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METHOD FOR EVALUATING PERFORMANCE OF TESTING APPARATUS FOR ADIABATIC TEMPERATURE RISE OF CONCRETE

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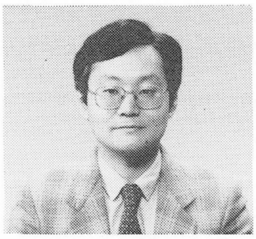

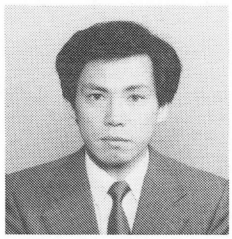

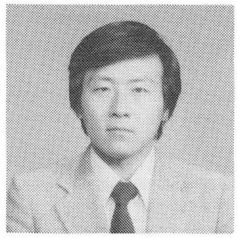

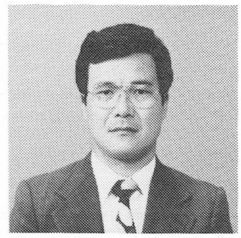

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SYNOPSIS

There is no reliable method to evaluate performance of testing apparatus for adiabatic temperature rise of concrete. This paper reports measurement of specimens of different size, measurement for a long term, repeated measurement and measurement discrepancy among testing apparatus of the same type. Comparison is also made between the time-dependent temperature change in mass concrete and the experimental adiabatic temperature rise under the same concrete placing conditions. The series of tests has revealed that not only achievement of adiabatic condition in testing apparatus, but also comparison with time-dependent temperature changes in mass concrete is necessary to evaluate performance of the testing apparatus.

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1. INTRODUCTION

For the purpose of planning effective measures to cope with the problems after predicting the likelihood of thermal cracks , temperature of mass concrete and thermal stress in general have been evaluated. In this thermal analysis, the adiabatic temperature rise is normally used as the calorific value of concrete.

Even though heretofore a large number of apparatuses for measuring the adiabatic temperature rise have been devised, test models built, and improvements piled up, it is difficult to say that the performance of these has been sufficiently scrutinized.

It is reported that the experimental data which were used for establishing the standard values of the adiabatic temperature rise in RC specification and JCI guideline varied considerably with testing methods and testing apparatuses.[1] Furthermore, there are numerous points unclear concerning the performance testing methods of the adiabatic temperature rise testing apparatus itself.

This study reports not only the procedure of investigating performance of our pilot testing system for adiabatic temperature rise, but also a comparison between the adiabatic temperature rise and the temperature history in an actual structure or a large scale specimen of similar type, assuming the same conditions of concrete placement in both cases.

2. PREVIOUS RESEARCH

Here in Japan, research concerning the adiabatic temperature rise of concrete, has been reported from 30 years previously. However, researchers and scientists have been using different testing apparatuses to evaluate adiabatic temperature rise in concrete because of an absence of standardized testing methods.[2][3] Furthermore, the method of verifying the performance of the testing apparatus also has varied according to the researcher or scientist.

Takano and others proposed a variety of testing apparatus of water circulation type with an improved Fuel Research Institute type adiabatic calorie meter, and verified its performance by a comparison with the value from a large scale adiabatic calorie meter.[4]

Yanagida and others proposed a variety of water circulation type of testing apparatus called "Public Works Research Institute type", and performed verification of performance, based on a measurement of the adiabatic condition of the testing apparatus 3 or 4 days after that, by adding water of a specified temperature in the container of the specimen.[5]

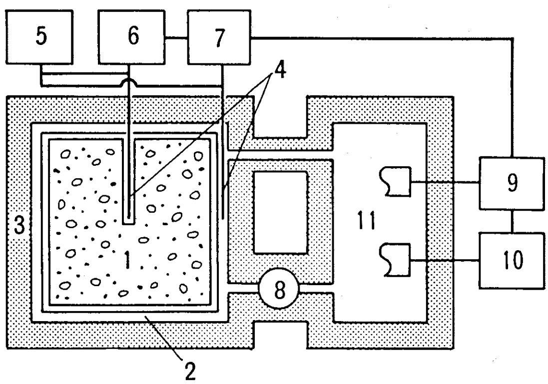
Tsukayama enlarged the apparatus of air circulation type for use on dam concrete , increased the amount of samples , and furthermore , based on control of the acceleration of the response rate, the performance was raised. Then he measured the adiabatic temperature rise of concrete with high heat generation rates with both new and old apparatus, and verified the improved effectiveness.[6]

