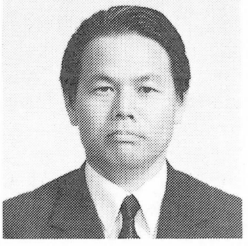
CONCRETE LIBRARY OF JSCE NO. 7, JULY 1986

THERMAL STRESS OF A CONCRETE DAM AT HEIGHTENING CONSTRUCTION

(Reprint from Transactions of JSCE, Vol. 337, Sep., 1983)



Tada-aki Tanabe　　Akira HARAGUCHI　　Toshihisa UCHIDA

SYNOPSIS

Kuroda Dam was heightened by the amount of 10.2 m. The reduction of thermal stress especially at the surface of upstream side of the old embankment due to the hydration heat of the new embankment was the main engineering objective to keep the overall safety of the integrated embankments.
An analytical method to simulate the thermal behavior of the heightaned dam was developed and applied successfully to achieve the objective. The observation of the temperatures and the strains of various portions of the embankment were started with the start of construction in July, 1977 and is continued untill now. The data during the period of early two and half years were reported in comparison with the calculated values in this report.
The influential line method for the thermal stress calculation was proposed and methods of reduction of thermal stress was discussed.

Tada-aki TANABE is a professor of Civil Engineering Department, Nagoya University. He is the chairman of JCI Committee on the Thermal Stress of Massive Concrete Structures besides a member of various committees of both JCI and JSCE. His main research is directed to the dynamic failure mechanism of RC structures besides the thermal stress.

Akira HARAGUCHI is the Head of Abiko Office of The Computation Center of Electric Power Industry. His research interest includes the thermal stress analysis of massive concrete structures and concrete material properties of young ages. He is a member of JCI Committee on Massive Concrete and also a member of JSCE.

Toshihisa UCHIDA is the Director of The Hydraulic Electric Power Divison of Chubu Electric Power Company. He has been the Vice Director of The Construction Office of Okuyahagi Hydraulic Pumping Up Power Station and engaged in the supervision of all the engineering projects concerned on the constraction. He is currently the executive board member of JSCE and also a member of numerous committees in civil engineering projects in Chubu Area in Japan.

1. INTRODUCTION

In recent years, heightening constructions of existing dams are increasing in Japan due to the increased demand for water supply and electric power supply as well as due to increased flood water levels of rivers. One of the most important engineering factors in the construction is the control of thermal stress which arises at the upstream face of an existing embankment due to the cement hydration heat. In other words, an old embankment which is in thermally stabilized condition recieves on the downstream face a large amount of thermal load at the placement of new concrete which volume is from several tens percents to equivalent of the volume of an old embankment.

The thermal load naturally gives rise to thermal deformation in the unified embankment and their stress field is affected by varying rigidity of the new embankment.

Thermal load effect remains until a dam is stabilized thermally, period of which is more than ten years depending on its scale. Besides, fluctuation of ambient temperature influences the stress intensity as well. If its intensity is larger than its tensile strength, cracks initiate. Hence, mix propotion of concrete, dimensions of a block and a curing method must be decided in such a way that thermal stresses be sufficiently controlled. Once the cracks initiate at upstream face, they give rise to far higher water pressure inside of an old embankment than the design presure and give undesirable effects on the stability of a dam. Besides above mentioned factors, concrete strength at the joint must be maintained against normal separation and parallel slippage so that unification of a new and an old embankment is assured.

Examples in the past of heightening constructions of dams are Balk Dam, Mansfield Dam in the United States [1], Ohdomari Dam of Chugoku Electric Power Company [2] and Sakurayama Dam of Fuji Steel Manufacturing Company in Japan [3], [4].
The most extensive study on thermal stress was carried out for Ohdomari Dam, the height of which was raised by the amount of 10.5 meters from 34.8 meters (2). Numerical investigation was carried out assuming that Young's Modulus of concrete of the new embankment and of the old embankment was equal and that the temperature of the new embankment drops instantaneously to the stabilized value from the temperature obtained at the concrete age of 28 days. The maximum stress calculated at the upstream face was 8.0 kg/cm^2. Due to its comparativly high intensity, the adoption of pipe cooling was decided. This example shows that the accuracy of the tensile stress intensity in the calculation is needed to great extent as concrete tensile strength capacity is very limited. In view of the fact, the thermal stress analysis code is developed that can consider the following factors which are important but are neglected in the past investigations, i.e.,

(1) lifting up schedule of a new embankment,
(2) varying rigidity of new concrete in each lift,
(3) varying rate of heat generation in new concrete,
(4) variation of ambient temperature, and
(5) creep of concrete which is affected by its age.

Using the developed code, the thermal stress analysis was carried out for the Kuroda Dam of Chubu Electric Power Company and basing on its results, the heightening construction started in July 1977 and ended in July 1978 [5]. Various observations of the dam behavior started at the same time with the placement of concrete of the first lift and are continued until now. Theses data were compared with the theoretical values.

