Temperature Control of Mass Concrete in Japan

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•THE SCALE OF ROAD CONSTRUCTION in Japan in 1967, expressed in terms of annual expenditures for road programs, was approximately one trillion yen (about $2,780 million), and in 1969 it reached approximately one trillion two hundred billion yen (about $3,300 million) to occupy approximately 2 percent of the gross national product. However, the state of development of highways is still very far from being adequate, and it is further planned to appropriate annual budgets exceeding one trillion yen in accordance with a 5-year highway development plan.

A conspicuous feature of the numerous concrete structures that will be constructed to accompany this road development is that these structures are becoming of larger size with the marked increase in traffic, particulary in and around large cities. Specific examples include the proposed construction of large suspension bridges such as Kanmon Bridge, connecting the islands of Honshu and Kyushu, and Honshu-Shikoku Interconnection Bridge, connecting the islands of Honshu and Shikoku. The concrete structures used in long suspension bridges, of course, are also of large size.

These concrete structures are of such size that they may be called mass concrete structures. They differ from dam concrete only in that unit cement content is higher. In the design and construction of such structures, it is necessary to consider volume change of concrete due to heat generation as in the case of dams in order to prevent cracking.

TEMPERATURE STRESSES IN MASS CONCRETE

Compared to concrete of thin cross sections, concrete of large cross sections such as thick slabs have a slower rate of heat diffusion, and thus the concrete temperature of massive structures rises markedly immediately after placement. When left standing to cool in a natural state, it requires from about 2 weeks to several hundred days to return to around air temperature.

In the volume change of concrete arising from such temperature changes, it is common for free deformation to be restrained externally by outside conditions such as in the case of a foundation and internally by inside conditions such as temperature distribution within the concrete, shape of cross section, and differences in coefficients of thermal expansion and moduli of elasticity between adjoining portions. Unless suitable considerations are made in design and construction, numerous cracks will be found as temperature stresses occur.

The examples of such crack formation are many. Figure 1 shows some cases of cracks formed in concrete bridge piers (1). Cracks with widths of 0.1 to 0.3 mm and spacings of 3 to 4 m were the most numerous, with the cracks running through the full cross section. The age at which cracks appeared was between 13 and 23 days with the rate of crack-extension in one day being 25 to 40 cm. Thermocouples were embedded at the centers to measure temperature changes, and as indicated in Figure 2 it was found that cracking occurred due to extensive temperature change.

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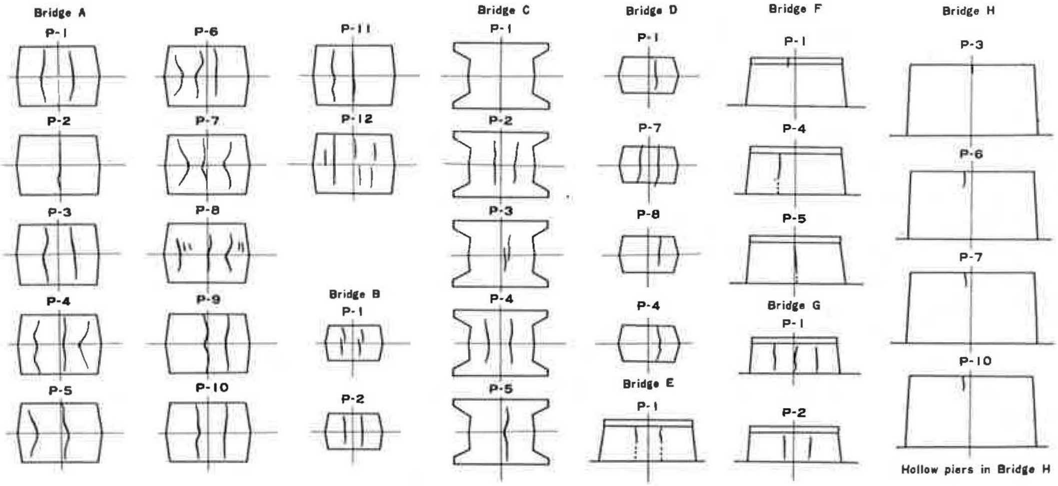


Figure 1.

