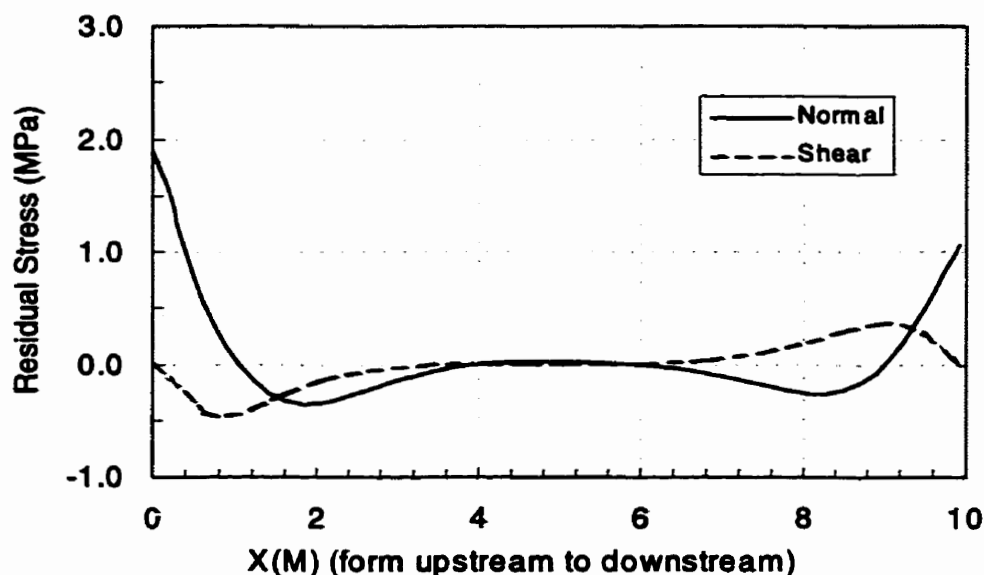


Figure 7.3 *Residual Stress along the Construction Joint*

If the leaking joint is going to be fixed, the interface must be strong enough to stand the above stress level. Experiments have been performed to test the strength of a concrete joint (Irhouma 1997). The interface strength is found to be around 4MPa for a laboratory specimen. In practical repair, it is not expected that the grout can create an interface with such strength. Therefore, the joint is bound to crack again after one or two year service. To simply inject grout into the cracked joint does not seem practical.

We have to turn to other suitable ways to get the construction joint fixed. Anchors have been used in Manitoba Hydro projects in the past. They can stand for relatively high stress level, as needed. It is expected that the anchors can bear a large part of the tensile

stress as in the case of reinforced concrete. Numerical studies have been performed to determine the proper anchor installation time and the type of anchors.

7.3 Time for Anchor Installation

To properly install anchors, the installation time and location must be chosen carefully. The concrete above the construction joint acts as a block piled on the lower dam body. The block curls during cold weather and uncurls in warm spell, as shown in Figure 7.4.

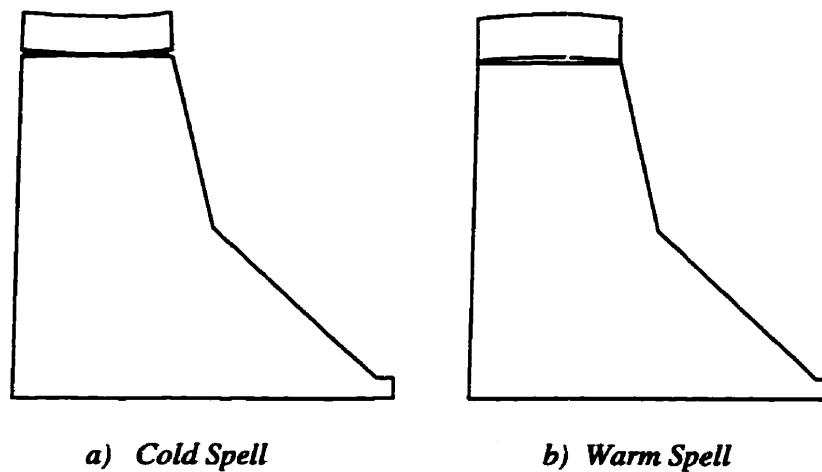


Figure 7.4 *Deformation Pattern of the Dam with Debond Joint*

The time for the anchor installation should be the time when the upper block becomes flat, or when the joint opening displacement is small. The joint opening displacement on the first day of every month from 1993 to 1996 is depicted in Figures 7.5a through 5d. There are two times in every year when the upper block becomes flat and the deformation pattern changes from curl to uncurl stage or vice versa. From Figures 7.5a through 5d, it is found that the first period occurs between March and April, when the upper block is going to change from a curl stage to a uncurl stage. The second time is between September and October when the block is going to curl.