It should be mentioned that though the observation of the behaviour of a dam is commonly done, the comparison with the analytical values are rather rare, specially in regards to the thermal behaviour due to cement hydration heat. In this paper, the compared results are discussed and the factors influencing thermal stress at heightening construction such as concrete mix, lifting up schedule and so on are evaluated.

2. THE METHOD OF ANALYSIS

For the simulation, the FEM program was developed. The code comprised of three different parts. The first part is unsteady heat flow analysis, the second part is the stress analysis and the last part is the creep analysis, all by incremental form.

The two dimensional Laplace's diffusion equation is written as

$$\frac{\partial}{\partial x}\left(kx\frac{\partial\theta}{\partial x}\right)+\frac{\partial}{\partial y}\left(ky\frac{\partial\theta}{\partial y}\right)+Q(t,\theta)=\rho C\frac{\partial\theta}{\partial t} \qquad ------(1)$$

where kx,ky = thermal conduction coefficients to X and Y direction respectively and C,ρ,Q = specific heat, specific gravity and heat generation rate of concrete respectively. Though cement hydration heat rate is not only time dependent but also temperature dependent, we use the functional of the following form considering that $\left(\frac{\partial\theta}{\partial t}\right)$ is constant at specific time interval,

$$\pi=\iint\left[\frac{1}{2}\left\{kx\left(\frac{\partial\theta}{\partial x}\right)^2+ky\left(\frac{\partial\theta}{\partial t}\right)^2\right\}-\left(Q-\rho\frac{\partial\theta}{\partial t}\right)\theta\right]dxdy+\int_s q\theta ds+\int_s\frac{1}{2}h(\theta-\theta_\infty)ds \qquad --(2)$$

where q, θ_∞, and h are heat flux at a boundary, ambient temperature and heat convection coefficient respectively.
By adopting appropriate shape functions, Eq.(2) is reduced to the following simultaneous algebraic equations,

$$[H]\{\theta\}+[P]\left\{\frac{\partial\theta}{\partial t}\right\}+\{F\}=0 \qquad -----(3)$$

where [H], [P] and {F} matrices are heat conduction matrix, thermal gradient matrix and thermal load vector respectively.
Wilson and Clough's finite difference scheme to solve Eq.(3) is written as

$$\left([H]+\frac{2}{\Delta t}[P]\right)\{\theta t\}=[P]\left(\frac{2}{\Delta t}\{\theta\}_{t-\Delta t}+\left\{\frac{\partial\theta}{\partial t}\right\}_{t-\Delta t}\right)-\{F\}_t \qquad -----(4)$$

Each component of $\{F\}_t$ matrix is written as the sum of the contributions from all elements conected to a node,

$$\{F_i\}=\sum_j\int Q(t)N_j dV \qquad -----(5)$$

where Q(t) should be evaluated at each time step. Once the temperature distribution is obtained by the above mentioned process, stresses can be obtained treating the free thermal strain as initial strain. However it should be recognized that Young's Modulus of concrete varies at each time step as well. Hence, the stress strain relation is written as

$$\{d\varepsilon_e\} = \{d\varepsilon\} - \{d\varepsilon_T\} - \{d\varepsilon_c\} = \frac{1}{E(t)} \begin{bmatrix} 1 - \nu(t) & 0 & \\ -\nu(t) & 1 & 0 \\ 0 & 0 & 2\{1+\nu(t)\} \end{bmatrix} \{d\sigma\} \qquad ----- (6)$$

Where $\{\varepsilon_e\}$, $\{\varepsilon_t\}$, $\{\varepsilon_T\}$ and $\{\varepsilon_c\}$ denote elastic strain, total strain, thermal strain and creep strain respectively. Thermal strain is given as

$$\{\varepsilon_T\} = [\alpha\Delta T, \alpha\Delta T, 0]^T \qquad ----- (7)$$

where α is the thermal expansion coefficient of concrete, for which no time dependency is assumed. As for the creep law, we adopted the strain hardening law for the field where both stress and temperature varies at each time step. The unit creep strain function is assumed to be the product of a logarithmic function in terms of loaded period and stress intensity and a linear function of temperature,

$$\begin{aligned} \varphi(\sigma,T,\tau) &= \sigma\cdot(AT + B)\{C(\tau) + D(\tau)\ln(t + 1)\} \\ &= \sigma\cdot(AT + B)E(t) \end{aligned} \qquad ----- (8)$$

where σ,T,τ = stress intensity, temperature and loading age. A,B,C and D are constants which are to be experimentally decided.

With all the afore-mentioned method, it is possible to carry out detailed thermal stress analysis for versatile mass concrete constructions as far as a structure is idealized to be in plane stress or plane strain condition.

However, the FEM analysis is not suited for the parametric study which carries out similar caluculations changing one or two parameters consecutively as in the case of deciding the most appropriate mix proportion of concrete or most suitable lifting up schedule

For this purpose, we developed the efficient calculation method which is called the thermal influential line method that follows.