Analyses of temperature stresses of members may be performed by elasticity theory; but generally, when there is foundation restraint, temperature stresses are estimated by the equation

$$\sigma_t = R \cdot E_c \cdot \alpha \cdot T \tag{1}$$

where

σ_t = tensile stress;
R = degree of restraint;
E_e = effective modulus of elasticity of concrete, in general obtained as $E_c/1 + 0 \left(4 \frac{E_c}{E_f}\right)$, where E_c is modulus of elasticity of concrete and E_f is modulus of elasticity of foundation rock;

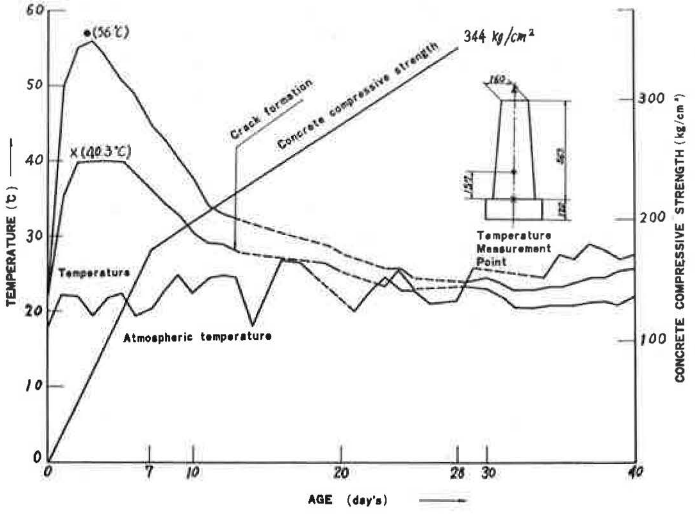


Figure 2.

α = coefficient of thermal expansion; and
T = temperature difference of concrete.

For the proper values of these constants, studies are being conducted in Japan through photoelasticity (2, 3) and by the finite element method (4) regarding degree of restraint. Investigations are also being carried on with respect to engineering properties such as modulus of elasticity and thermal characteristics. In estimating temperature difference, the Carlson method is employed with a combination of the Carlson and Glover methods used when cooling is performed.

A study of whether or not cracks will be formed confirmed either that the tensile stress sought by this method is within the range of the tensile strength or that the tensile strain is within the limits of extensibility.

In the case of a structure to be built near the sea coast or under water, reduction of cracking is of particular importance from the standpoint of durability of the structure.

TEMPERATURE CONTROL

It may be said that it is impossible to completely prevent temperature cracking. It is also difficult to predict the formation of cracks with high accuracy. Therefore, in actual practice, in order at least to reduce cracking, it is effective to limit the maximum temperature rise of the concrete by selecting suitable block dimensions and carrying out temperature control.

The measures that can be taken to limit peak temperature are reduction in heat generation through lowering of concrete temperature at the time of placement and reduction in unit cement content, facilitation of heat loss through longer intervals between concreting and reduction in lift heights, and removal of developing heat through artificial cooling.

In ordinary road construction in Japan where ready-mixed concrete is used, methods facilitating heat radiation or artificial cooling are chiefly employed. Cement generating less heat, other than cases of special production for the job by the manufacturer, would not be available, and normal portland cement is generally used. Therefore, although precooling may be given special consideration for large-scale projects, temperature control would ordinarily comprise calculations involving combinations of factors such as artificial cooling, time of placement (season and time of day), height of concrete lift, time interval between lifts, and concrete mix proportions. Although temperature calculations may be analyzed theoretically, at present when the hypotheses used in these calculations are not proven, emphasis is placed on simplicity rather than precision, and calculations by electronic computers are held to a minimal level.

In cases where temperature of concrete is lowered to the vicinity of final stable state temperature, such as when there is a necessity to perform grouting at an early stage, secondary cooling is performed.