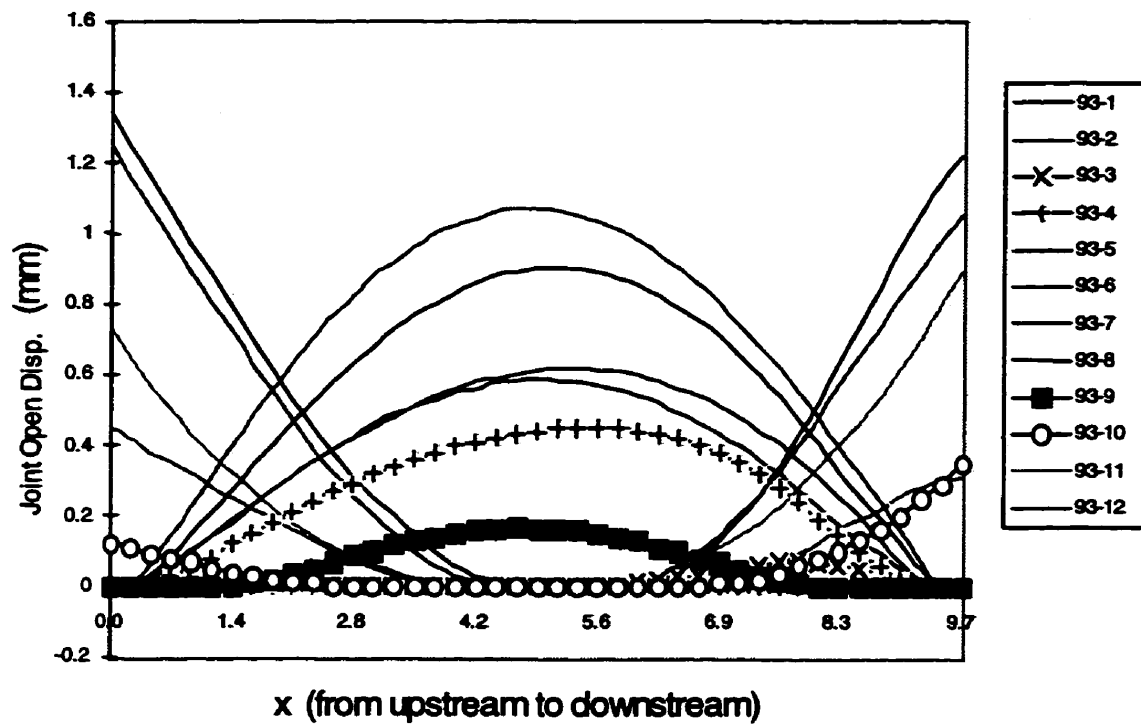


Figure 7.5a Joint Opening Displacement in 1993

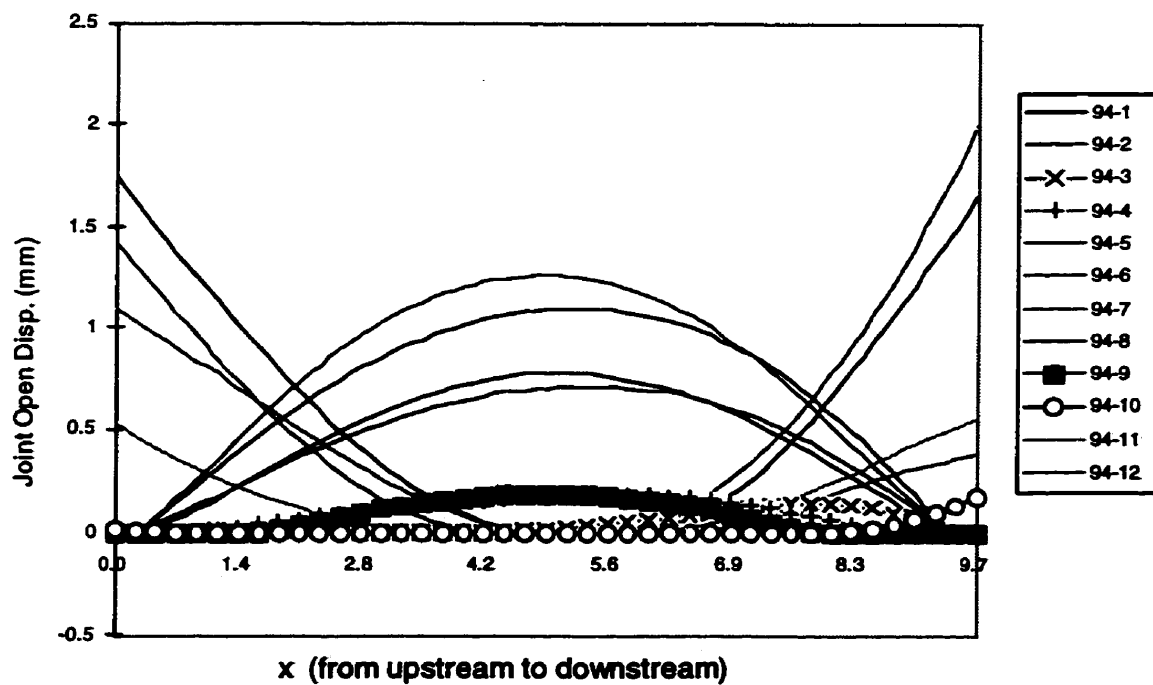


Figure 7.5b Joint Opening Displacement in 1994

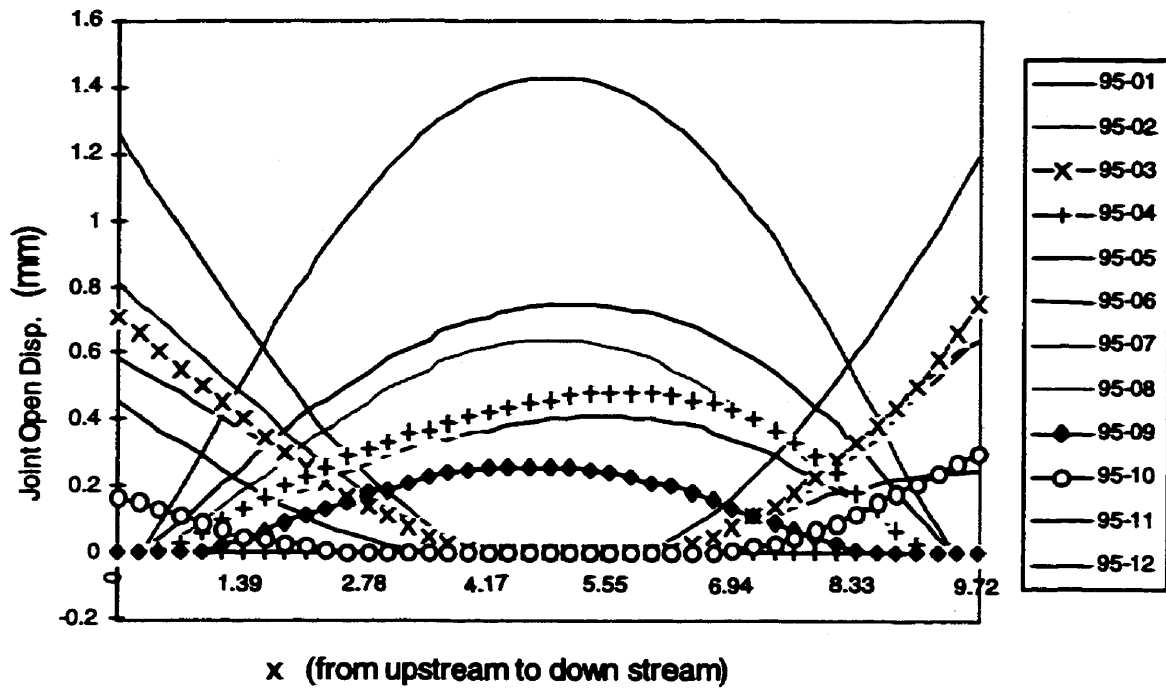


Figure 7.5c Joint Opening Displacement in 1995

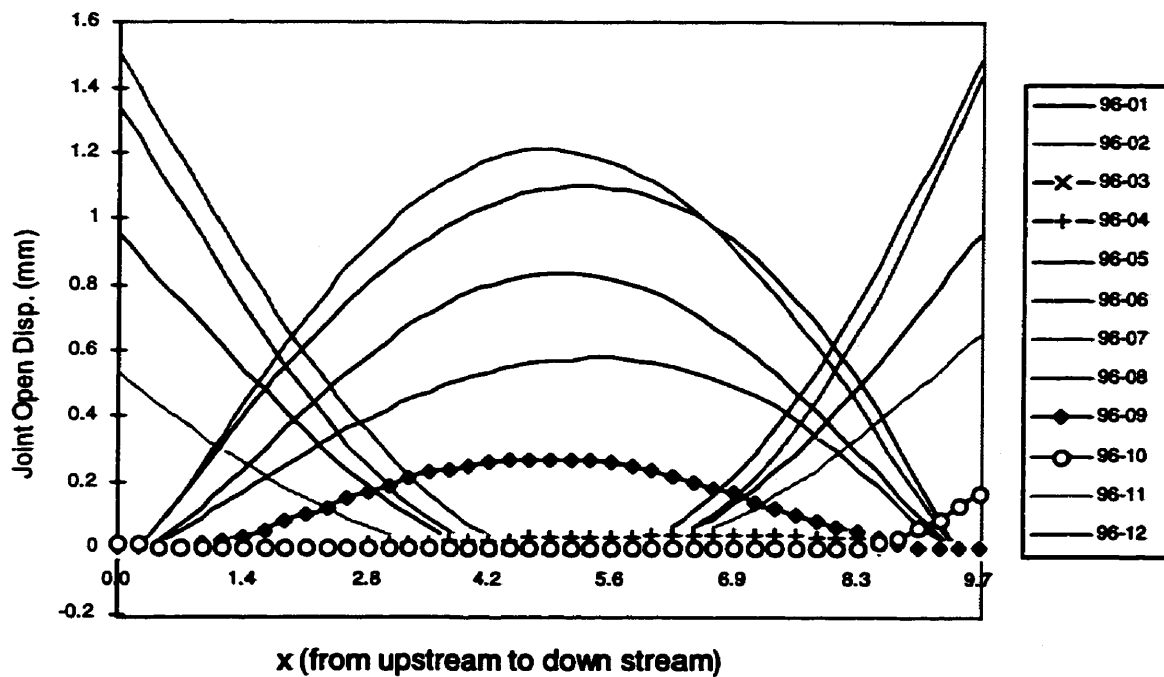


Figure 7.5d Joint Opening Displacement in 1996

The time period between March and April is not the proper time for the installation of anchors. The ambient air temperature during this period is relatively low. A temperature analysis shows that the top part of the dam is in a freezing condition. Besides, there is an ice layer inside the joint, which jacks up the upper block. The predicted displacement itself is questionable because of the ice jacking effect.