Thermal stress increment during the i-step interval of time is written as considering the variation of rigidity and temperature,

$$\{\Delta\sigma_i\} = [D_i]([B]\{\Delta U_i\} - \{\Delta\varepsilon_{o,i}\}) \qquad ----- (9)$$

where $[D_i]$, $\{\Delta U_i\}$ and $\{\Delta\varepsilon_{o,i}\}$ denote the stress-strain relation matrix, the incremental displacement vector and the initial strain vector respectively.

The existing stress intensity at certain time point is written as the summation of each stress increment

$$\{\sigma\} = \Sigma[D_i]([B][K_i]^{-1}\{\Delta F_i\} - \{\Delta\varepsilon_{o,i}\}) \qquad ----- (10)$$

In Eq.(10), $[K_i]$ and $\{\Delta F_i\}$ denote stiffness matrix and thermal load vector respectively at i-th interval of time. Total stress intensity is written in the following form as the sum of the stress that has occurred during the period where concrete stiffness is in the process of hardening and the stress that has occurred after the hardening of concrete is practically finished,

$$\begin{aligned} \{\sigma\} &= \sum_i([D_i][B][K_i]^{-1}\{\Delta F_i\} - [D_i]\{\Delta\varepsilon_{o,i}\}) + ([D][B][K]^{-1}\{\Delta F\} - [D][\{\varepsilon_0\}) \\ &= \{\sigma_I\} + \{\sigma_{II}\} \end{aligned} \qquad ----- (11)$$

where

$$\{\sigma_I\} = \sum_i([D_i][B][K_i]^{-1}\{\Delta F_i\} - [D_i]\{\Delta\varepsilon_{o,i}\}) \qquad ----- (12,a)$$

$$\{\sigma_{II}\} = [D][B][K]^{-1}\{\Delta F\} - [D]\{\varepsilon_o\} \qquad ----- (12,b)$$

Though we need to solve the stiffness equation at each time step to obtain $\{\sigma_I\}$ as $[K_i]$ varies in each time increment, we can obtain the values of $\{\sigma_{II}\}$ in the following simple way.
Firstly, the thermal load vector when j-th element undergoes unit temperature drop is written as

$$\{F\}_j^e = -\int_{vj}[B]^T[D]\{\alpha\}dV \qquad ----- (13)$$

Hence, the stress which occures under the load in the other elements can be expressed as

$$\{\sigma_{II}\}_j = [D][B][K]^{-1}\begin{Bmatrix} 0 \\ \{F\}_j^e \\ 0 \end{Bmatrix} \qquad ----- (14)$$

Consecutively changing the element one after another where unit temperature drop is taken place to all the elements in a structure, σ_{II} is calculated. Thus calculated σ_{II} values in the specific element shows the degree of the influence of unit temperature drop of the other part of a stracture to the stress of that specific element. The connection of this stress intensity forms a line, which is called a thermal influential line. Once, the influential line is obtained and temperature variation within a structure is obtained, it is easy to calculate the thermal stress due to that temperature variation as it is given by the summation of the products of the intensities of the influential line and the temperature variations of all the elements in a structure.
As for σ_I, a similar influential line is obtained, at each time step if necessary, during which the relative stiffness within a structure is assumed constant. Critical temperature difference which is also necessary to apply thermal influential line method is defined as the temperature difference between the maximum and the stabilised to obtain the ultimate thermal stress and as the temperature difference between the maximum and the temperature at specific time of a year for instance the temperature at the coldeset time of the winter of the first year after the completion of heightening construction.
Hence, there are several critical differential temperatures according to the number of peak points of the stress fluctuation curves. For example, the peak stress occurs in every winter at the upstream surface and in every summer at the core of an embankment due to the ambient temperature fluctuation.

3. APPLICATION OF THE METHOD TO KURODA DAM

3.1 Material Constants

The application of the method was tried for the Kuroda Dam which is the upper reservoir of Okuyahagi Pumping up Electric Power Station of Chubu Electric Power Company.

Kuroda Dam was constructed in 1934 at the very upstream of Yahagi River at the elevation of 850 meter above the sea level. It was planned that the existing 34.8 meter tall embankment is heightened about 10.2 meters casting new concrete of the same volume of the existing embankment as shown in Fig. 1. The detailed thermal stress investigation was carried out. The initial and boundary conditions and the material constants are decided in the following way.
(1) The initial temperature distribution of the embankment is taken to be the

values of the time just prior to the placement of concrete of the first lift of the new embankment that is calculated by the unsteady heat flow analysis of the embankment under the varying ambient temperature. Thus obtained initial temperature is shown in Fig. 2.

