Article

Modelling the Strength and Fracture Parameters of Dam Gallery Concrete Considering Ambient Temperature and Humidity

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Abstract: Due to the complex structure and stress distribution of dam galleries, cracks often appear during the construction, operation and maintenance periods of dams. This paper proposes a method to determine the real strength and fracture parameters of gallery concrete, considering environmental temperature and humidity. The strength and fracture tests were carried out for gallery concrete at various ages and under different curing temperature and humidity conditions. The influence of curing conditions on the mechanical properties of gallery concrete was quantitatively analyzed. The prediction equations of the strength and fracture parameters of gallery concrete under arbitrary temperature and humidity were established. Based on the measured temperature and humidity data, the real mechanical parameters of gallery concrete were predicted. The research results show that the influence of environmental conditions on mechanical parameters cannot be neglected, as this can result in a strength reduction of up to 33.81%. The proposed equations can be used to predict the mechanical parameters of gallery concrete, subject to real environmental conditions, which can help to correct a maximum deviation of 54.62% on parameters calculated using actual ages. The proposed method can provide a scientific basis for the cracking risk analysis and safety assessment of the gallery structure under actual conditions.

Keywords: dam; gallery concrete; strength; fracture parameters; maturity model; cracking risk; ambient temperature; ambient humidity

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1. Introduction

In recent years, with the continuous improvement of China's dam intelligent construction technology [1–3], the dam engineering community has put forward higher requirements for the risk prevention of dam projects. The cracking of dam surface and dam body concrete is one of the important factors affecting the safe operation of dam structures [4–6]. Among them, dam galleries are openings or passageways, constructed in the dam structure, for the purpose of drainage, inspection, foundation grouting, gate operation and equipment control and, due to the complex structure and stress distribution of dam galleries, cracks often appear during the construction, operation and maintenance periods of dams [7–15]. For example, the Punt dal Gall double curvature arch dam, with a height of 130 m, on the border between Switzerland and Italy, has formed multiple cracks in the top arch of the gallery during its operation period, and the maximum crack opening in one year reached

1.5 mm [7]. On another gravity arch dam, with a height of 45 m, a horizontal crack has been observed in the upper gallery after 25 years of operation, which has propagated along almost the full length of the dam and the crack opening continued to increase at a rate of 0.1 mm per year [8]. The Dagangshan high arch dam in China experienced different degrees of gallery cracking in the top arch of the traffic gallery and foundation gallery, within two months of the completion of the concrete pouring, with the maximum crack depth reaching 3.7 m [9]. In addition, the Dworshak gravity dam in the western United States [10], Konar gravity dam in India [11], Sefid-Rud buttress dam in northern Iran [12], Xiaowan super-high arch dam [13] and Shaxikou gravity dam in China [14], all experienced varying degrees of gallery cracking problems during the construction or operation period. In order to accurately analyze the working behavior and cracking risk of the dam gallery structure, one of the prerequisites is the acquisition of real strength and fracture parameters for dam gallery concrete. In the traditional gallery cracking analysis process, the strength and fracture parameters of gallery concrete, at different ages, provided by different inspection units, are generally taken directly [15]. However, existing studies have shown [16–22] that the environmental temperature and humidity of concrete have a greater impact on the development of its strength and fracture parameters, hence, the gallery structure cracking analysis carried out directly, using the strength and fracture parameters at a specific age, may lead to the large deviations between the analysis results and the real anti-cracking safety factor of the gallery concrete. Therefore, it is necessary to propose a method for determining the real strength and fracture parameters of dam gallery concrete, considering the environmental temperature and humidity, which can provide a scientific basis for the cracking risk analysis and safety assessment of the dam gallery structure under actual conditions.

In order to quantitatively analyze the influence of environmental temperature on the strength of concrete, various scholars have carried out a large number of experimental studies [23–26]. For example, Kim et al. [23] carried out strength tests at ages of 1 day, 3 days, 7 days and 28 days, and under curing temperatures of 10 °C, 23 °C, 35 °C and 50 °C, for five types of concretes with different mixture proportions, which were Type I cement concrete with water–binder ratios of 0.4 and 0.5, Type V cement concrete with water–binder ratios of 0.4 and 0.5, and Type V cement concrete with water–binder ratios of 0.4 and a fly ash replacement rate of 15%. The research results show that high curing temperature conditions, such as 35 °C or 50 °C, are conducive to the development of early-age concrete strength. However, when the age is greater than 7 days, the strength of concrete subjected to high temperature conditions may be less than the strength of concrete under normal temperature conditions, such as 10 °C or 23 °C; the effect of the curing temperature on the strength of concrete varies with the variation of concrete mixture proportions. Castellano et al. [24] carried out compressive strength tests, at ages of 2 day, 7 days, 28 days, 90 days and 365 days, and with curing temperatures of 20 °C, 40 °C, 60 °C for cement pastes with water–binder ratios of 0.3, 0.4 and 0.5. It was found that when the water–binder ratio is 0.3, the compressive strength of cement paste at 2 days and 7 days increases with the increase in the curing temperature, while it decreases as the increase in curing temperature for cement paste at 28, 90 and 365 days. For cement pastes with water–binder ratios of 0.4 and 0.5, the effect of the curing temperature on cement strength is basically the same, but the compressive strength started to decrease with the increase in curing temperature, at the age of 7 days. Regarding the effect of curing temperature on concrete fracture parameters, Li et al. [27] cast and cured the dam concrete wedge-splitting specimens on the construction site, during the high temperature condition, in summer, and the low temperature condition, in winter, and carried out the fracture tests at different ages. The results show that the fracture parameters of concrete poured in summer are greater than those of the same age in winter. Yu and Ansari [28] performed fracture tests on three-point bending concrete beams, cured at 14 °C, 23 °C and 35 °C. It was established that for concrete of the same age, the higher the curing temperature, the greater the fracture energy of concrete. Mi et al. [29,30] carried out concrete wedge-splitting