On the other hand, in recent years, numerous examples of measurement of temperature in mass concrete structures have been published. Of these, in several of the reports, there has been an indication that the test results of the adiabatic temperature rise that has been previously reported is considerably below measured temperature of the actual structure.[7][8][9]

The adiabatic temperature rise testing apparatus of concrete that has been utilized until now has been applied to concrete like dam concrete where the unit cement content is small, and the adiabatic temperature rise is comparatively small. However, currently, in RC structures, the ones where thermal cracking is becoming a problem are those with a comparatively rich mixture where the unit cement content is on the level of 300 kg/m³ or above, and cases of concrete with a rapid rate of heat generation are numerous. Accordingly, in a state where a standardized testing method has not been established, it is thought necessary to compare the actual measurement of the temperature history in large size structures which are considered in an adiabatic state or similar large size specimens where the mix proportion conditions of the placed concrete are the same and the adiabatic temperature rise measured with the testing apparatus, and to verify the performance of the concrete adiabatic temperature rise testing apparatus.

3. THE ADIABATIC TEMPERATURE RISE TESTING APPARATUS USED IN EXPERIMENTS

This study employed the concrete adiabatic temperature rise testing apparatus shown in Fig.1. This apparatus is one newly developed by the authors, and its conception is as follows;
the specimen is placed in an isolation wall type thermal medium jacket with a circulation system of partitioned structure of 60 cm in inner diameter and 60 cm in inner height, without providing thermal insulation in the clearance between the specimen and the jacket which is surrounded by thermal insulation. Thus, the specimen is kept heat-isolated condition, and the adiabatic temperature rise is measured making the thermal medium temperature in the jacket follow that in the center of the specimen.[10][11] In this case, even if there is an extremely rapid change of the core temperature, assuming the largest value on the level of 30 °C/h, a delay compensating computing element is provided in order to cause the jacket temperature to follow such a change by making possible control within an accuracy of 0.2 °C. Also,the temperature of the specimen as well as that of the insulation, was detected with platinum resistance thermometer (JIS class 0.2).



1 Sample 2 Jacket 3 Insulation 4 Temperature Sensor 5 Recorder
6 Monitor 7 Controller 8 Pump 9 Heater 10 Cooler 11 Tank

Fig. 1 Adiabatic temperature rise testing apparatus

As thermal medium, water was primarily used. Furthermore, a test apparatus using silicon oil as thermal medium also was fabricated, the use of which makes it possible to measure the concrete adiabatic temperature rise even in the case where such temperature becomes over 100 °C with a high unit cement content and with a high concreting temperature. Moreover, this testing apparatus is designed so that the specimen may be replaced together with the thermal medium jacket, using two types of specimen, one with a diameter of 30 cm and height of 30 cm and one with a diameter of 60 cm and height of 60 cm; because of this, it is possible to evaluate concrete specimens sufficiently uniform in mixture of high representativeness taking into consideration the maximum size of the coarse aggregate.

4. VARIOUS FACTORS THAT HAVE AN EFFECT ON THE ADIABATIC TEMPERATURE RISE

(1) THE EFFECT OF SPECIMEN SIZE

The Test Result of the adiabatic temperature rise in the case of different specimen sizes is shown in Fig.2. The mix proportion that was used in the experiment, in C1 of Table 1, in the case where the unit cement content was 350 kg/m³ . This figure shows that when a cylindrical specimen of diameter 60 cm and height 60 cm is compared, in adiabatic temperature rise, with the one of diameter 30 cm and height 30 cm, each dimension being twice as large, in other words even though the volume is 8 times as large, the former is only 0.5 ℃ or less larger than the latter.