The period between September and October is considered to be the most suitable time for dam repair. During this time the weather is still good for outdoor work and the temperature in large part of the dam structure is above 0°C . The whole month of September gives us enough time to install the anchors and for the grout to consolidate.

7.4 Anchor Element

The anchor element consists of several truss elements, as indicated in Figure 7.6.

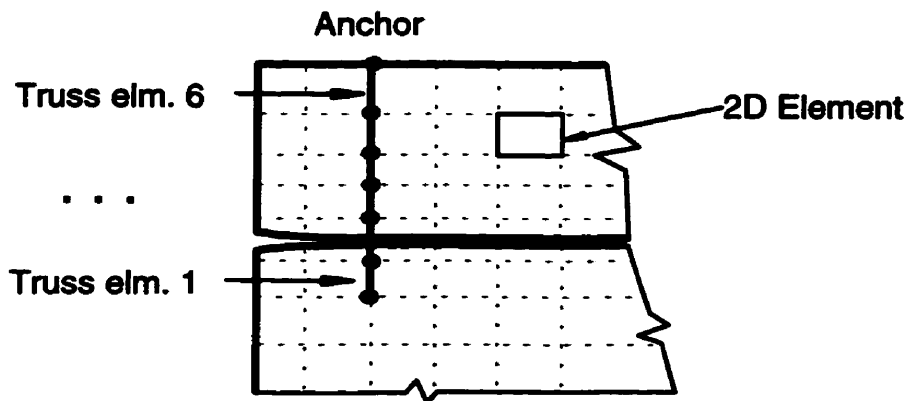


Figure 7.6 Anchor Element

One end of the anchor element is on the dam top, and the other end crosses the joint line inside the lower dam part. It is supposed that the anchor has a good bond with the concrete for a large part. Therefore, adjacent nodes along the anchor line are connected with a truss element inside the dam. The truss element is not connected to the two nodes on the interface.

The consistency relation of the truss element can be written as:

$$l + \alpha(T - T_0) + \frac{Fl}{EA} = (u_2 - u_1) + D \quad (7.1)$$

where

l : the length of the element; α : thermal expansion coefficient;

T_0 : installation temperature; T : temperature of the truss element;

E : Young's modulus; A : cross section area;

F : The axial force.

D is the distance for the pre-stress stretched truss element to meet when the anchor is installed.



Figure 7.7 Truss Element

At the time when the anchor is installed,

$$D - l = \frac{l}{EA} f_p - \delta. \quad (7.2)$$

where f_p is the pre-stress and δ is the relative displacement between the local *node1* and *node2* when the dams deforms under the action of all the thermal, surface pressure, gravity loads and the installation forces. Thereafter, we obtain the relation between the end force F and the displacements u_1 and u_2 as:

$$\frac{EA}{l} [(u_2 - u_1) - [\alpha(T - T_0) + \delta]] + f_p = F \quad (7.3a)$$

or

$$\frac{EA}{l} [(u_2 - u_1)] = F + \left\{ \frac{EA}{l} [\alpha(T - T_0) + \delta] - f_p \right\} \quad (7.3b)$$

The second term on the right hand side of equation (7.3b) forms the additional node force on the truss element ends. For every anchor, the relative displacement δ need be determined first in the analysis. It is worth noting that this relative displacement depends on the installation forces of all the anchors, as well as other loads on the structure.

7.5 Joint Opening Analysis

It has been seen that the thermal stresses cause the upper block to curl and uncurl. Very high interface stresses are needed to hold the deformation if the joint is supposed to be perfect. The bending moment produced by the interface normal stresses is going to overcome the thermal curl and uncurl deformation. Therefore, if we want to fix the entire joint very strong anchors have to be used for this purpose. An alternative way to avoid this situation is to only install anchors in the upstream side, as shown in Figure 7.8.

Anchors can be installed near the upstream side within a distance of one to two meters to keep the joint either closed or with a small opening in this region. Though the entire joint is not fixed in this way, leaking can be reduced or even stopped if the joint opening displacement near the upstream end is well controlled. In this case the anchor forces only need to keep the joint closed. The downstream side can still move freely as indicated in Figure 7.8. It is expected that the anchor forces will be significantly smaller than in the case where it is intended to fix the entire joint.

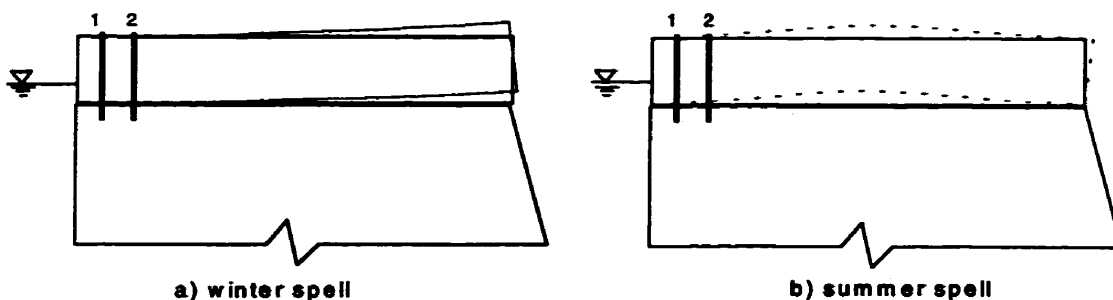


Figure 7.8. Anchors Installation Position

Numerical tests have been carried out to simulate the dam motion with the joint fixed by anchors in the upstream side to study the joint opening displacement and necessary anchor forces. Anchor elements are planted in the finite element model cross the cracked construction joint as sketched in Figure 7.6. Anchors are supposed to have a certain installation pre-stress f_p . Because the finite element model is set to be one meter thick in the dam span direction, therefore, the number of anchors and their pre-stress are refer to one meter distance in the dam span direction.

The double corrosion protected threadbar anchors (DCP anchor) have been used in Manitoba Hydro projects. The diameter of the anchors often used is $1\frac{3}{8}$ inch with an ultimate stress of 150 kips/in^2 . The anchors are tested to a load equal to 189.6 kips (843.4 KN), "locked off" at a load of 165.9 kips (738 KN), and they have a design load of 142.2 kips (632.5 KN). This type of anchor is used in the numerical studies.