Temperature Calculation Method

Not considering analysis of temperature distribution in a structure, for the purpose of comparing construction methods, it would suffice to assume a flat plane with infinite expanse and estimate the temperature rise of this plane. The method (5) of estimating maximum temperature rise described here is a combination of the Carlson and Glover methods by which temperature calculations were made for 160 cases and charted. The assumptions employed in these calculations are as follows:

1. The standard temperature is outdoor air temperature (0 C) and does not vary.
2. Concrete temperature at time of placement is equal to outdoor temperature. The concrete temperature at the surface is equal to outdoor temperature at all times.
3. The temperature of water used in pipe cooling is lower than outdoor temperature by 5 or 10 C.
4. The thermal properties of the concrete are as follows:

Thermal Diffusivity (m^2h)	Thermal Conductivity (kcal/m · h · c)
0.004	2.6
0.003	2.0
0.002	1.4

5. The adiabatic temperature rise curves are taken to be the 4 types indicated in Figure 3.

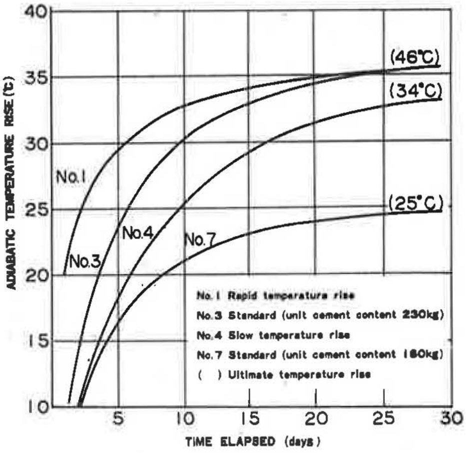
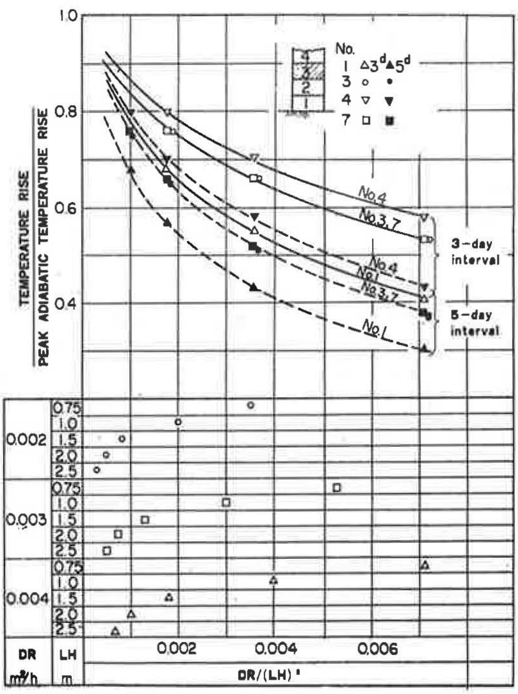


Figure 3.

Concrete Temperature Rise Under Conditions of Natural Heat Radiation—The relation between temperature rise of the third lift above either the foundation rock or the surface of old concrete and the peak temperature from adiabatic temperature rise can be expressed as a function of [thermal diffusivity/(lift height)^2], if the shape of the adiabatic temperature rise curve and the time interval between concrete placements are taken as shown in Figure 4. Selecting any rate of diffusivity and lift height and plotting the value of [thermal diffusivity/(lift height)^2] upward as a value on the x-axis and then moving laterally from the point intersecting the curve determined by the shape of the temperature rise curve and the interval of placement, one obtains the average temperature of the lift (Lift 3) on the y-axis as a ratio of the adiabatic temperature rise.