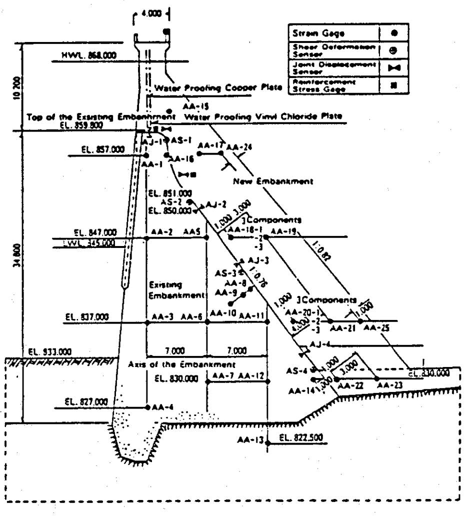


Fig. 1 Kuroda Dam

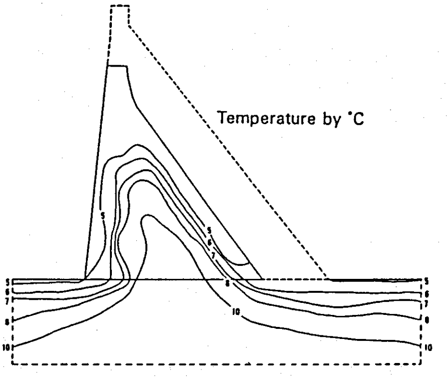


Fig. 2 The Temp. Distribution at The Time prior to The First Placement of Concrete

(2) Material constants such as the compressive strength, the tensile strength and Young's modulus of the concrete of the old embankment was determined from the core specimens sampled by boring [7]. However, the material constants of foundation rock was estimated from the condition of the existing rock mass. These values are shown in Table 1.

Table - 1 Parameters for The Rock Foundation and The Old Embankment

	Rock Foundation	Old Embankment
Thermal Expansion Coefficient α ($\times 10^{-6}$/°C)	9.6	10.8
Thermal Conduction Coefficient K (Kcal/m·h·°C)	2.16	2.0
Specific Heat C (Kcal/kg·°C)	0.24	0.247
Specific Gravity ρ (kg/m^3)	2593	2343
Thermal Convection Coefficient (Kcal/m^2·h·°C)	10	10
Poisson's Ratio	0.3	0.17
Young's Modulus (kg/cm^2)	50000	109000

(3) Thermal constants of the new concrete, i.e., coefficients of coduction, specific heat, coefficients of convection and heat generation rates of each mixes are experimentally decided. The results are shown in Table 2.

Table - 2 Thermal Coefficients for the New Embankment

	C = 220 kg/m³	C = 170 kg/m³	C = 148, F = 37 kg/m³
Thermal Expansion Coefficient α ($\times 10^{-6}$/°C)	9.5	9.6	9.6
Thermal Conduction Coefficient K (Kcal/m·h·°C)	2.4	2.4	2.4
Specific Heat C (Kcal/kg·°C)	0.267	0.267	0.267
Specific Gravity ρ (kg/m³)	2417	2417	2389
Adiabatic Temperature Rise (Kcal/m³)	$Q = 16.07 \times (1-e^{-0.3t-0.453})$	$Q = 12.42 \times (1-e^{-0.3t-0.453})$	$Q = 10.8 \times (1-e^{-0.3t-0.453})$
Thermal Convection Coefficient (Kcal/m²·h·°C)	10	10	10
Poisson's Ratio	0.17	0.17	0.17

(4) Creep functions of new concrete was also experimentally decided and shown in Fig. 3 [6].

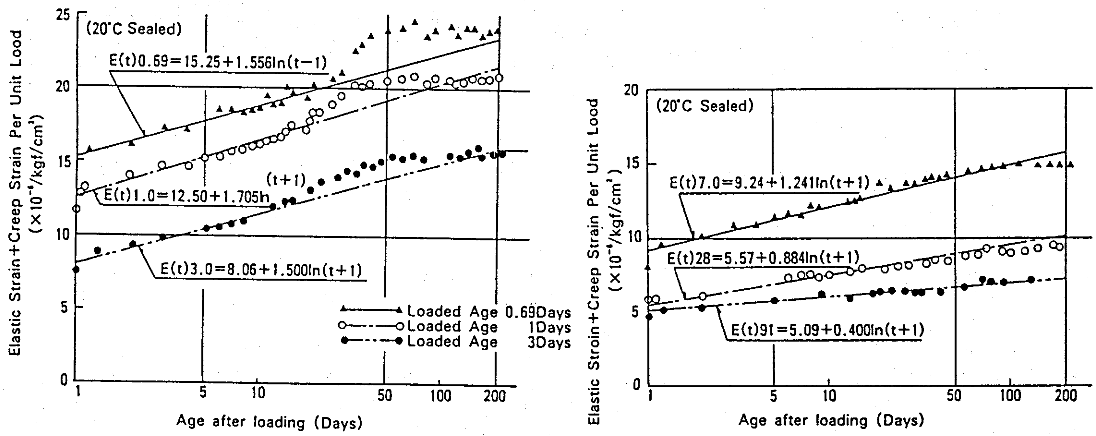


Fig. 3 Creep Function experimentally decided

The Young's modulus of new concrete under varying curing temperature was decided in such a way that the relationship of Young's modulus and compressive strength experimentally obtained is satisfied in which the compressive strength of new concrete under varying curing temperature is decided using maturity factor which is defined as

$$M = \int_0^t \theta(t)dt \qquad \text{- - - - - (15)}$$

In using Eq.15, the curing temperature was assumed to be the same as the adiabatically obtained temperature. So obtained Young's modulus is shown in Fig. 4 (8).