fracture tests under curing temperatures of 5 °C, 20 °C, 40 °C, and 60 °C and found that at early ages, the fracture parameters of concrete increase with the increase in the curing temperature, but from 14 days to 60 days of age, the fracture energy and equivalent fracture toughness of concrete under the curing condition of 60 °C are smaller than the fracture parameters of the concrete at the same age when the curing temperature is 40 °C; hence, the high curing temperature condition may have an adverse effect on the development of concrete fracture parameters. From the above research results, it can be seen that a higher curing temperature can promote the development of early-age concrete strength and fracture parameters, but an excessive curing temperature may have an adverse effect on the growth of later-age concrete's mechanical properties. Furthermore, for different concretes, the curing temperature has different effects on the development of strength and fracture parameters. Therefore, for important projects, such as dams, it is necessary to carry out the tests under different curing temperature conditions, on the construction site, to clarify the impact of the curing temperature on concrete strength and fracture parameters.

Similar to curing temperature, curing humidity also has a significant impact on the development of concrete strength [31–36] and fracture parameters [18,37,38]. Powers [31] found that when the relative humidity inside the concrete is lower than 80%, the hydration rate of the cementitious material will gradually decrease or even stop, which will slow down the development of the macro mechanical properties of the concrete. Liao et al. [32] carried out the compressive strength test of concrete, under curing temperatures of 12 °C, 27 °C and 40 °C, and curing humidity of 70%, 80%, 90% and 100%. The results show that under certain curing temperature conditions, the difference in the compressive strength of concrete at an early age, due to changes in curing humidity, is small. With the increase in age, the reduction proportion of compressive strength, due to the decrease in curing humidity, gradually increases. For example, when the curing temperature is 12 °C, the age is 17.33 days, and the curing humidity is 75%, the compressive strength is about 34% lower than the strength at the same age when the curing humidity is 100%. Liang et al. [33] performed the flexural tensile strength test of pavement cement concrete, when the curing humidity was 40%, 60%, and 100%, and found that the lower the relative humidity, the lower the flexural tensile strength of concrete at different ages, and the difference in flexural tensile strength grows with the increase in age. Compared with the experimental research carried out by scholars, on the influence of curing humidity on the strength of concrete, there are still few studies on the effect of environmental humidity on the fracture parameters of concrete. Mi et al. [18] carried out concrete wedge splitting fracture tests, with curing temperatures of 5 °C, 20 °C, 40 °C and 60 °C, and curing humidity of 30%, 50%, 70% and 98%. The results show that curing humidity has little effect on the fracture parameters of early-age concrete, but as the age increases, the lower the relative humidity, the lower the fracture toughness and fracture energy of concrete at each age, and the difference in fracture parameters becomes larger with the increase in age; that is, the detrimental effect of the decrease in curing humidity on the fracture parameters of concrete becomes more and more obvious. Other scholars [37,38] paid more attention to the influence of water content on concrete fracture parameters. For example, Bažant [37] carried out an experimental study on the effect of concrete internal water content on fracture energy, at different curing temperatures, and found that under normal temperature conditions, concrete internal water content has little effect on concrete fracture energy. As the curing temperature increases, the influence of the internal water content on the fracture energy of the concrete gradually increases. Lau et al. [38] also studied the influence of the internal water content of concrete on fracture toughness. The test results show that the fracture toughness of concrete decreases with the increase in its internal water content. In summary, the increase in curing humidity is conducive to the development of concrete strength and fracture parameters and its influencing effects vary, depending on the concrete material and curing temperature. For actual projects, concrete strength and fracture tests under different curing temperature and humidity conditions should be carried out at the same time, so

as to provide a basis for the determination of concrete mechanical parameters under real environmental conditions.