Effect of Pipe Cooling—In order to obtain the effect of pipe cooling, Figure 5 is employed. This is the ratio of temperature rise of a lift that loses heat naturally and a lift subjected to pipe cooling and is expressed as a function of the cooling constant (6). The cooling constant, X, is obtained as a function of h^2t/D^2 and $K{:}L/C_W \cdot \rho_W \cdot q_W$. Here, h^2 is the thermal diffusivity of concrete; t is the cooling time; D is the diameter of a cylindrical volume of concrete handled by one cooling pipe, in general taken as $D = \sqrt{(\text{lift height}) \times (\text{pipe spacing})/\pi}$; K is the coefficient of thermal conductivity of concrete; L is the length of cooling pipe; C_W is the specific heat of water; ρ is the specific gravity of water; and q_W is the discharge of cooling water. Therefore, variations in lift height, pipe spacing, thermal properties of concrete, pipe length, and discharge of cooling water may all be considered as varying the cooling constant, X.



DR: Rate of diffusivity LH: Lift height

Figure 4.

To obtain the maximum temperature rise of a lift that is pipe-cooled, one first finds the temperature rise of a lift left to natural cooling in Figure 4, determines the cooling water temperature, and then employs Figure 5.

Thermal Properties of Concrete

Very seldom are proper numerical values employed in carrying out temperature calculations.

Thermal properties of concrete include items such as coefficient of thermal conductivity, specific heat, thermal diffusivity, and coefficient of thermal expansion. There are various methods of testing these properties, but in general the method used by the Bureau of Reclamation and modified in part is employed (6).

1. The coefficient of thermal conductivity is obtained by heating the center portion of a cylindrical specimen to produce a constant flow of heat in a radial direction. Measurements are made of the input required to maintain both the temperature difference between the interior and exterior of the specimen and a regular state; then calculations are carried out.
2. The specific heat is obtained from calculations on measurements of the input required to raise a concrete specimen placed in a calorimeter to a designated temperature.
3. The thermal diffusivity is obtained from calculations on measurements of temperature variation caused at the center portion by irregular heat conduction of a specimen of a designated temperature placed in a water tank of a different temperature.
4. The coefficient of thermal expansion is obtained by calculations based on the length change of a specimen embedded with a strain gage caused to vary in temperature in a range between 20 and 60 C.

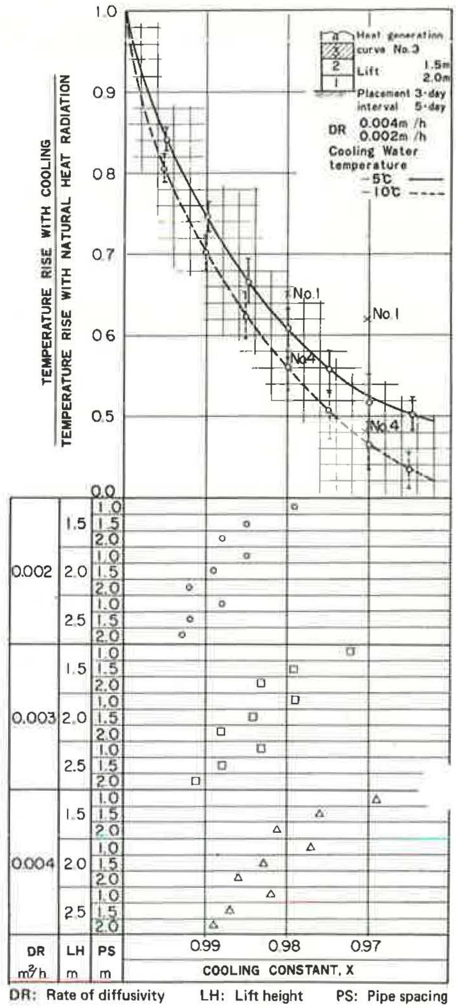


Figure 5.

Figure 6 shows the influences on thermal properties of factors such as variety of cement, water-cement ratio, aggregate content, and rock quality of aggregate. This figure shows that the factors most influencing the thermal properties of concrete are aggregate content and rock quality of aggregate.

A discussion follows of diffusivity, the value of which is the most needed item for carrying out temperature calculations.