With these initial conditions and material constants, numerical simulations were carried out and the optimal lifting up schedule, the mix proportion and jointing

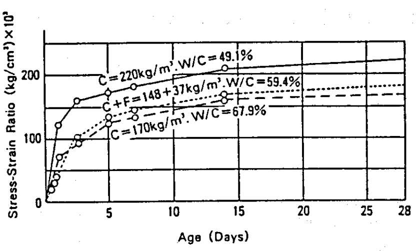


Fig. 4 Relation of Young's Modulus and Concrete Age

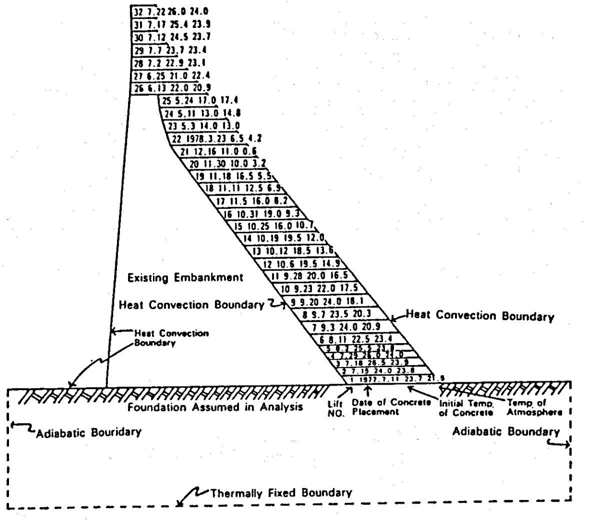


Fig. 5 Lifting up Schedule, Temp. of Concrete at placement and The Temp. of Atmosphere

method were decided. No necessity of cooling system was judged and the lifting up schedule shown in Fig. 5 was finally decided. The construction started in July 1977 and completed in July 1987 with a pause in placement of concrete in the winter.

Measurements of temperatures, strains and displacements started with the placement of concrete in the first lift and are continued until now. In this paper, the results of the former two years are reported.

3.2 Sensors and Its Locations

The block where various sensors were buried in is the central block which rise up from the lowest elevation of the site. The dimensions of the horizontal plan of the block is 10 meters to the dam axis direction and 11 meters to the river flow direction. Twenty five Carlson type strain meters, 23 of which are one component strain meters and two of which are three component strain meters, 4 shear displacement sensor, 4 joint displacement sensor and two steel strain meters were buried in.

The joint displacement sensors and the shear displacement sensors are buried in four different locations along the jointing surface of the old and the new embankment. One strain meter was buried in the rock foundation 2.3 meter below the base line of the old embankment. Twenty four strain meters were buried in the old and the new embankment of five different levels as shown in Fig. 1.

3.3 The Observed Temperature Fluctuations

In the Fig. 6, the temperature fluctuations measured at four different levels of the united embankment are shown in comparison with the calculated values during the period from the summer of 1977 to the autumn of 1979. The locations temperatures of which are shown in the figure, are mostly in the new embankment and some in the old embankment. The figures show that the agreement between the estimated temperatures and the observed is generally reasonable. However, it is pointed that the measured temperatures at AA-18 and AA-19 drop rapidly while the calculated temperatures do not in the winter when the pause of placement of concrete was taken. This may be due to the fact that the location of the points AA-18 and AA-19 is near to the surface of the lift which was left exposed to the winter atmosphere without further placement of new concrete above it and the assumed value of the coefficiient of heat convection might not have been appropriate.

Except there, it is concluded that the simulation gives good estimation of temperatures during the two years after the beginning of the construction. The figures show as well that the temperature fluctuation at the elevation of 837 meters is as small as 5°C while it is as large as 13°C at the elevation of 860 meters and both curves are coverging to the fixed constants.