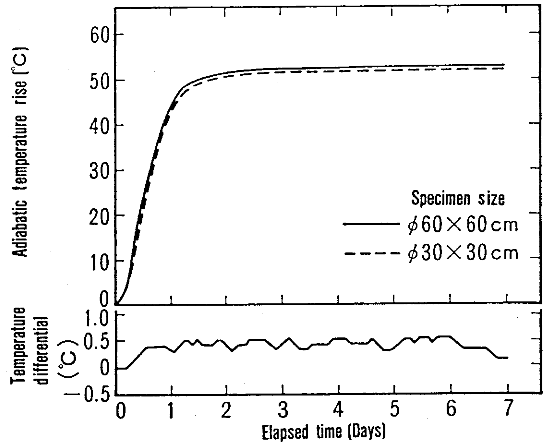


Fig. 2 Comparison of adiabatic temperature rise with different size of specimens

Table 1 Mix proportions used in the experiment

Mix Type	Series	Water cement ratio (%)	Sand aggregate ratio (%)	Unit weight (kg/m³)					Concreting temper-ature (℃)
				Water	Cement	Fine aggregate	Coarse aggregate	Chemical* admixture	
Concrete	C 1	42.3	40.4	148	350	736	1113	1.750	20.0
Concrete	C 2	50.8	39.0	132	260	726	1164	0.650	20.0
Concrete	C 3	50.0	40.6	150	300	760	1129	0.750	30.0
Mortar	M 1	75.1	——	226	352	1525	——	0.106	26.0
Concrete	C 4	74.5	42.1	158	212	812	1118	0.425	15.2
Concrete	C 5	44.0	40.5	169	384	707	1078	0.960	26.0

* AE & water-reducing agent

(2) VERIFICATION OF THE ADIABATIC STATE PRESERVATION CAPABILITY AS WELL AS REPRODUCIBILITY OF TEST RESULTS

With moderate heat Portland cement as the base, a three component type cement, with blast furnace slag and fly ash mixed in, was used. Concerning the C2 mix proportion concrete shown in Table 1, the placement temperature was 20 ℃, and in order to verify the reproducibility of the testing apparatus, the experiment was performed and repeated. The measurement time period was set at 14 days. The materials used in both experiments were the same, and the experiments were performed with approximately one month gap in between.

From the experimental results shown in Fig.3, the variation in the measured value between the first and second times was no more than 0.9 ℃ so it can be

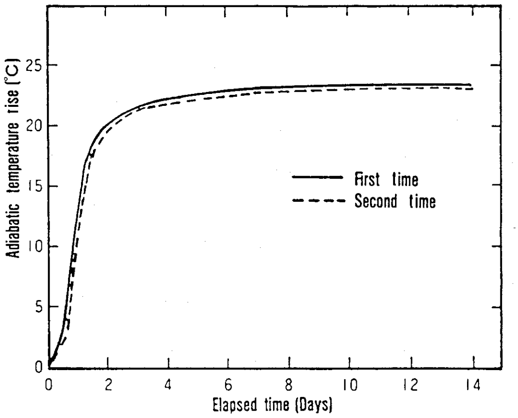


Fig. 3 Adiabatic condition preservation capability as well as reproducibility of test results

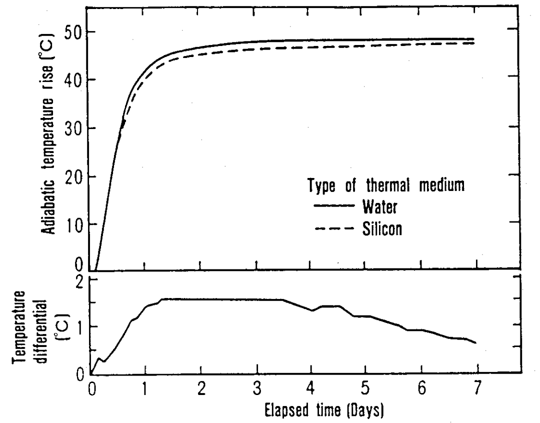


Fig. 4 Errors among the same type of testing apparatus

said that the reproducibility of the test was satisfactory. Furthermore, for both the first and second times, from the tenth day after placing, adiabatic temperature rise changed very little, demonstrating what is called an adiabatic state. Then, that state was preserved for about 4 days continuously.