Relation Between Rock Quality of Aggregate and Diffusivity—The specimens were of 20-cm diameter and 40-cm height and were embedded at the centers with thermocouple of 0.3-m diameter. The thermocouples were connected to an electron tube-type automatic equilibrium recorder for recording the temperature changes of the specimens, and calculations were made by the following equation:

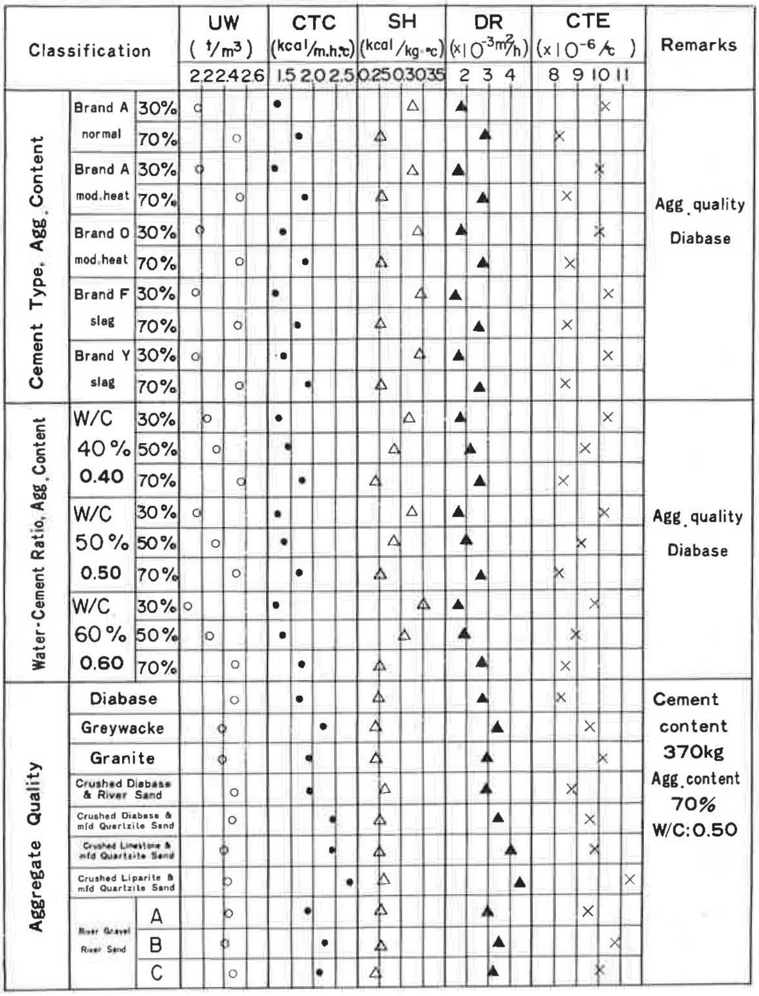
$$\frac{\theta_2 - \theta_W}{\theta_1 - \theta_W} = \frac{8}{\pi}\left\{\sum_{n=1}^{n=\infty}\left[\frac{e^{-(h\cdot n\cdot\pi/H)^2 t}}{n}\right] x \sin\frac{n\pi}{H} Z\right\}$$

$$\left\{\sum_{n=1}^{X=\infty}\left[\frac{J_0\left(\frac{X_n\cdot r}{R}\right)}{X_n\cdot J_1(X_n)}\right] x\left[e^{-(h\cdot X_n/R)^2 t}\right]\right\} \quad (2)$$

where

θ_1 = specimen temperature, deg C, at start of measurement;

θ_2 = temperature, deg C, of specimen center after immersion in water tank for specified period of time;



UW: Unit weight CTC: Coefficient of temperature conductivity
SH: Specific heat DR: Rate of diffusivity CTE: Coefficient of thermal expansion

Figure 6.

θw = temperature, deg C, of water tank;
h^2 = thermal diffusivity, m^2/h;
H = specimen height, m;
R = radius of specimen, m;
Z = measurement point of temperature in axial direction of specimen, in this case H/2; and
r = measurement point of temperature in radial direction, in this case 0.