Previous displacement and stress analyses show that large opening displacement and high tensile stress zones are from the upstream surface to 2m inside the dam for this end. Therefore, the location for *Anchor 1* is chosen 0.47 m away from the upstream surface. The second anchor location is set to be 1.15 m (case 1) and 1.4m (case 2) away from the upstream surface, respectively. The anchor planted 0.92m deep cross the construction joint inside the dam. In practical analysis, the temperature analysis is performed first from October 1992. After the transient temperature analysis result converges to the real temperature filed distribution, the motion analysis starts from October 1993 to ensure that the installation force is correctly calculated.

Case 1. (Anchor1 at 0.47m, Anchor2 at 1.15m from upstream surface)

As can be seen in Figure 7.8, the anchors supply forces against the bending deformation in the summer spell. It is expected that anchor 2 experiences higher axial deformation than anchor 1 during summer time. Therefore, anchor 1 and 2 are set to have a pre-stress force of *600KN* and *500KN*, respectively. It is found that the anchor axial force is not uniform. Outstanding forces happen in element 2, which crosses the interface, and element 6 on the top of the dam. Figure 7.9 shows the yearly variation of the anchor forces. For *anchor 1*, its axial force drops in the beginning because of the winter shrinkage of concrete. In the summer, the axial force (element 6) goes up to *650KN* about *50KN* higher than the pre-stress force level. Similarly, the axial force of *anchor 2* drops first and increases in summer. In element 2, the force goes up to about *80KN* above its pre-stress level.

With the anchors installed, the joint opening displacements at the anchor locations are plotted in Figure 7.10. The maximum joint opening displacement was recorded near location 1 in 1993-1994 winter and theoretically predicted as *1.9 mm*. The anchor forces the maximum opening displacement down to *0.1 mm*. In the summer span the joint keeps closed. At location 2, the joint is closed in the winter. During the summer, the thermally induced internal force is very strong. Therefore, there is a *0.2~0.3mm* opening displacement even with the anchor force action. In comparison with a *0.5mm* opening displacement before anchor installation, the opening displacement is not reduced much.

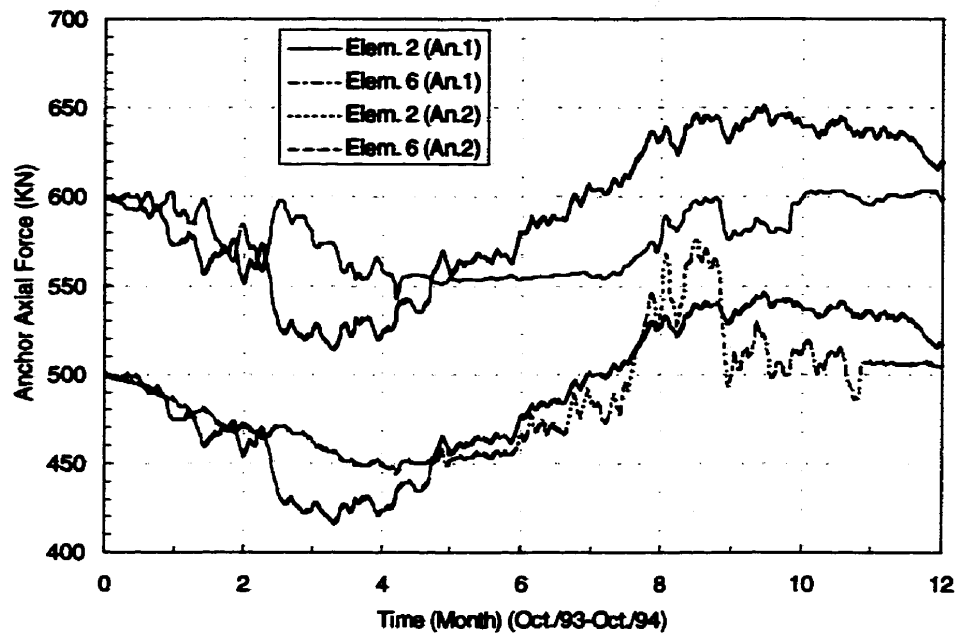


Figure 7.9 Anchor Axial Force (Case 1)

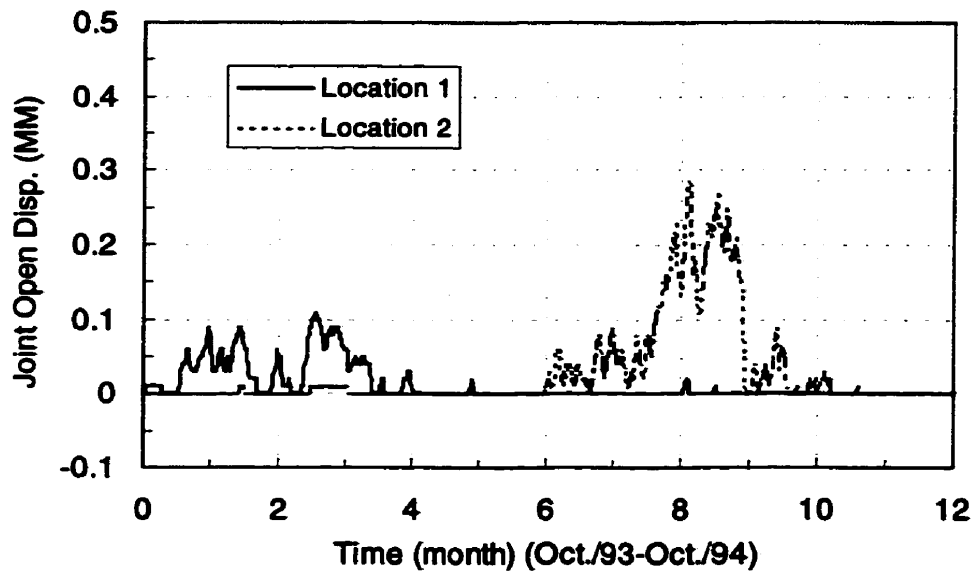


Figure 7.10 Joint Opening Displacement at Anchor Locations (Case 1)

Case 2 (Anchor 1 at 0.47m, Anchor 2 at 1.4m from upstream surface)

In case 2, the anchor has the same location and pre-stress level as case 1. Anchor 2 also has a 500KN pre-stress inside. At the location 2, the joint has a larger opening displacement in the summer; therefore, the anchor axial force is expected to be higher than in case 1. Similarly, the outstanding forces are in element 2 and element 6. Their force values are depicted in Figure 7.11. The situation for anchor 1 is almost the same as in case 1. The maximum axial force increment is about 50KN. However, the axial force inside anchor 2 (elem.2) now has a significant 120KN increment higher than its pre-stress level of 500KN during the summer time. The joint opening displacements at anchor locations of case 2 are plotted in Figure 7.12. It can be seen that the opening displacements are slightly higher than in case 1.

A typical joint opening displacement in winter is shown in Figure 7.13 (case 1). Because the upstream side is fixed, the joint opening displacement in the downstream side is enlarged significantly. The contact zone of the interface is about 2.5m when the downstream side reaches its maximum displacement. The maximum opening displacement in the downstream side was recorded to be 1.7mm for 1993. After the installation of anchors, its value can be 3.8mm.

The extreme joint opening displacement in summer time is shown in Figure 7.14. The joint is closed within half meter from the upstream surface. The opening displacement is 0.2mm at a distance of one meter from the upstream surface. In comparison with the situation without anchors, the displacement curves coincide in the down stream side. The displacement is brought down 0.2mm in the upstream side and the contact zone increases 0.2m by the installation of anchors.