(3) ERRORS IN MEASUREMENT BETWEEN THE SAME TYPE OF TESTING APPARATUS

In the case where the mix proportion of concrete shown in C3 in Table 1 was used, what degree of error occurred between the three apparatuses of the same type is shown in Fig.4. Of the three testing apparatuses, water was used as thermal medium for two of them, and for the remaining one, silicon oil was used. Furthermore, the three testing apparatuses were made at approximately one year intervals.

The variation in measured value in the two testing apparatuses that used water as the thermal medium was extremely small, and Fig.4 shows the larger of their measured values. Among the same type of testing apparatus, regardless of whether the different thermal mediums of water or silicon oil was used, with regard to concrete where the final value of the adiabatic temperature rise was approximately 50 °C, the error between both types that occurred throughout the entire measurement period was 1.5 °C at the maximum.

5. A COMPARISQN OF TEMPERATURE RISES WITH THOSE IN LARGE SIZE BLOCK SPECIMENS

A comparison was made between the concrete adiabatic temperature rise measured with the testing apparatus, and the temperature history in the core portion of the large size block specimen, which was a 2 m cube insulated on all sides with a foamed styrene of thickness 20 cm. Experiments were conducted for both the case where the unit cement content was high, using mortar with a high concreting temperature, and a rapid heat generation rate with cement hydration, as well as for the opposite case where the unit cement content was low and using concrete with a low concreting temperature, and a slow heat generation rate with cement hydration (M1 and C4 in Table 1).

The adiabatic temperature rise of concrete and the change with time of the core temperature of the large size block specimen are shown together in Fig.5. Since the concrete adiabatic temperature rise until about 3 days after placement was limited to the range below 1.0 ℃, with reference to the time dependent change of temperature in the core section of the block specimen which is thought to be very close to an adiabatic state, both cases are able to be said to be nearly consistent in the process of temperature rise.

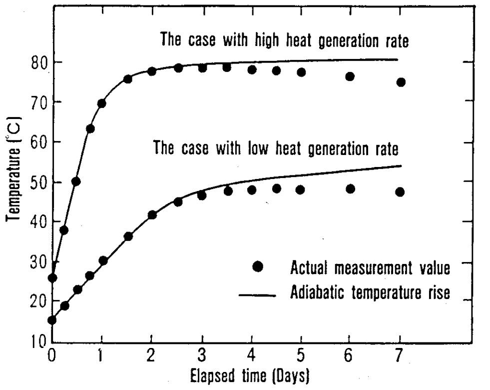


Fig. 5 Comparison of time-dependent changes with the 2m cube block specimens core temperature and adiabatic temperature

Even saying that in the process of temperature rise, the adiabatic temperature rise by the testing apparatus is nearly consistent with the core temperature of the large size block specimen, it is not possible to immediately accept that value as the correct adiabatic state value. This is because since even in the core of the cubic testing block of 2 m which is covered with foamed styrene insulation of the thickness 20 cm, thermal flows generate there and transfer toward the outside, gradually exerting considerable effect 2 or 3 days after placement, and it is impossible to confirm the accuracy of the final value of the adiabatic temperature rise due to the testing apparatus. Therefore, in order to investigate validation of the general adiabatic temperature rises due to the testing apparatus, using the values obtained, the time dependent change of the internal temperature of each large size block specimen is estimated through FEM analysis, and that result is compared with the actual measurement.