This study used the gallery concrete produced by the on-site mixing plant of a dam under construction in southwest China as the research object, then designed and carried out the strength and fracture tests of gallery concrete, at various ages and under different curing temperature and humidity conditions. Based on the test results, theoretical study was carried out to establish the prediction formula of the strength and fracture parameters of the gallery concrete, under arbitrary temperature and humidity history conditions. Finally, combined with the measured temperature and humidity data inside and outside the dam gallery, as well as the temperature of the gallery concrete, the real strength and fracture parameters of the gallery concrete at any age were predicted.

This research aims at establishing the strength and fracture properties of gallery concrete under real environmental conditions. Both temperature and humidity effects on the development of mechanical parameters of gallery concrete have been studied experimentally and theoretically, and the prediction equations of strength and fracture parameters have been established accordingly, which can help to correct the deviations of mechanical parameters of gallery concrete determined by traditional methods, using the actual age. This research provides a basis for all parties involved in dam engineering to better analyze and control the gallery cracks, which often appear during the construction, operation and maintenance periods of dams, thereby ensuring the safety of the dam gallery structure, as well as the whole dam structure.

2. Materials and Methods

China's dam construction has entered the era of intelligence, and a complete set of technologies, such as intelligent water-cooling, intelligent grouting, and intelligent spraying, based on extensive monitoring measurements, real analyses, and intelligent decision making, as well as closed-loop feedback control theory, have been gradually formed [3]. As an integral part of this set of technologies, this study proposes a method for determining and applying the real strength and fracture parameters of dam gallery concrete considering the ambient temperature and humidity. The specific process is shown in Figure 1. This method can determine the reliable material parameters of the gallery concrete at any time under real environmental conditions, which can be used for real-time cracking risk analysis of the dam gallery structure, and can then provide a basis for the optimization of the dam structure's operating status, the formulation of the construction schedule, or the control of gallery's temperature and humidity, etc.

This article focuses on the method of determining the real strength and fracture parameters of dam gallery concrete considering the environmental temperature and humidity. This method directly uses the gallery concrete produced by the on-site mixing plant of the dam project to pour concrete specimens, then carries out tests on the strength and fracture performance of gallery concrete under different ages and curing temperature and humidity conditions. Furthermore, a temperature–humidity maturity method is used to analyze the test results and determine the prediction equations of the gallery concrete strength and fracture parameters. Finally, based on the measured temperature and humidity data inside and outside the dam gallery, and the measured temperature data of the gallery concrete, the real strength and fracture parameter development curves of the gallery concrete at any age are established, which can be used to evaluate the cracking risk of the gallery structure under different working conditions.

2.1. Strength and Fracture Tests

2.1.1. Raw Materials and Mixture Proportions

The test directly used the gallery concrete with a maximum aggregate size of 40 mm and a slump of 70–90 mm produced by the concrete mixing system at the construction site of a dam project in southwest China. The water–binder ratio of concrete is 0.42, and the compressive strength for 180 days is 40 MPa. The mixture proportion of dam gallery

concrete is shown in Table 1. The cement is P·LH42.5 low-heat Portland cement with a density of 1450 kg/m^3 , which is produced by Sichuan Jiahua Jinping Special Cement Co., Ltd. located in Mianning County, China. The fly ash is produced in Qujing City, China, by Xuanwei Zhonghecheng Building Materials Co., Ltd. The fly ash conformed to Type F Class I, according to the standard of Chinese hydraulic engineering industry DL/T 5055-2007 [39]. The replacement rate of fly ash was 35% of the cementitious material, which was used to reduce the hydration heat release of large volume concrete and save cement. The chemical composition of the cement and fly ash is shown in Table 2. Both coarse aggregate and fine aggregate were limestone produced by a Hangudi sand and aggregate production system located in Qiaojia County, China, with a sand ratio of 34%. Air-entraining agent and water reducing agents were GYQ-I air-entraining agent and SBTJM-II retarder type II high-range water reducing admixture, respectively, both of which are produced by Sobute New Materials Co., Ltd. in Nanjing City, China.

Figure 1. Determination and application method in intelligent dam construction platform (IDAM 2.0) of real strength and fracture parameters of dam gallery concrete considering ambient temperature and humidity.

Table 1. Mixture proportion of dam gallery concrete.

Water (kg/m ³)	Cement (kg/m ³)	Fly Ash (kg/m ³)	Sand (kg/m ³)	Aggregate (kg/m ³)		High-Range Water Reducing Admixture (kg/m ³)	Air-Entraining Agent (kg/m ³)
				5–20 (mm)	20–40 (mm)		
112	173	93	687	669	669	1.33	0.093

Table 2. Chemical composition of cementitious material (% by mass).