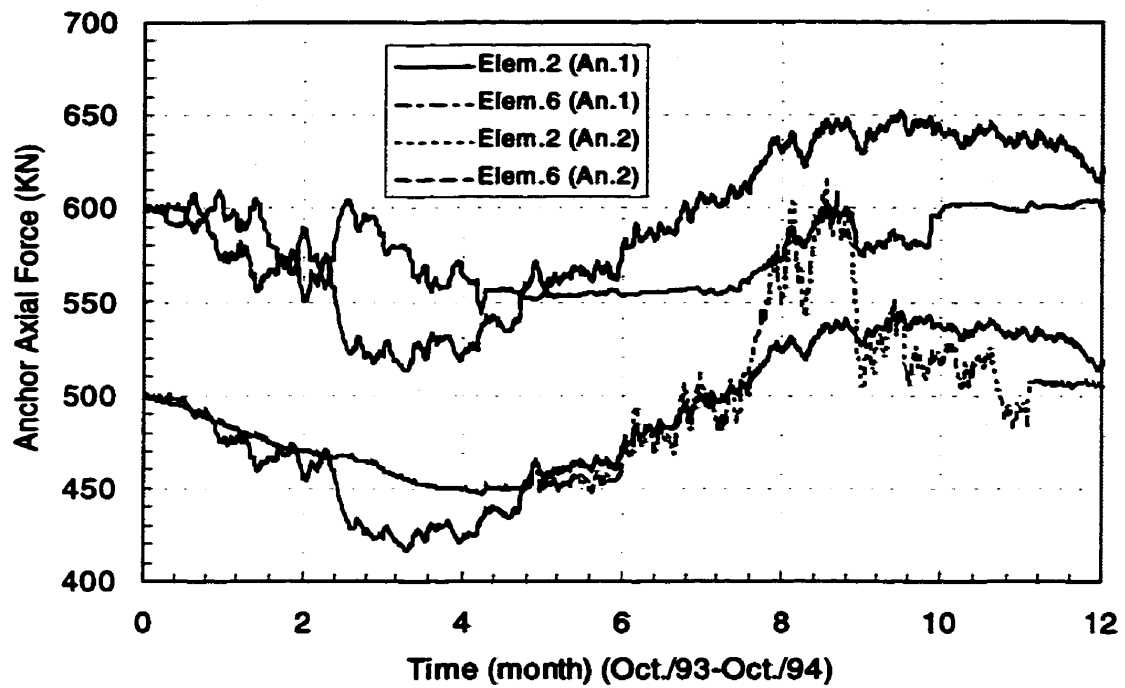


Figure 7.11 Anchor Axial Force (Case 2)

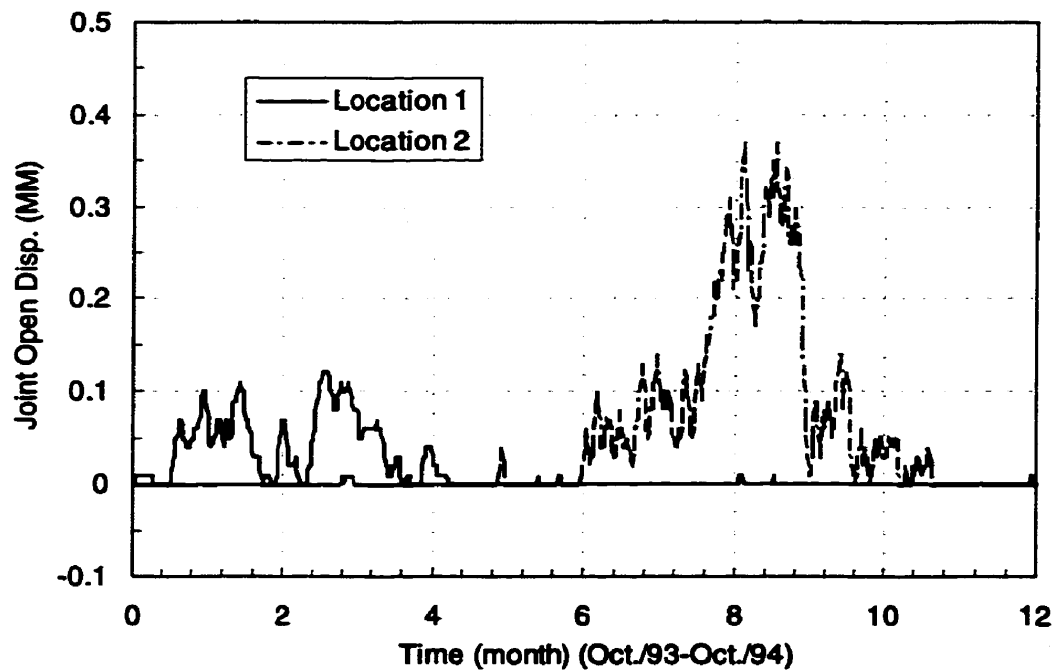


Figure 7.12 Joint Opening Displacement at Anchor Locations (Case 2)