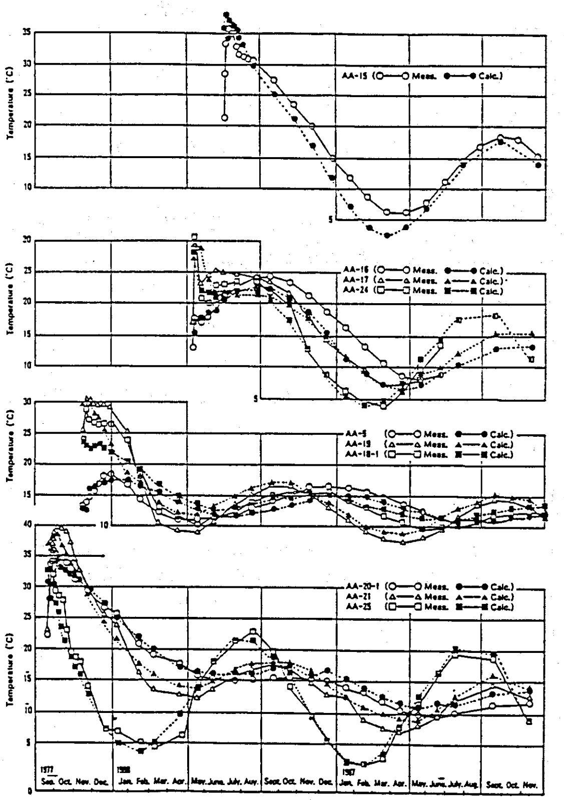


Fig. 6 Temperature Variation of the Embankment

3.4 The Observed Strain Fluctuations

In Fig. 7, the strain fluctuations at four different levels of the embankment are shown in comparison with the caluculated values during the same period as in the Fig. 6.

The strain shown are the total strain which is defined as the sum of the strain due to temperature, the strain due to creep and the strain due to stress. Though there are some disagreements of both values at initial two or three months after the placement of concrete, the agreements as a whole are reasonable.

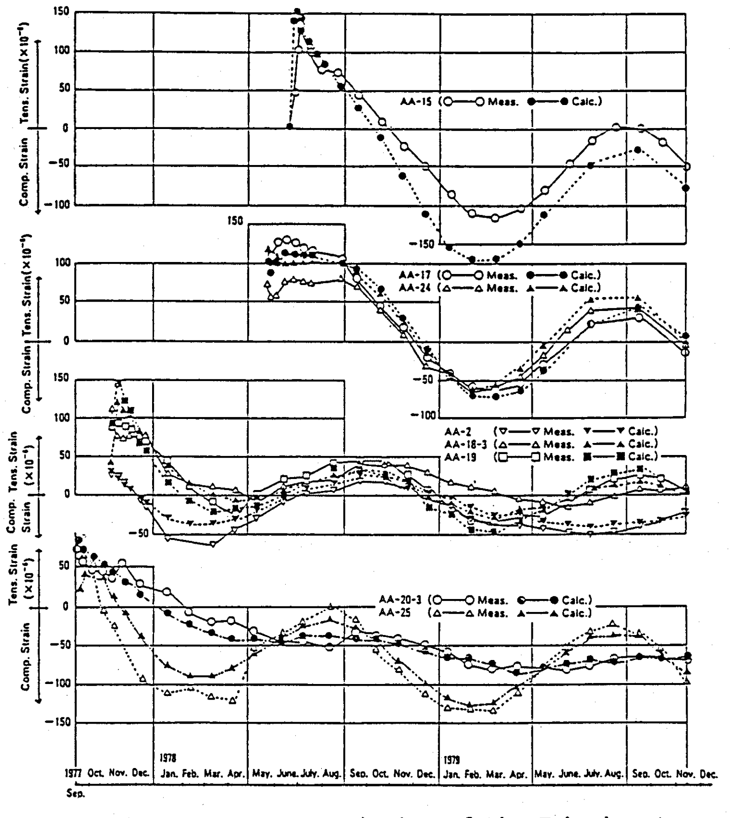


Fig. 7 Stress Variation of The Embankment

It should be noted that the fluctuation of strains as well as of temperatures are observable at the center of the embankment, width of which is smaller than 15 meters. As such, the jointing area near the top surely recieves stress fluctuation following the temperature fluctuation, which indicates that careful jointing method to endure the stress fluctuation is necessary at the portion. The calculated maximum tensile stress at the upstream face of the old embankment is shown to be about 9.5 kgf/cm^2 which is almost same amount that occures in the same place without new concrete.

3.5 Observation of The Slip at The Joint

At the jointing surface of the new and the old embankments, 4 shear displacement meters and 4 joint meters were embeded. Theses sensors are about 30 cm long and the observed values are needed to be corrected for thermal expansion or thermal contraction of the sensor itself and the concrete. Theses give rise to certain amount of errors. Therefore the reliability of the observed values during the period when the temperature goes up rapidly with cement hydration and the Young's modulus increases rapidly are somewhat lower than the values obtained when these material constants are stabilized. Hence, the long term fluctuation after the temperature and the rigidity stabilized are more dependable. Out of 8 sensors, one which is buried in the crown portion of the old embankment showed the seasonal fluctuation amplitude of which is about 0.8 mm per year. This may show the local separation of the two embankments. However, the rest of the sensors do not exibit the fluctuations and the strains observed in the various locations of the embankment agreed with the calculated values which has obtained assuming the unity of the old and the new embankments. These results are considered to show that the two embankments are intergrated to the unity. The calculated maximum normal and shear stresses which occur at the jointing surface are less than 10 kgf/cm^2 and the strength of the portion could easily been over the occurring stresses.

4. A LIMIT STATE IN TERMS OF TENSILE CRACKING IN GENERAL CONCRETE DAMS

In the section to follow, the more general thermal stress problems of a heightening construction of a dam are discussed using the analytical method, the accuracy of which is assured as mentioned in the preceding sections.
The temperature variation together with the rigidity variation of newly placed concrete give rise to the critical state in terms of tensile cracking in the following three locations in the embankment, i.e.,

(1) the upstream surface of the old embankment,
(2) the upper surface or side surface of the newly placed concrete block, and
(3) jointing surface of the two embankments.