Composition	CaO	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	MgO	SO ₃	K ₂ O	Na ₂ O	R ₂ O	Others
Cement	60.66	22.92	3.27	4.58	4.02	2.50	0.46	0.17	0.47	0.95
Fly ash	3.25	55.58	21.68	8.99	1.30	0.35	1.16	0.20	0.96	6.53

2.1.2. Test Scheme

To study the effects of curing temperature and humidity on the strength and fracture parameters of the gallery concrete, this research designed the strength and fracture tests of gallery concrete for 7 days, 14 days, 28 days, 60 days and 90 days and under the following 5 different kinds of curing conditions: curing humidity of 98%, curing temperature of 10 °C, 20 °C, 40 °C, and curing temperature of 20 °C, curing humidity of 30%, 60%. The detailed test scheme is shown in Table 3. The test used the large-scale temperature and humidity environmental chambers manufactured by Chongqing Haoyuan Environmental Test Equipment Co., Ltd. located in Chongqing City, China, to accurately control the curing temperature and humidity of the specimens. It should be noted that the temperature and humidity range designed by the experiment is basically consistent with the temperature and humidity variation range of the construction site of dam projects over the years and the actual temperature and humidity history of the gallery concrete. Therefore, the test results can better reflect the influence of actual environmental conditions on the strength and fracture parameters of the gallery concrete.

Table 3. Test scheme for the strength and fracture parameters of the environmental chamber-cured dam gallery concrete.

Curing Conditions	Temperature (°C)	Humidity (%)	Specimen Size (mm × mm × mm)	Number of Specimens	Number of Groups	Measured Parameters	Test Ages (Days)
T10RH98	10	98	150 × 150 × 150	30	10	f_c, f_t	7, 14, 28, 60, 90
			φ150 × 300	30	5	f_{acs}, E_c	
			330 × 300 × 120	15	5	K_{IC}, G_F	
T20RH98	20	98	150 × 150 × 150	30	10	f_c, f_t	7, 14, 28, 60, 90
			φ150 × 300	30	5	f_{acs}, E_c	
			330 × 300 × 120	15	5	K_{IC}, G_F	
T40RH98	40	98	150 × 150 × 150	30	10	f_c, f_t	7, 14, 28, 60, 90
			φ150 × 300	30	5	f_{acs}, E_c	
			330 × 300 × 120	15	5	K_{IC}, G_F	
T20RH60	20	60	150 × 150 × 150	30	10	f_c, f_t	7, 14, 28, 60, 90
			φ150 × 300	30	5	f_{acs}, E_c	
			330 × 300 × 120	15	5	K_{IC}, G_F	
T20RH30	20	30	150 × 150 × 150	30	10	f_c, f_t	7, 14, 28, 60, 90
			φ150 × 300	30	5	f_{acs}, E_c	
			330 × 300 × 120	15	5	K_{IC}, G_F	

φ is the diameter of the specimen.

In this study, a total of 300 gallery concrete strength specimens and 75 fracture specimens were cast. There are 60 strength specimens and 15 fracture specimens of 5 different test ages for each temperature and humidity condition. The three different types of specimens cast in the experiment are shown in Figure 2. Among them, the cube specimens with dimensions of 150 mm × 150 mm × 150 mm are used to determine the compressive strength f_c and splitting tensile strength f_t of concrete. The cylinder specimens with a size of φ150 mm × 300 mm are used to measure the axial compressive strength f_{acs} and static compressive elastic modulus E_c of concrete. The wedge-splitting specimen with a size of 330 mm × 300 mm × 120 mm and initial crack length to height ratio of 0.4 is used to determine the equivalent fracture toughness K_{IC} and fracture energy G_F of the concrete.

2.1.3. Specimen Preparation

The production, transportation and pouring of concrete is shown in Figure 3. To ensure the test results obtained in this research can be directly applied to the cracking risk analysis of the dam gallery structure, the concrete used in this test and in the construction of the dam gallery was produced at the same time and under the same conditions. After picking up the concrete material at the outlet of the mixing plant on the construction site, the concrete was transported to the pouring site by a thermal insulation dump truck and unloaded on the floor which had been sprinkled with water in advance, and then started to cast the 3 types of specimens at the same time, namely 150 cube specimens with a size of 150 mm × 150 mm × 150 mm, 150 cylinder specimens with a size of φ150 mm × 300 mm, and 75 wedge-splitting specimens with a size of 330 mm × 300 mm × 120 mm.

Figure 2. Three types of specimens cast on-site: (a) cube specimen; (b) cylinder specimen; (c) wedge-splitting specimen.

Figure 3. Production, transportation and pouring of concrete: (a) concrete mixing system at the construction site; (b) concrete transportation with thermal insulation dump truck; (c) pouring of specimens.