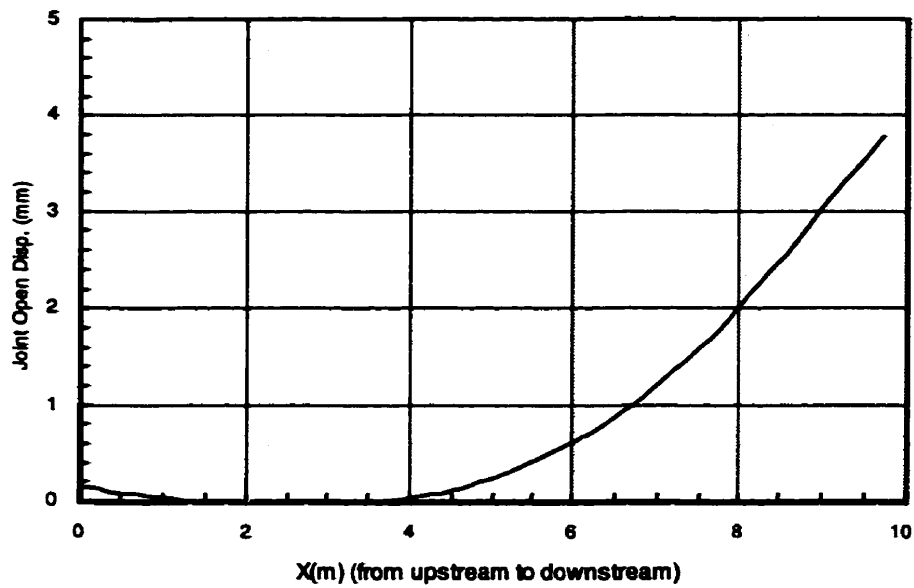


Figure 7.13 *Opening Displacement along Joint Line (Dec 20th, 1993)*

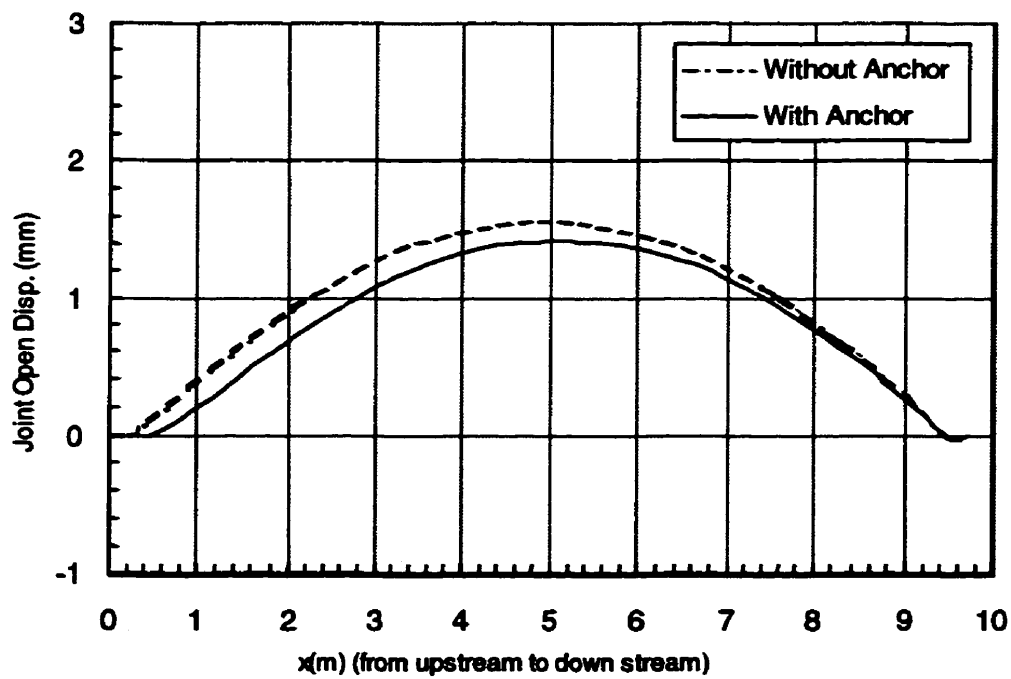


Figure 7.14 *Opening Displacement along Joint Line (June 20th, 1994)*

Case 3.

In case 1 and case 2, there is a zone of about 4m from the upstream side which keeps closed in winter (See Figure 7.13.) However, the joint does not remain close in summer. In summer, the downstream end can't move freely. The action of the anchor forces is actually against the thermally induced bending deformation. The maximum opening displacement at 2m from the upstream side is nearly 0.6 mm. To lower this displacement, another anchor need be installed in the middle of the dam cross section. The third location should be as close as possible to the center of the dam to effectively reduce the opening displacement. In another aspect, this anchor also should not increase the axial forces and opening displacements at other locations in the winter. From Figure 7.13, it can be seen that if the third location is beyond 4m, the anchor will bring up the upstream side opening. Based on these considerations, the third location should be chosen about 4m from the upstream side.

In the numerical analysis, the third location is chosen 4.5m from the upstream side. As can be expected, the anchor in the third location should be stronger than the anchors at other locations. Numerical test shows that the effect of a single DCP anchor is limited. More anchors are needed within one-meter dam span direction. The decrement of the maximum displacement at the point 2m from the upstream side with a different number of anchors is shown in Figure 7.15. It is found that 4 to 6 anchors can bring the maximum displacement to 0.3 mm in the summer time. The joint opening displacement level in a 2 meter zone can be kept lower than 0.3 mm, with the action of the anchors at the third location.

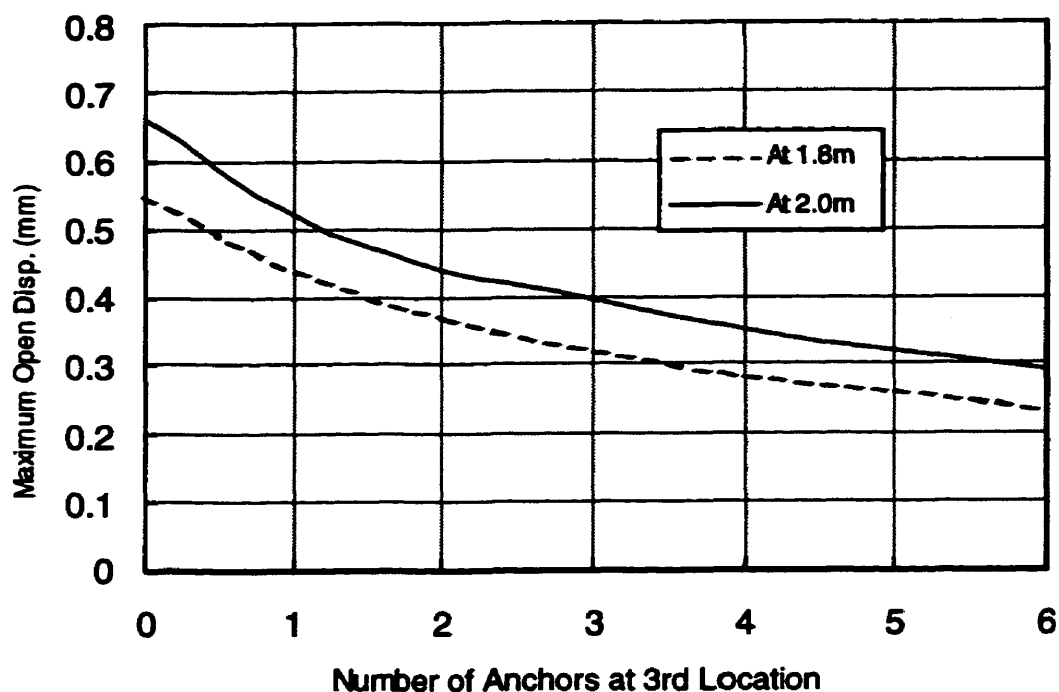


Figure 7.15 *Decrement of the Opening Displacement with the Number of Anchors*
(2m from upstream side)

The anchors at location 3 experience a high level deformation. The pre-stress should be set lower to 100 KN per anchor. Numerical tests show that the pre-stress level has a small influence on the joint opening displacement in the summer. The axial force in one anchor usually increases to 400-500 KN because of the joint opening in summer.

7.6 Conclusion

Both anchors and grout are necessary to fix the cracked construction joint. Although grout is not included in the analysis model, it is quite useful to stop leaking and protect anchor from corrosion. Grout can also supply a certain interface stress to keep the joint closed. Because of the dam thermal deformation character, it is not suitable to fix the entire joint. To overcome the curl and uncurl deformation of the upper block, a very

strong interface is needed. To grout the joint with anchors installed near the upstream side, as studied above, is a feasible strategy for the dam repair.

In winter, anchors installed near the upstream surface keep the upstream side closed. From the upstream surface, there is a zone about $4m$ long in which the joint is almost closed (maximum opening is around $0.1mm$ to $0.2mm$). Even the grout may become brittle in winter, the grout in this zone can be expected to stand a low interface stress, and therefore keeps good bond. Outside this zone to the downstream side, the joint opening displacement is enlarged. It is hard for the grout not to be broken in this region. However, if the grout keeps a good bond in the upstream side, water leaking can be stopped and less ice forms inside the joint.

For the cases studied above, the anchor force and the correspondent joint displacements are smaller in case 1. This case is preferred over case 2. The anchor locations and pre-stress levels can be recommended for practical anchor installation.