The diffusivity rate can also be obtained from

$$h^2 = \frac{K}{C \cdot \rho} \tag{3}$$

For the sake of convenience the former is here called the Direct Method and the latter the Indirect Method.

Figure 7 shows the relationship between aggregate content and diffusivity when ag- gregates of varying rock qualities are used. Figure 8 shows the relationship between

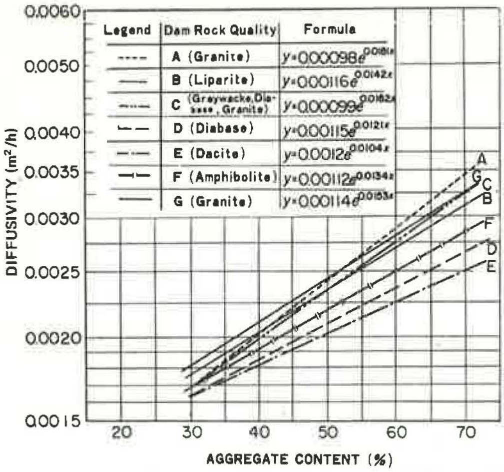


Figure 7.

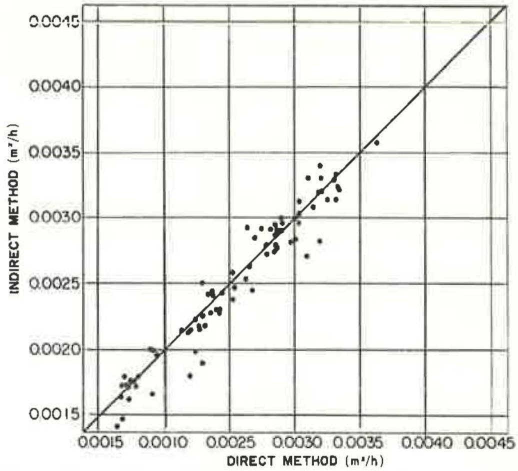


Figure 8.

the Direct Method and the Indirect Method, and it is seen that the two agree fairly well. Generally speaking, the aggregate content of mass concrete is 70 to 80 percent, and the heat diffusivity may be considered to be in the range from 0.0025 to 0.004 m^2/h.

Relations Between Temperature and Moisture Content of Specimen and Diffusivity— The relation between specimen temperature and diffusivity is shown in Figure 9; there is little variation. The relation with the specimen moisture content is shown in Figur 10. Specimens were moist-cured for 6 months and then left standing for 1 year in an atmosphere of 60 to 80 percent humidity. They were then waterproofed before measurements were made.

Relation Between Diffusivity and Coefficient of Thermal Conductivity—The relation between diffusivity and coefficient of thermal conductivity is shown in Figure 11 and may be considered to be linear.

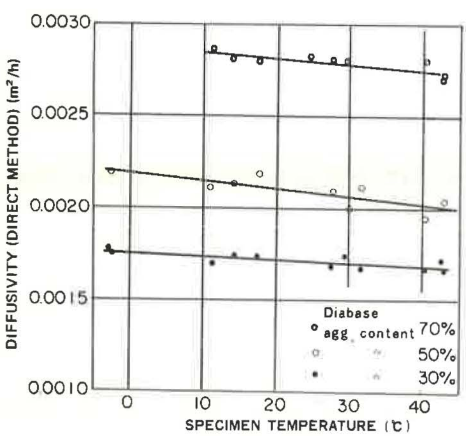


Figure 9.

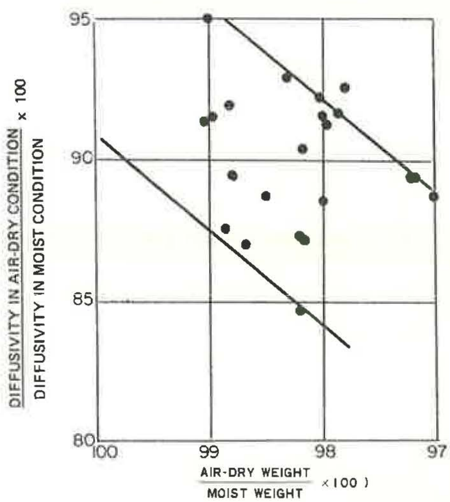


Figure 10.