The cube and cylinder specimens were cast using high-quality engineering plastic molds, and the wedge-splitting specimens were cast using steel molds. A groove was reserved at the top of the steel mold for fixing a square tube with a size of 30 mm × 50 mm, so that the groove (as shown in Figure 2c) could be formed on the top of the specimen. The two side plates of the steel mold were designed with a seam with a height of 120 mm and width of 3 mm, which was used to embed a 3-mm thick steel plate with V-shaped bottom, so an initial crack (as shown in Figure 2c) could be formed in the molded specimen. Before casting the specimens, each mold needed to be brushed with oil in advance for later demolding. To minimize the impact of the casting batch and the environmental conditions during pouring on the test results, all the specimens in this test were cast at one time, that is, the casting was completed within 2 h of the concrete being transported to the pouring site.

To reduce the possible influence of the loss of moisture on the test results, all specimens were covered with plastic wrap after pouring, as shown in Figure 4a. After curing for 24 h, the specimens were removed from the mold and placed in the large environmental chamber shown in Figure 4b with the curing temperature and humidity set in advance. During the operation of the environmental chamber, the temperature and humidity in the environmental chamber were regularly monitored, to ensure that the curing conditions always met the design requirements. Figure 4c shows an example of three different types of specimens cured in an environmental chamber.

Figure 4. Curing of concrete specimens: (a) the surface of the specimen was covered with plastic wrap to prevent the loss of concrete moisture; (b) large temperature and humidity environmental chamber; (c) an example of three different types of specimens cured in the environmental chamber.

2.1.4. Test Methods

In this study, the strength tests were carried out on a Servo Hydraulic Universal Testing Machine with a range of 1000 kN, manufactured by Shanghai Hualong Test Instruments Corporation in Shanghai City, China, and the fracture tests were conducted on a computer-controlled electronic universal testing machine with a range of 300 kN, produced by Jiangsu Tianyuan Test Equipment Co., Ltd. in Yangzhou City, China, as shown in Figure 5a,b, respectively. During the fracture test, the data acquisition system shown in Figure 5c was used to record the applied vertical load F and crack mouth opening displacement (CMOD) by a 10 kN capacity force transducer (Shanghai East China Electronic Instrument Factory, Shanghai City, China) placed on the stiff steel profile and a clip gauge (Central Iron Steel & Research Institute, Beijing City, China) with a range of 4 mm installed ahead of the initial crack.

Figure 5. Test equipment: (a) Servo Hydraulic Universal Testing Machine; (b) computer-controlled electronic universal testing machine; (c) data acquisition system.

The strength tests were carried out in accordance with the relevant test standard DL/T 5150-2017 [40] of Chinese hydraulic engineering industry. According to this standard, the average value of the test results of 3 specimens in the same group for a certain test age and curing condition was taken as the final strength result.

The compressive strength tests adopted the stress loading method with a loading rate of 0.3 MPa/s. Before the test, the specimen was placed on the pressure plate of the testing machine, ensuring that the pressure surface of the specimen was not the pouring surface. Then, the load was applied with the preset rate until the specimen was broken, the failure load was recorded, the specimen was taken out, then all the above steps were repeated, and all the compressive strength tests for certain groups were completed. After the tests were completed, the failure loads were substituted into Equation (1) to calculate the compressive strength.

$$f_c = \frac{P}{A} \quad (1)$$

where f_c is the compressive strength, MPa; P is the failure load, kN; A is the pressure bearing area of the specimen, mm².

The splitting tensile strength tests also adopted the stress loading method with a loading rate of 0.04 MPa/s. Before the test, the center lines of the specimen were drawn to ensure that the splitting surface was parallel to the pouring surface. We then put the specimen in the center of the pressure plate of the testing machine and placed two rectangular strips (with cross-sectional dimensions of 5 mm × 5 mm) between the upper and lower pressure plates and the specimen. The test was conducted until the specimen was broken, the test data was then recorded, taken out the test piece, the above steps repeated, and all splitting tensile tests were completed. Substituting the failure load obtained from the tests into Equation (2) allows us to calculate the splitting tensile strength of the gallery concrete.

$$f_t = \frac{2P}{\pi A} = 0.637 \frac{P}{A} \quad (2)$$

where f_t is splitting tensile strength of concrete; P is failure load; A is cross-sectional area of the cube.

The test method of the axial compressive strength test was the same as the compressive strength test, and the loading rate was also 0.3 MPa/s. Substituting the failure load obtained from the test into Equation (3), we can calculate the axial compressive strength.

$$f_{acs} = \frac{P}{A} \quad (3)$$

where f_{acs} is the axial compressive strength, MPa; P is the failure load, kN; A is the pressure bearing area of the specimen, mm².

For the test of static compressive elastic modulus, the strain gauge method was adopted. After starting the test, we preloaded the specimen, then applied the load step by step at 0.5 MPa intervals, recorded the strain value of each step until the pressure value exceeded 40% of the axial compressive strength. Afterwards, the load remained at a constant rate of 0.3 MPa/s until the specimen was broken. The static compressive elastic modulus E_c can be calculated according to the stress–strain relationship obtained during the step-by-step loading process.