The thermal properties used in FEM analysis, excluding the heat transfer coefficient, were all found experimentally. In other words, the heat diffusivity of the mortar and the concrete are obtained based upon Glover's Law, through measurement under the conditions that the hot water temperature is kept at 60 ℃, the cold water temperature at 20 ℃, and the diameter of the specimen was 20 cm. The specific heat of each of the component materials of both the mortar and the concrete was determined using twin isoperibol calorimeter, and the specific heat was computed from the various component percentage of mix proportion. The heat conductivity was computed from Equation (1) using the heat diffusivity, the specific heat, as well as the density, found experimentally through the above procedure.

$$\lambda = c \rho h^2 \qquad (1)$$

Where, λ : heat conductivity (kcal/mh ℃)
c : specific heat (kcal/kg ℃)
h^2 : thermal diffusivity (m^2/h)
ρ : density (kg/m^3)

As for a block specimen where the hydration heat is generated rapidly, the heat transfer coefficient α is set at 0.425 kcal/m^2h ℃ through the FEM analysis, so that during the period of 14th day to 28th day after placement when almost no heat generation is considered to occur, the estimation values at the block center may be confined to a range of ± 0.5 ℃ of the actual measurement. Thus, the heat transfer coefficient was obtained independently of the calorific

Table 2 Thermal properties for the large size block specimen

Type of experiment	Thermal properties of concrete and mortar				Heat transfer coefficient (kcal/m²hr °C)
	Thermal conductivity (kcal/mhr °C)	Specific heat (kcal/kg °C)	Density (kg/m³)	Thermal diffusivity (10^{-3} m²/hr)	
The case of rapid heat generation rate	1.89	0.299	2145	2.95	0.425
The case of slow heat generation rate	2.02	0.247	2300	3.55	0.425

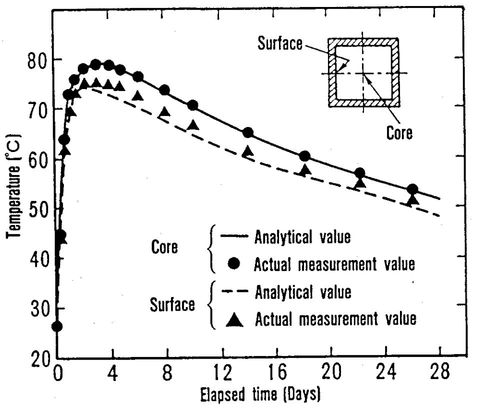


Fig. 6 Temperature analysis results and actual measurement values of the 2m large size cube block specimens (The case of high heat generation rate)

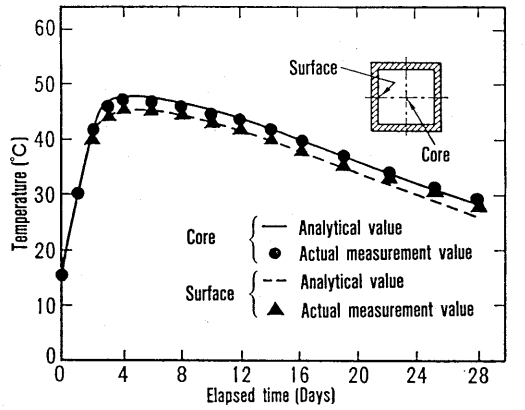


Fig. 7 Temperature analysis results and actual measurement values of the 2m large size cube block specimens (The case of low heat generation rate)

value of concrete, in other words from thermal properties determined experimentally. These thermal properties are summarized and shown in Table 2.

One-forth of the section of the block specimen was used for thermal analysis, referring to the actually measured temperature distribution. The surface of the specimen was assumed to be a heat transfer boundary and foamed styrene was not taken into consideration as an element, but its effect was considered into calculation by converting it to the equivalent heat transfer coefficient, as mentioned above. For the ambient temperature , the measurement values were used. If the adiabatic temperature rise of concrete, as a function of time, is formulated to a regression equation, errors therefrom are not negligible. For this reason, the value obtained by multiplying the actually measured temperature rise per unit time by heat capacity is used as the amount of the generated heat.