Adiabatic Temperature Rise

In measurements of adiabatic temperature rise, it is difficult to maintain an insulated condition for a long period of time due to the nature of the apparatus. By employing the apparatus shown in Figure 12, which is a modification of the traditionally used apparatus, it became possible to conduct measurements for about 2 weeks.

The relationship between adiabatic temperature rise and unit cement content obtained from these measurements is shown in Figure 13. The peak temperature employed to obtain these was the results of measurements up to 14 days with the n th day and the n th + 1 day plotted on the x- and y-axes respectively in making an estimation (Fig. 14).

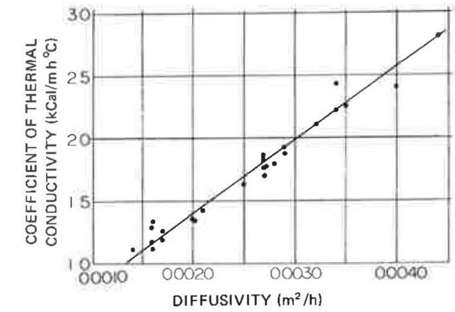


Figure 11.

The relation between concrete temperature at the time of placement and temperature rise, shown in Figure 15, indicates a difference of approximately 4 C in the case of placement temperature approximately between 10 and 30 C, but this was not very distinct.

The temperature rise history is influenced greatly by the temperature of concrete at the time of placement as shown in Figure 16.

The adiabatic temperature rise curve has heretofore been expressed by $\theta_t = \theta_0(1 - e^{-at})$ or $\theta_t = \theta_0\left(1 - e^{-at^n}\right)$, but both independently cannot very well express the temperature rise for a 2-week period. However, with today's computers, it would not be necessary to go to extremes to find a numerical expression. It is thought sufficient for information to be available for selecting the values for the various ages.

FOUNDATION RESTRAINT

When concrete placed on foundation rock or on old concrete has volume changes caused by temperature, it is subjected to foundation restraint, and temperature stresses result.

Research is being carried out in Japan on foundation restraint by various methods. In model tests employing photoelasticity, it has

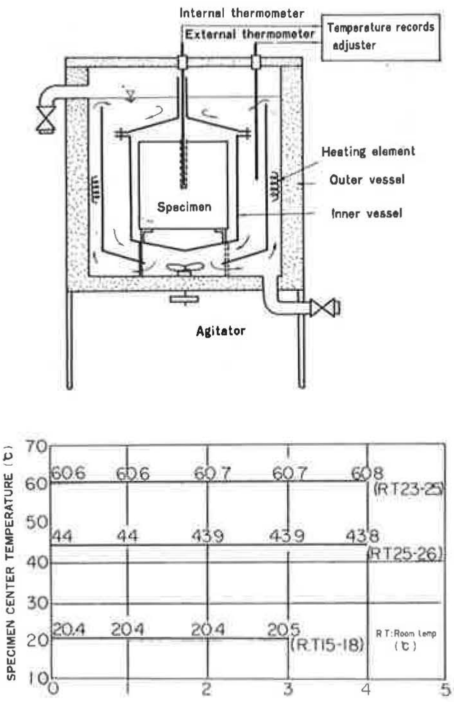


Figure 12.

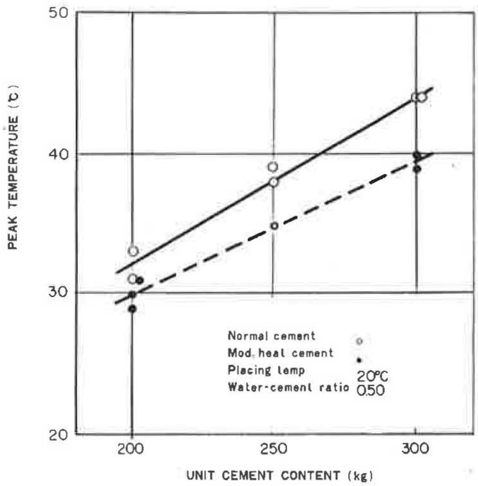


Figure 13.