The wedge-splitting method was adopted for fracture tests and a diagram of the loading device is shown in Figure 6a. The test data was collected by a data acquisition system, as shown in Figure 5c. The vertical loading was applied with a constant displacement rate of 0.05 mm/min. In order to eliminate the influences of the self-weight of the specimen and the vertical component of the applied load on the fracture test results, two hinge roller supports were placed at the quarter points of the bottom of the specimen, which were also aligned with the centers of the roller bearing, as shown in Figure 6a. Figure 6b shows a schematic diagram of the F_H -CMOD curve, where F_H was obtained by transforming the vertical load F collected by the force transducer after considering the wedge angle of 15°, and the crack mouth opening displacement CMOD was directly recorded by the clip gauge.

Figure 6. Wedge-splitting fracture test: (a) diagram of the loading device and (b) schematic diagram of the F_H -CMOD curve.

The fracture parameter of equivalent fracture toughness was calculated by the expressions proposed by Xu et al. [41], as shown in Equations (4)–(8), as follows:

$$K_{IC} = \frac{F_{Hmax} \times 10^{-3}}{th^{1/2}} f(\alpha) \quad (4)$$

$$F_{Hmax} = \frac{(F_{max} + mg \times 10^{-3})}{2 \tan 15^\circ} \quad (5)$$

$$f(\alpha) = \frac{3.675[1 - 0.12(\alpha - 0.45)]}{(1 - \alpha)^{3/2}}, \alpha = \frac{a_c}{h} \quad (6)$$

$$a_c = (h + h_0) \left(1 - \sqrt{\frac{13.18}{CMOD_c \times E \times t / F_{Hmax} + 9.16}} \right) - h_0 \quad (7)$$

$$E = \frac{1}{tc_i} \left[13.18 \left(1 - \frac{a_0 + h_0}{h + h_0} \right)^{-2} - 9.16 \right] \quad (8)$$

where K_{IC} in Equation (4) is the equivalent fracture toughness, $\text{MPa} \cdot \text{m}^{0.5}$; F_{Hmax} is the maximum horizontal load, kN; t and h are the thickness and effective height of the specimen, which equal 0.12 m and 0.3 m, respectively, in this study; $f(\alpha)$ is a dimensionless factor that can be calculated by Equation (6); F_{max} in Equation (5) is the maximum applied vertical load, kN; mg is the weight of the wedge loading fixture, N; a_c in Equation (6) is effective crack length, m; $CMOD_c$ is critical crack mouth opening displacement, μm ; E is the modules of elasticity calculated by Equation (8), GPa; $h_0 = 0.006$ m, which is the thickness of the thin steel plate where the clip gauge is installed; c_i in Equation (8) is the initial compliance of the ascending branch of the F_H -CMOD curve (shown in Figure 6b); a_0 equals 0.12 m, which is the initial crack length.

In addition to the effective fracture toughness, another parameter to evaluate the fracture performance of concrete is the fracture energy G_F , which is the average energy consumed per unit area during the crack propagation process. According to its definition, the fracture energy calculation formula is as follows:

$$G_F = \frac{Q}{A_0} = \frac{\int_0^{CMOD_{max}} F_H dCMOD + mg CMOD_{max}}{t(h - a_0)} \quad (9)$$

where G_F is the fracture energy of concrete, N/m; Q is the work of fracture, J, which is the area enclosed by the F_H - $CMOD$ curve shown in Figure 6b; A_0 is the ligament area of the specimen, m^2 ; $CMOD_{max}$ is the maximum crack mouth opening displacement, m. The meanings of the remaining variables are consistent with the former equations.

2.2. Temperature–Humidity Maturity Method

2.2.1. Maturity Functions

The maturity of concrete is the extent of the development of its properties. Based on this concept, the maturity method which establishes the relationship between the concrete properties and maturity index has been proposed by scholars [42–46]. The maturity method can be used to predict the strength parameters of concrete or other properties related to the chemical reactions of the cementitious material. With the development of the maturity method, various maturity functions [46] have been proposed to calculate the maturity index. Among the different maturity functions, the equivalent age model proposed by Freiesleben Hansen and Pedersen [45] based on the Arrhenius equation has been widely used in academia and engineering. The calculation formula of the maturity index, that is, the equivalent age is as follows:

$$t_e = \sum \exp \left[\frac{E_a}{R} \left(\frac{1}{273 + T_r} - \frac{1}{273 + T} \right) \right] \Delta t \quad (10)$$

where t_e is the equivalent age at a specified temperature, hours or days; T is the average curing temperature of the concrete during time interval Δt , °C; T_r is the specified temperature, °C, generally $T_r = 20$ °C is adopted; E_a is the activation energy, J/mol, when $T \geq 20$ °C, $E_a = 33,500 - T$ J/mol; R is the gas constant, 8.3144 J/mol/K; Δt is the time interval, hours or days.