Time-dependent temperature changes in the actual measurement value and the value calculated through analysis of the core section and the surface section of the large size 2 m cube block specimen are shown in Fig.6 and in Fig.7. For the case where the the heat generation rate of hydration is slow as well, the heat transfer coefficient is assumed to be the same as the case where the heat generation rate of hydration is rapid.

In the case where hydration heat generates rapidly, the difference with the measurement value for the surface temperature is no more than 1.6 °C. Furthermore,the difference between the measurement and the analytical value on the surface is assumed to come not only from failing to consider an experimental error of heat diffusivity and specific heat, but also from the fact that temperature-dependency is not taken into consideration for these thermal properties.

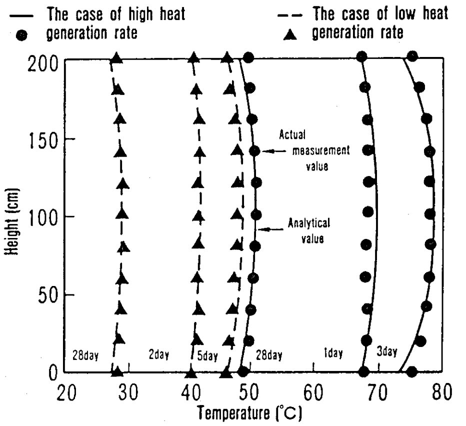


Fig. 8 Temperature distribution of the 2m large size cube block specimens

For the case where the heat generation with hydration is slow as well, the value for the analytic temperature of the surface as well as that of the core section of the large size block specimen agree well with actual measurement. Further, from the temperature distribution of the inner cross section shown in Fig.8, it can be observed that the analyzed value of the temperature agrees well with the actual measurement over the entire cross section.

6. COMPARISON OF THE ADIABATIC TEMPERATURE RISE OF CONCRETE AND THAT OF AN ACTUAL STRUCTURE

As shown in Fig.9, the adiabatic temperature rise of concrete and the temperature history of a massive concrete block (a part of a waterbreak) which is 11 m wide, 4.4 m long and 7 m high were compared, and applicability of adiabatic temperature rise was studied.[12] The mix proportion of concrete that was used was C5 in Table 1, and the same materials as well as mix proportion were used as that of the concrete placed at the actual construction site. Moreover, based on the same concreting temperature, an adiabatic temperature rise experiment was performed.

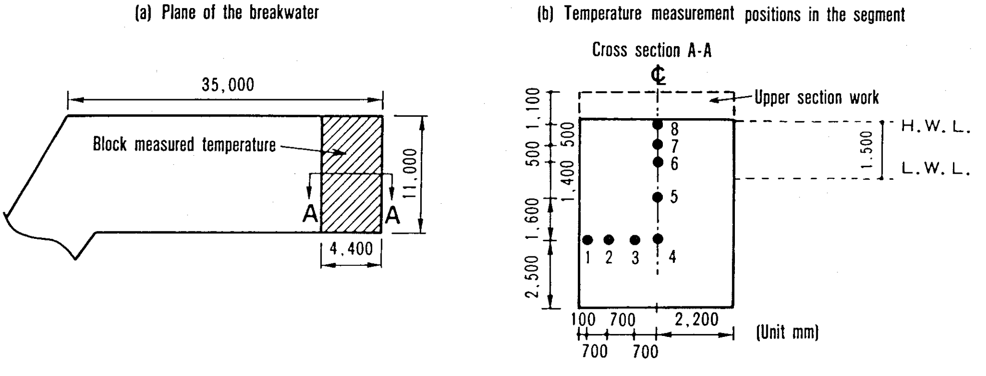


Fig. 9 Plane of the breakwater concrete and temperature measurement positions in the segment

The adiabatic temperature rise of concrete and time dependent temperature change at measurement point No.4 in Fig.9 until the level of 2 to 3 days after placing, which is deemed to be virtually in an adiabatic state, are shown in Fig.10. The adiabatic temperature rise of concrete is nearly in agreement with the temperature change with time of measurement point No.4, as for the temperature rise process, but more than in the case of the large size block specimen, where the error was slightly larger, becoming at about 2.2 °C at the maximum. Breakwater concrete is concrete which is placed under water according to the tremie method, and it is thought that the cause of the error becoming slightly larger is the occurrence of the segregation of concrete though small.