The time when the maximum tensile stress occurs is different in each location of (1) to (3). However, they all follow the stress fluctuation pattern shown in Fig. 8, that is, the first stress peak A is reached between 2 to 4 weeks after the placement of concrete and the second stress peak B is reached in mid winter or in mid summer. The first peak point is the direct effect of cement hydration heat. The second peak point is due to the effect of more general thermal gradient.

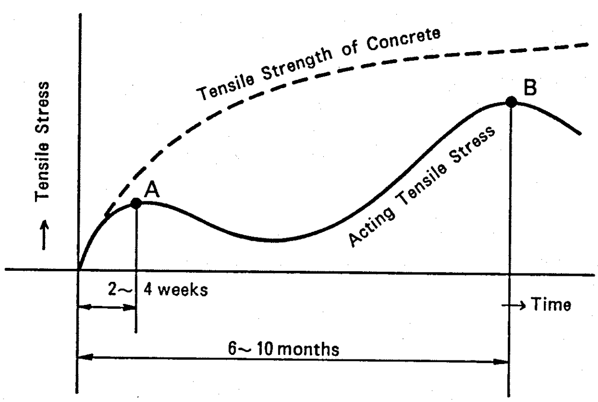


Fig. 8 Lmit State in Terms of Tensile Cracking

Location (1) do not have a stress peek point A, as the location is far from where new concrete is placed but have a peak point B. Hence, the stress becomes maximum in several months after the completion of new embankment. The location (2) has the peak stress point A at a time relatively short after the placement of concrete. The location (3) is considered to have both peak stress points A and B.

5. FACTORS AFFECTING THE CRITICAL STRESS

It is often misunderstood that the reduction of unit cement content reduces thermal stress in all cases. However, it is not true in the case of the thermal stress which occures at the location (1), at the upstream face of an old embankment during a few years after the completion of heightening. This reasoning is shown in the Fig. 9 (a) where the critical temperature differences of various concrete mix propotions are shown. In the figure, the unit cement content of case (a) is 220 kgf/cm^2, the unit cements contents of case (b) are 170 and 220 kgf/m^3 for inner concrete and outer concrete respectively, and case (c) has 148 kgf/m^3 cement and 37 kgf/m flyash.

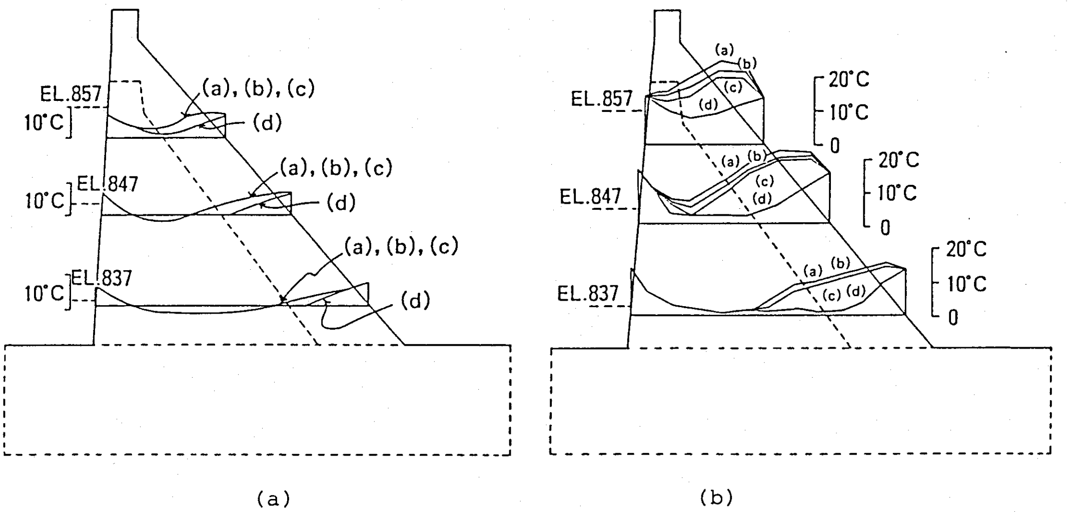


Fig. 9 The Critical Temperature Difference in Short Term (a), and in Long Term (b)

Critical temperature differences are calculated taking the temperature obtained at 7 days after the completion of all concreting as initial values and the temperatures obtained in the mid winter that comes first after the completion as the final values. It is understood from the figure that there are no temperature differences in the values between case (a), case (b) and case (c) and it may be concluded that the stresses arising due to these temperature drops are all same from thermal influential line theory. However, it should be stated that the thermal stress discussed so far is concerned with the values of σ_{II} in Eq. (11). The total thermal stress is the sum of σ_I and σ_{II} and σ_I in the heightening construction is comparatively small compared with σ_{II}. Indeed in this case σ_I is almost zero.
Compared with the three cases discussed so far, the effect of setting a cooling off period inbetween continuous concreting, case (d), is very large. It may be seen in the same figure. Indeed, the stress calculation showed that the thermal stress induced by the cement hydration heat of new concrete was nil in the model dam in this case.
Another critical temperature difference is the one that occurs at the stabilization of temperature which is expected to be realized in more than ten years after the completion of heightening.