The above maturity function assumes that the cementitious material has sufficient water supply during the hydration process, so the effect of curing humidity on the strength of concrete can be neglected. However, existing studies [18,31–38] have shown that curing humidity also has a great influence on the development of concrete properties. In practice, the curing humidity of concrete is often difficult to reach a state where the relative humidity equals to 98% in a laboratory. Therefore, if the traditional maturity function is directly used to calculate the maturity index and then predict the concrete properties for a real structure, it may cause a large deviation between the predicted results and the actual concrete properties. Especially for surface concrete, the development of its properties is often greatly affected by the environmental humidity. Therefore, when using the maturity method to predict the properties of concrete, the effect of curing humidity should better be considered. Some scholars have carried out the humidity correction work based on the traditional temperature maturity functions. However, the parameter calibration of those modified functions is often complicated, hence, the direct application of these functions to the actual structure can be difficult. In recent years, in order to characterize the influence of environmental temperature and humidity on concrete fracture parameters, based on a humidity rate function, Mi et al. [30] established the following temperature–humidity maturity function, which reads:

$$t_e = \sum \frac{1}{\alpha_u - \alpha_0} \left(\frac{T}{T_r} \right)^{-m} \exp \left[\frac{E_a}{R} \left(\frac{1}{273 + T_r} - \frac{1}{273 + T} \right) \right] \times \left[H \Delta t + \frac{(1 - H)}{\gamma} \ln(1 + \gamma \Delta t) \right] \quad (11)$$

where α_u is the final degree of hydration; α_0 is the initial degree of hydration; m is defined as a microscopic coefficient; γ is the water diffusion coefficient; H is the average value of the curing humidity of the concrete during time interval Δt . The remaining variables are the same as the ones in Equation (10). To ensure the consistency of Equations (10) and (11), and for simplicity, the constants in Equation (11) are chosen as follows in this study: $\alpha_u = 1$, $\alpha_0 = 0$, $m = 0$, $\gamma = 1$. It should be noted that normally these constants can be calibrated by the best fit of the test results. However, these parameters only change the calculation

results of equivalent age and will not affect the prediction results of concrete properties, hence, they can be pre-set according to the relevant research results [30].

2.2.2. Strength–Maturity Relationships

Once the maturity index is determined, the appropriate strength–maturity relationship should be selected to establish the equation for the prediction of the concrete properties. At present, the commonly used strength–maturity relationship equations mainly have the following three forms: logarithmic, exponential and hyperbolic functions. The logarithmic relationship reads [47–49], as follows:

$$S = a + b \log(M) \quad (12)$$

where S represents the concrete properties such as strength or fracture parameters; M is the maturity index such as temperature–time factor or equivalent age; a and b are the coefficients determined by best fitting of the test results. The prediction result given by the logarithmic function would monotonically increase with the increase in the maturity index, which is a shortcoming of the form of the logarithmic function for concrete with long curing age, while the exponential and hyperbolic functions could give the ultimate stable predictions. The expressions [46,50] are as follows:

$$S = S_{\infty} e^{-\left(\frac{\tau}{M}\right)^d} \quad (13)$$

where S_{∞} represents the ultimate material properties of concrete; τ is a time characteristic parameter and d is a shape factor. They can be determined by the test results.

$$S = \frac{M}{1/A_r + M/S_{\infty}} \quad (14)$$

where A_r is the initial slope of strength–maturity curve and other variables share the same definitions as the former equations.

In this study, the above three functions were adopted to fit the test results, and it was established that for the equivalent age range considered in this paper, the logarithmic function provides the best overall fitting accuracy. Hence, for the sake of simplicity, only the predictions based on the logarithmic function are given in this article.

3. Results and Discussion

3.1. Test Results

3.1.1. Strength Test Results

Table 4 shows the test results for compressive strength f_c , splitting tensile strength f_t , axial compressive strength f_{acs} and static compressive elastic modulus E_c of environmental chamber-cured dam gallery concrete, with different ages and different curing conditions.

Figure 7 shows the variation of strength parameters of low-heat cement gallery concrete, with age, when the curing humidity is 98% and the curing temperature is 10 °C, 20 °C and 40 °C. It can be seen from Figure 7 that as the age increases, the strength parameters of concrete at different curing temperatures continue to increase; not all of the strength parameters reached their final stable values when the test age reached the designed maximum age of 90 days. This is because the low-heat cement has a relatively high content of dicalcium silicate and a slow hydration rate, so the strength can continue to increase. Taking the compressive strength at a curing temperature of 20 °C (in Figure 7a as an example), the strength of the specimen increased to 17.61 MPa within 7 days of the completion of pouring, and the growth rate was 2.52 MPa/day. With the increase in age, the growth rate of compressive strength decreased to 0.92 MPa/day, 0.43 MPa/day, 0.28 MPa/day and 0.11 MPa/day, in the several designed test age intervals of 8–14 days, 15–28 days, 29–60 days and 61–90 days. This shows that as the age increases, the growth

rate of strength gradually decreases, so the concrete strength parameters will eventually reach their stable values.