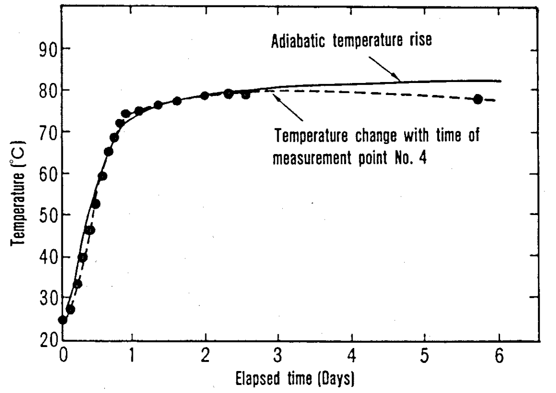


Fig. 10 The change with time of the temperature of the breakwater core section and the adiabatic temperature rise

In the case of the breakwater concrete, as was the same for the large size block specimen , there was not a perfect adiabatic state. At this point, using the adiabatic temperature rise of the concrete obtained from the test, a temperature analysis of the breakwater concrete was performed based on the FEM, and the validation of that adiabatic temperature rise overall was examined.

Thermal properties of concrete other than the adiabatic temperature rise were fixed with reference to the previous literature.[13] However, the heat transfer coefficient is determined through FEM analysis as in the case of a large block specimen in 5), using measurement values at the point No.4 which is positioned at the center of the cross section of the lowest part of the waterbreak, in such a manner that, for the period from 14th day to 28th day after placement, the analytical curve, with an error of ± 0.5 °C , may agree with the downward measurement curve. These thermal properties values are as shown in Table 3.

Table 3 Thermal properties used in the thermal analysis of the breakwater concrete

Type	Thermal properties of concrete and bedrock			Heat transfer coefficient (kcal/m²hr °C)
	Thermal conductivity (kcal/mhr °C)	Heat capacity (kcal/m³ °C)	Thermal diffusivity (10^{-3} m²/hr)	
Concrete	2.4	715	3.36	50.0
Bedrock	2.0	650	3.25	50.0

The analyzed cross section includes the bottom section bedrock as well as the existing breakwater concrete, and element discretization like that shown in Fig.11 was performed for FEM analysis. The concrete upper surface exposed to air, the concrete lateral surface contacting with sea water and the bedrock upper surface contacting with sea water were assumed to be a heat transfer boundary respectively. Moreover, positions below 4 m from the upper section of bedrock was taken as a fixed temperature boundary.

At the measuring points close to the upper section work, which was executed on the shore, that placement effect appeared very strikingly in the actual temperature measurement. At that point, by analysis, from the element discretization shown in Fig.11, a temperature analysis of the concrete section under water was performed up to the time that it was jointed to the upper section, and at that point in time the temperature of each node was found. Next, discretization of the mesh elements was performed anew, including the upper section work, and after correcting the boundary conditions, analytical value of each node was given as the starting temperature, and once again analysis was implemented. For the ambient air temperature as well as the water temperature, rather than the value actually measured, the average daily temperature was determined and used.

A comparison of the actual temperature of the breakwater concrete and the analytical value is shown in Fig.12 and in Fig.13. From these figures it can be observed that, not only for the breakwater lower section, but also for the breakwater upper section, where the effect of the