This difference is shown in Fig. 9 (b). The figures show that there exist 2°C to 3°C differences between each cases. Hence, linear analysis will indicate that there exist differences in the intensities of thermal stresses in the stabilized temperature condition. However, it should be noted that the final stabilization will take place after more than ten years and the differences may be reduced by the time dependent factors such as creep. Another point that shuld be noted is that there exist temperature fluctuation at the surface area even after the stabilization of temperature due to the ambient temperature fluctuation effect, which is the important factor to calculate the maximum thermal stress of upstream face.

Considering those observation, it was decided at the construction of Kuroda dam to take a few months pausing of concreting in winter time and cool off the heat of the portion which affect most in the generation of thermal stresses of the upstream face.
The thermal stresses working normal and paralell to the jointing surface were less than 10 kgf/cm^2 and it is considered not so difficult to strengthen the jointing surface to withstand the maximum stress occurring at the portion.

6. THE PROPOSED METHOD TO EVALUATE THERMAL STRESS IN HEIGHTENING CONCRETE DAMS

The most important engineering subject at the heightening construction of a concrete dam is the unifying method of an old embankment and a new embankment. Hence, the thermal stress cracking in the jointing surface and in the upstream surface should most carefully be prevented. Tensile stress arising in an embankment is affected by the mix propotion of new concrete, lifting up schedule, cooling method and so forth. The very simple and effective method to evaluate these factors is proposed as follows.
(1) The thermal stress influential line is developed by giving unit temperature drop to each discretized element of a dam embankment.
(2) Critical temperature difference is calculated using the transient heat flow analysis. Lifting up schedule, mix propotion and other factors influencing the temperature should be included in the analysis.
(3) By the product of the influential line intensity and the critical temperature difference, the thermal stress is obtained.
(4) If the thermal stress thus obtained is larger than the allowable stress, reduce the magnitude of temperature difference at the portion where the influential line intensity is large. This may be the change of lifting up schedule or local cooling or local change of the mix propotion.

The proposed method must calculate unsteady heat flow several times but need not to carry out FEM stress analysis which is almost 4 times time consuming compared with the heat flow analysis as the degrees of freedom is twice as many as the one of the heat flow analysis. The method is applied to the thermal stress calculation of Kuroda dam and was powerful to reduce the 18 kgf/cm^2 tensile stress at the upstream face. The reduction of the stress and the detection of true reason would have been otherwise impossible why the reduction of unit cement contents do not reduce the thermal stress during a year or two after the placement of concrete.

7. CONCLUSION

Analytical study on thermal stress at the heightening construction of Kuroda Dam was made and the results were compared with the observed data which was obtained during a year and half after the completion of the construction. The results showed that the analysis gives reasonably acculate prediction of thermal behavior of a heightened concrete dam. Basing on the assured analytical method, a simple and design oriented thermal stress prediction method was proposed. The proposed method consists of constructing a thermal influential line and calculating critical temperature differences. Once these factors are obtained, it is easy to estimate the intensity of thermal stress and the effectiveness of each construction method.

Finally, sincere thanks are due to the members of the Construction Office of The Okuyahagi Hydraulic Electric Power Station of Chubu Electric Power Company and also due to Mrs. Eiko Egashira of the Central Reseach Institute of Electric Power Industry for the data processing and computation.

REFERENCES

[1] M. Kondo, "The Report of the 20th Congress of Large Dams," Large Dams, No.9, 1959

[2] Chugoku Electric Power Comp., "Study on the Heightening Construction of Ohdomari Dam," August, 1956

[3] Y. Sakaguchi, Y. Niwa, and C. Mori, "Study on The Heightening Construction of Sakurayama Dam, No.1," Hydraulic Electric Power, No.55, Sep., 1961

[4] Y. Sakaguchi, Y. Niwa, and C. Mori, "Study on The Heightening Construction of Sakurayama Dam, No.2," Hydraulic Electric Power, No.56, Nov., 1961

[5] Chubu Electric Power Comp., "Project on Okuyahagi Electric Power Station by The method of Pumping Up," Jan., 1974

[6] H. Abe, Y. Aoyagi, and T. Tanabe," Creep of Concrete at Elevated Temperature and Analysis of PCPV," Concrete Journal, No.6, June, 1974

[7] Y. Yoshida, "Concrete Strength of Kuroda Dam," Reseach Report of N.I.T., Nov., 1973

[8] A. Haraguchi, T. Tanabe, M. Kawasumi and T. Okazawa, "Study on The Concreting Method at Heightening Construction of Kuroda Dam," Research Report No.375561, Central Research Institute of Electric Power Industry, March, 1976.