In summer, the downstream end can't move freely. The action of the anchor forces can't totally eliminate the thermally induced uncurl deformation. The joint opening displacement is even dangerous to the grout layer. The opening displacement for a large part of the joint can be above 0.6 mm . The effect of the anchors is not obvious. The remaining grout zone (not damaged in winters) is vulnerable. Although the grout material is less brittle in summer, it is not expected that the grout layer can stand a relatively high interface deformation. The length of the grout zone with a good bond might shrink some distance. To keep more of the joint in good bond, the anchors at the third location is helpful. In consideration of the deformation pattern of the upper block in winter, the third location should be about $4m$ away from the upstream surface.

Only within a zone of $2m$ long near the upstream surface, the joint opening displacement can be controlled to a very small level. The grout in this zone is expected to keep bonded for years, which is significant for stopping water leaking. The number of anchors in the middle (at the third location) need be determined in terms of the grout strength. For instance, if the grout layer can stand for a maximum opening displacement of $0.3mm$, 4 to 6 anchors are needed. More anchors may be needed for a lower deformation tolerance.

Chapter 8

Summary and Conclusions

A severe seasonal temperature variation causes serious damage to electrical hydraulic structures in the arctic region. Thermally induced stresses and ice loads are the main cause of structural degradation. Transient changes of the temperature field inside the massive concrete structure results in a high level of tensile stresses near the structure's surface region. Forebay ice loading during winter creates a very large local stress concentration, and may be dangerous for local structural failure. One of the most significant influences of the load combination is the partial or total failure of the construction joint. It can be seen, from the situation of the Long Spruce South Transition structure, that cracks on construction joints may occur after only a few years of service even for a newly built dam. The grout repair of such structures can only stand loads for one or two years.

At the construction joint, a stress of high level usually appears at its ends, where the worst thermal and ices loads act. A theoretical motion analysis of the structure, with or without cracked construction joints, is expected to be instructive for rehabilitation work. In the present study, a set of numerical procedures has been developed to simulate the behavior of a dam structure under thermal and ice load actions, with special emphasis on the interface displacements and stresses on the cracked construction joint. The research is focused on the transient temperature field analysis, the motion analysis method for the structure with a cracked construction joint, ice load and the interface stresses in the joint. The following conclusions are drawn based on the present study

8.1 Transient Temperature Field Analysis of Dam structure

A transient temperature field analysis of dams forms the base of thermally induced motion and stress computations. The temperature analysis method has been well established. For practical analyses, only the initial conditions and boundary conditions may cause problems.

The composition of transient temperature solutions was analyzed through *the mode superposition method*. The important role of the fundamental heat frequencies in the transient heat transfer analysis is emphasized. Generally, an assumed initial temperature field has to be used for temperature analyses of dam structures, i.e., an error has to be introduced in the beginning of the analysis. This study shows that no matter what type of initial condition we assume, after a certain time the error will decrease to a negligible level or the solution will converge to the real solution. The influence of initial conditions on the long-term transient solutions decays exponentially. The convergence time can be well estimated from the fundamental heat frequency of the structure. The traditional data repetition criterion for temperature analyses of dams is not necessary.

The direct integration of the first order semi-discrete finite element equations by using the finite difference method is preferred for the analysis of transient heat transfer processes because of its efficiency. The integration time step length can also be assessed from the fundamental heat frequency. An analysis shows that a daily average or even a weekly average integration scheme is suitable from the numerical stability aspect. The daily average scheme performs well for capturing the thermal response of dams to the variation of the ambient air temperature.

The determination of the convective heat exchange coefficient between the dam and the ambient environment is another concern of the studies. Numerical processes have

been conducted to test the influence of this coefficient to the results of the transient temperature field analysis. The finite element analysis model for the structure of Long Spruce Generating Station South Transition was established. A variety of the combinations of the convective coefficients between air-dam and between water-dam are tested. The frequency analysis and transient temperature analysis prove that the thermal behaviors of the dam structure are not sensitive to changes in the convective coefficients when the coefficients are in certain range. Therefore, the complicated process of determining the heat convective coefficient can be avoided by using less accurate values.

The transient temperature analysis of the Long Spruce Generating Station South Transition was performed over the period of October 1992 to March 1997, based on the structural and ambient air temperature data. At the locations of three in-site thermistors, one near the upstream face, one near the downstream face and one in the center of the dam, the numerical time history of the temperature has very good agreement with the recorded data obtained from the thermistors. The good agreement between the recorded data and computed results shows that the joint causes negligible disturbances on the temperature resulting from a continuous structure model.

This study indicates that the temperature field in the dam structure can be well traced by the finite element method. In addition, the temperature analysis based on simplified FEM model can supply the reliable thermal load needed for the displacement and stress analysis of seasonal variations.

8.2 The structure analysis method of dams with failure construction joints

Debonded construction joint behaves like a crack in the dam structure. As in the case of the Long Spruce South Transition, the crack cuts the dam structure into two

blocks. The upper blocks moves freely with respect to the lower block. Interface stress and contact analyses are involved for the dam motion driven by thermal and ice loads. A new interface stress evaluation method is first developed in this study to form a good base of an algorithm for a contact analysis.

In the new interface stress evaluation method, the starting point is the virtual work principle instead of the traditional Gaussian point stresses. The stress evaluation equations are derived from an integration form of the virtual work principle. No differentiation step is involved during the stress evaluation process. Therefore, no more numerical errors will be introduced to the final stress results. The accuracy of the stress values mainly depends on the element nodal equivalent forces. Numerical results show that the Newton-Cotes integration strategy is more suitable for this stress evaluation method than the Gauss integration strategy. The stress evaluation process is finally made independent of the choice of interpolation pattern between nodes with the Newton-Cotes integration. Better quality interface stress values are yielded by this new method.

The motion analysis of the dam with through-thick debonded joints is the main topic of this study. The main difficulty for this sort of problem is eventually the contact between two surfaces of joints. The evaluation of interface stresses plays a crucial role in this analysis. Especially, the stress is needed at the nodal points for the determination the opening/closing state of the interface. In many classical methods, the interface stresses are evaluated by the product of spurious interface penetration and the interface stiffness, which is actually the penalty factor. The quality of the interface stress is questionable. With the new interface stress evaluation method, the algorithm of contact analysis is developed for this study. The finite element formulation of an interface element is

presented. The iteration process and the interfacial constitutive relations are designed for this interface element. The interface stiffness is adjusted by the interface stress and a progressive interface stiffness release process results in the contact zone and interface stresses as well. A model test shows that very smooth stress and displacement distributions can be solved. The new algorithm serves as the practical numerical analysis method of cracked dam structures.

8.3 Ice Load Analysis of Dam Structures

The forebay ice sheet in winter exerts large forces on the dam and causes high local stresses beside thermal load. Ice loads are believed to be another major factor to cause damage in dam structures. To obtain accurate ice loads on the structures can be an extremely difficult task. Certain load data can be obtained by field measurement. However, ice loads are strongly environmental dependent. The measurement in one place can't be used for other places. Many environmental factors such as the air temperature, snow cover, solar radiation, wind drag, water flow and wave can influence the ice formation and ensure ice loads. It is the air temperature variation that plays the most important role. Attempts were made to develop several models to relate the ice thickness, vertical load and horizontal load with the ambient air temperature. The widely accepted freezing-degree-day method is used to estimate the ice thickness change. The predicted ice thickness coincides well with available field measurements.