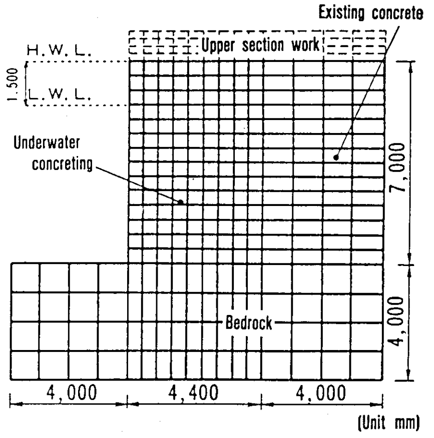


Fig. 11 Element discretization diagram of the breakwater concrete

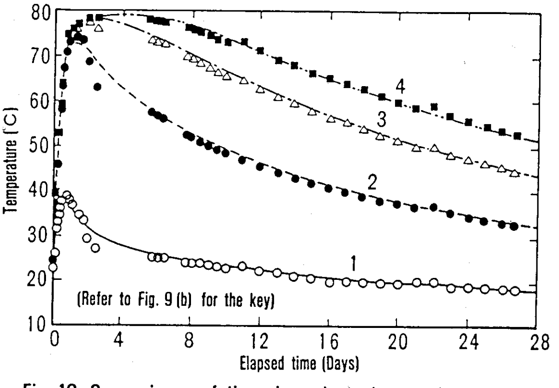


Fig. 12 Comparisons of time-dependent changes between measured and analytical temperatures for the breakwater concrete (No. 1)

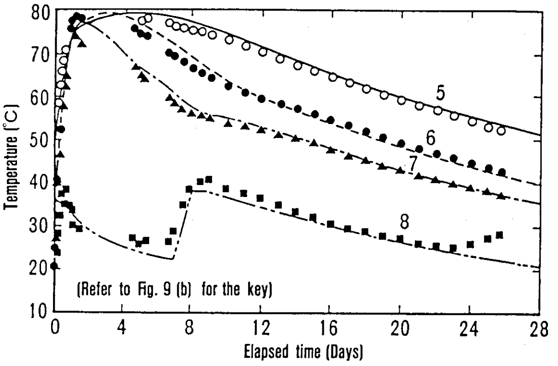


Fig. 13 Comparisons of time-depent changes between measured and analytical temperatures for the breakwater concrete (No. 2)

placing of the upper section work appears, the actual temperature measurement and the analytical value agree comparatively well.

7. CONCLUSION

In this research, first of all we examined the capabilities of the apparatus itself with reference to the experimentally produced adiabatic temperature rise testing apparatus. After that, the various conditions of the concrete to be placed were same, and a comparison of adiabatic temperature rise and the temperature history of an actual structure or a similar large size specimen was performed. Within the results of this research, the following findings were obtained.

(1) In the experimental model of test apparatus, even if the specimen becomes 8 times larger in volume, the difference in adiabatic temperature rise is only 0.5 °C or less, and it was confirmed that the adiabatic state is able to be maintained even during the four-day period from the 11th day to the 14th day after concreting during which almost no adiabatic temperature rise occurs.

The test repeated with the same material and the same placing temperature shows that the temperature difference given as test result was about 0.9 °C only, and that in the case of the testing apparatuses of the same type, the temperature difference among them was 1.5 °C at maximum for all the measurement period.

(2) In a comparison of the adiabatic temperature rise according to the experimentally produced testing apparatus and the temperature history of an actual structure or a similar large size specimen in which the various conditions of the concrete to be placed were same in both the case where the heat generation in the concrete was rapid as well as the case where it was slow, adiabatic temperature rise was very nearly in agreement with the actual measured temperature of a core segment of an internal cross-section. However, the error in the adiabatic temperature rise and the actual measured temperature of a core segment of an internal cross-section was greater in the case of an actual structure than for a large size specimen.

(3) In verifying the capability of the concrete adiabatic temperature rise testing apparatus, the various conditions of the placing concrete were same, and the implementation of a comparison of the temperature histories of a large size specimen thought to be in virtually an adiabatic state or an actual structure was, as above mentioned, valid. However, depending on the smallest dimension of the structure, because maintaining an adiabatic state even in the core segment is difficult, it is very important that, in addition, the precision of the final value of the adiabatic temperature rise be investigated. In that case, a comparison of the actual measured value and the analyzed value of the temperature change with time of each internal cross section by FEM analysis was an effective method.

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