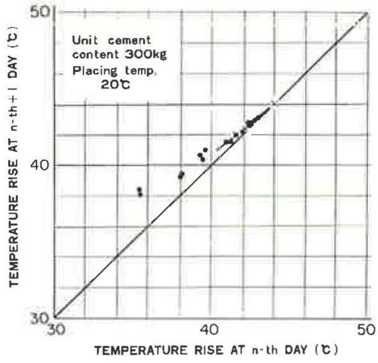


Figure 14.

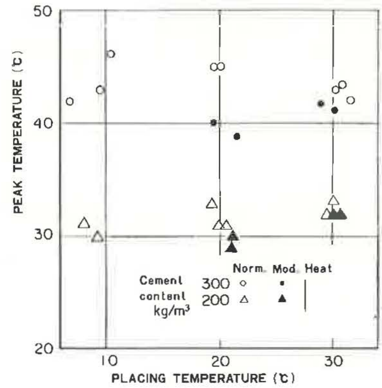


Figure 15.

been discerned that the degree of restraint (a) is a function of the ratio between the modulus of elasticity of the concrete and the modulus of elasticity of the foundation rock, (b) varies with the ratio between the length and the height of the restrained side of the concrete block placed on the surface of the foundation, and (c) varies with the shape of the concrete block. It was also learned that the foundation restraint does not extend beyond a height of 0.45 times the length of the restrained side. Further, Mori (3) has disclosed that degree of restraint, $\bar{R}$, on a semi-infinite foundation would be satisfactory at $R = 1\Big/\left(1 + 0.4\frac{E_C}{E_R}\right)$, but on a foundation of identical width, it would rather be better to employ $R = 1\Big/\left[1 + \left(\frac{E_C}{E_R}\right)^{0.7}\right]$. Hayashi has sought the degree of restraint of a concrete block on a semi-infinite foundation by the finite element method and has secured roughly the same results as Mori.

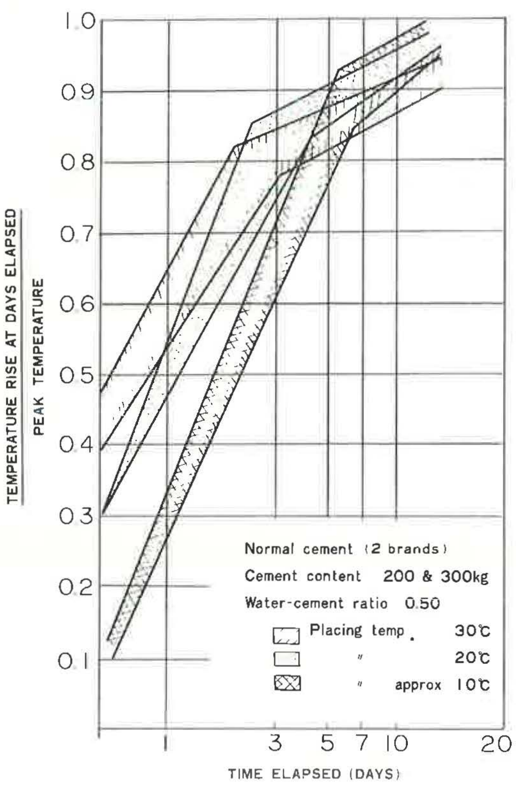


Figure 16.

CONCLUSIONS

In order to estimate temperature stresses accurately to arrive at measures for prevention of cracks, research should be continued on (a) estimation of temperature distribution, (b) calculation methods regarding restraint stresses, and (c) engineering properties of concrete at the early stage of hardening.

Each item represents a problem requiring long-term research, and all are fields in which there are relatively few researchers in Japan. However, with the continuing increase in construction of mass concrete structures there is no doubt that research in these fields will become more extensive.

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