An ice sheet, moving due to water waves, exerts a vertical load on the dam structure. The ice sheet is considered as a thin plate on an elastic foundation with one end fixed. Two cracks can be predicted to appear because of the high bending moment inside the ice plate. One crack appears at the ice plate root in the early stage of freezing and the

other appears later at a certain distance from the dam structure. The cracks act as hinges and release much of the bending moment of the ice plate. The vertical ice load is calculated by this two-crack-model and is set up as a function of the ambient air temperature and water level fluctuation.

More unknown factors are going to be faced for the horizontal load analysis than the vertical load analysis. The zero total strain assumption is adopted here to avoid the complicated boundary constraint conditions of the forebay bank. Only two major factors, the ice temperature and the creep behavior are taken into account. One dimensional heat transfer processes have been solved to relate the temperature of the snow covered ice with the air temperature. The horizontal stress inside the ice sheet is obtained by the numerical integration of a nonlinear rheological equation. The integration of distributed ice stress over the ice thickness forms the horizontal ice load. Because a rigid boundary condition of an ice sheet has been used, the horizontal load by this model is overestimated. We are not able to consider the wind and water drag effect. Therefore the overestimation is expected not to be too much higher than the real load.

8.4 Practical Motion Analysis of Cracked Dam

As a typical dam structure with a cracked joint, thermally induced motion of the Long Spruce South Transition Structure was analyzed with the newly developed computational method. In order to separately study the thermally induced motion and the effect of ice on the structure, a model without any consideration of ice is first analyzed. Although the field measurement of the joint opening displacement shows a three dimensional deformation pattern, a two dimensional plane strain model can still predict the displacement variation of the cracked joint.

A comparison between the numerical results and the recorded data show the significant influence of ice on the motion of dam structure in the winter spell. It can be estimated that 50-60% of the displacement peak value comes from the contribution of the thermally induced displacement.

The theoretically estimated ice loads are applied to the Long Spruce structure model. The numerical results of these loads turn out to agree much better with the recorded data than those of the no ice model. The pre-peak behavior of the construction joint opening displacement at upstream and downstream side locations can be predicted very well by the finite element method. The predicted time and value of the peak open displacement are quite close to the recorded values for every year between 1992 and 1997. The no ice model, in which only the thermal load influence is included, predicts about 60% of the peak displacement. It is the ice loads that bring the numerical peak displacement close to the measured values. This verifies that the ice loads cause nearly 40 % of the peak values. Therefore the ice loads are a must-be-considered factor for any stress and rehabilitation analysis of this kind of dam structures.

The influence of the vertical and horizontal ice loads is examined separately. The results show that the numerical joint opening displacement curves, by applying vertical load only and by applying both vertical and horizontal loads, are very close. With the application of even an overestimated horizontal load, the displacements do not change significantly. Actually the effect of the horizontal load on the joint open displacement can be of negligible order. The vertical load acts at the position very close to the construction joint. It causes a tensile normal stress on the joint interface. Meanwhile the horizontal load causes more shear stress on the interface. Along with the thermally induced stress, the vertical load is much more harmful to the joint interface.

The ice jacking effect only influences the post-peak behavior of the opening displacements. Therefore, it is considered to be less important to the winter deformation analysis of the dam structure.

For the purpose of an initial joint damage analysis and the rehabilitation analyses, the dam is assumed to have a perfect joint under the action of all kind of loads. The interface stress is calculated. It is found that high tensile stress zones of about $1.5\sim 2m$ long exist near the upstream side and downstream side ends of the construction joint in winter. The highest tensile stress value in the downstream side is as high as 7 MPa , and in the upstream side 4 MPa . For this tensile stress level, it is quite easy for the interface to peel. If the leaking joint is going to be fixed, the interface must be strong enough to stand the above stress level. With the stress data known, it will be easier to choose a feasible rehabilitation strategy.

8.5 On the Anchor Fixation

The repair strategy of dams with cracked construction joints is investigated. The simple grout fixation strategy is practically proved unsuitable for dams in cold weather. Because of the high interface stress, anchors are needed to fix the cracked dams. The anchor installation scheme is numerically studied. Choices of anchor location, strength and pre-stress level, and installation time have been tested using the finite element model.

Because of the dam thermal deformation character, it is not suitable to fix the entire joint. To entirely fix the cracked construction joint, many anchors have to be used over the whole dam cross section. The force supplied by the anchors is going to overcome the thermally induced bending deformation of the upper block. High tensile

stresses will appear in the upper and lower surface of the block in winter and summer respectively, and they may create vertical cracks in the middle of the upper block. To install anchors only near the upstream side is a suggested strategy for the purpose of stopping the dam leak. An analysis model is set up with anchors simulated by bar elements.

The measured seasonal displacement pattern of the dam shows two transformations a year from the curl to uncurl shape. The month of September is the best time to install the anchor, when the upper block is relatively flat. After several trials, two rows of anchors are suggested to be laid out near the upstream side. The anchor pre-stress level was chosen based on available anchor properties. Both anchors and grout are necessary to fix the cracked construction joint. Although grout stresses are not taken into account in the analysis model, it is useful to stop leaking and protect the anchors from corrosion. Grout can also supply a certain interface stress to keep the joint from closing.

In winters, the anchors can keep the upstream side closed. From the upstream surface, there is a zone about *4m* long where the joint is almost closed. The grout in this zone can be expected to experience low interface stress, and therefore the joint keeps in good bond condition. Outside this zone to the down stream side, the joint open displacement is large. It is hard for the grout not to be broken in this region. However, if the grout keeps a good bond in the upstream side, water leaking can be stopped and less ice forms inside the joint.

In summer, the downstream end can't move freely. The central part of the upper block is going to move up. The action of the anchor forces is actually against the

thermally induced bending deformation. It is not expected that the anchor forces can entirely eliminate the bending deformation. The remaining grout zone (not damaged in winters) is vulnerable. The size of the grout zone with good bond might shrink. The highest opening displacement level within 2m from the upstream surface is about *0.6mm* without the anchors in the dam center. To lower this displacement level, a few anchors have to be installed at the central location. The number of anchors actually depends on the open deformation tolerance of the grout layer. The joint opening displacement level within a zone of 2 meter long near the upstream side can be controlled to a certain low level with the action of anchors. In this zone, the joint is closed in winter. Even the grout material is more brittle, the bond will not be damaged. The joint open level in summer is controlled by the anchors. Therefore, it is expected that grout in this zone will keep its bond for long time. This zone is significant for keeping the dam from leaking.

8.5 Recommendation for future work

Design and rehabilitation of hydroelectric structures in the north is intimately tied to the repeat thermal and ice loads on the structures. These loads cause stresses of a very high level and structure cracks, especially at construction joints. Chemical or cemetery grout is not strong enough to fix the cracked construction joints. Therefore, high strength anchors have to be installed. For newly designed dams, it is recommended that vertical anchors should be embedded cross the construction joint closest to the water level to prevent the joint from cracking. The scheme of the anchor installation and its effect need be studied.

Dam damage and cracks are significantly influenced by ice loads. The vertical ice load is one of the major forces to drive the construction joint cracking. It should be of equally concern to the horizontal ice load in dam design and rehabilitation. Meteorology, snow and ice information need be collected for a long period at the structure site to supply necessary data for the analysis of ice loads.

Thermal isolation on the dam surface can change the heat influx to the dam body and reduce the temperature gradient. The effect of various isolation strategies, such isolation layer and surface painting, need be studied numerically and experimentally. An active adjustment of the dam surface temperature can be very meaningful for research and engineering practice.

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