Table 4. Strength test results of environmental chamber-cured concrete at different temperatures and humidity.

Curing Conditions	Test Age (Days)	f_c (MPa) [Mean (Standard Deviation)]	f_t (MPa) [Mean (Standard Deviation)]	f_{acs} (MPa) [Mean (Standard Deviation)]	E_c (GPa) [Mean (Standard Deviation)]
T10RH98	7	11.92 (0.53)	0.98 (0.05)	7.71 (0.23)	22.27 (0.59)
	14	16.58 (0.16)	1.29 (0.07)	13.37 (0.44)	26.93 (1.67)
	28	23.93 (1.52)	1.93 (0.11)	17.77 (0.65)	31.05 (1.15)
	60	30.88 (1.50)	2.25 (0.14)	24.89 (1.77)	35.23 (1.27)
	90	33.92 (1.88)	2.51 (0.13)	27.99 (1.93)	38.35 (1.24)
T20RH98	7	17.61 (0.76)	1.26 (0.05)	11.61 (0.65)	25.50 (0.37)
	14	24.03 (1.10)	1.86 (0.05)	19.03 (1.29)	30.25 (1.10)
	28	30.05 (0.95)	2.26 (0.14)	25.69 (0.84)	35.42 (0.47)
	60	38.86 (1.93)	2.63 (0.09)	30.40 (0.77)	39.73 (0.70)
	90	42.03 (1.15)	2.93 (0.12)	35.12 (1.58)	41.15 (0.56)
T40RH98	7	24.41 (1.58)	1.76 (0.09)	18.77 (1.08)	30.02 (1.57)
	14	33.70 (1.45)	2.34 (0.11)	27.28 (0.64)	35.62 (1.48)
	28	37.43 (1.46)	2.73 (0.16)	30.68 (0.70)	39.20 (0.64)
	60	46.68 (2.27)	3.16 (0.13)	37.46 (1.18)	44.40 (1.25)
	90	50.79 (1.31)	3.31 (0.13)	39.67 (1.48)	46.77 (2.03)
T20RH60	7	15.49 (0.81)	1.27 (0.05)	10.71 (0.63)	24.25 (0.61)
	14	18.57 (0.94)	1.55 (0.04)	14.90 (0.44)	27.12 (0.14)
	28	25.05 (0.89)	1.86 (0.06)	20.77 (1.02)	31.43 (0.94)
	60	32.60 (1.50)	2.28 (0.12)	26.37 (1.29)	37.08 (1.07)
	90	36.30 (1.94)	2.51 (0.08)	29.06 (1.01)	38.82 (1.13)
T20RH30	7	14.32 (0.89)	1.24 (0.08)	10.78 (0.54)	24.00 (0.58)
	14	16.97 (0.76)	1.45 (0.08)	12.56 (0.77)	25.98 (0.57)
	28	19.89 (0.97)	1.62 (0.11)	16.29 (0.74)	28.12 (0.91)
	60	25.20 (0.81)	1.91 (0.08)	21.32 (1.41)	33.08 (0.14)
	90	30.73 (1.13)	2.23 (0.10)	24.83 (1.38)	35.03 (1.38)

On the other hand, when comparing the compressive strength of concrete under different curing temperature conditions (as in Figure 7a), we find that the higher the curing temperature, the greater the strength of the concrete at the same age, and the faster the strength growth rate. This is because a higher curing temperature speeds up the rate of hydration of the cementitious material. It should be noted that excessive curing temperatures may cause the deterioration of the internal microstructure of concrete, which will adversely affect the final strength and even have the opposite effect, a reverse growth phenomenon of concrete strength decreasing with age. It is generally believed that only when the curing temperature reaches 60 °C, will it have an adverse effect on the development of concrete strength. Therefore, in the curing temperature range of 10 °C to 40 °C, there will be no reduction in strength. It can be inferred that if the differences in early-stage pore characteristics and microcracks, that may be introduced by the difference in curing temperature, are ignored, as the age increases, the compressive strength of concrete under different curing temperatures will gradually approach its final compressive strength.

Figure 8 shows the variation of compressive strength, splitting tensile strength, axial compressive strength and elastic modulus of low-heat cement gallery concrete, with age, when the curing temperature is 20 °C and the curing humidity is 30%, 60% and 98%. It can be seen from Figure 8 that the strength development of low-heat cement concrete is closely related to its curing humidity. Under different curing humidity conditions, the growth law of each strength parameter with age is basically the same, which means that the greater the curing humidity, the faster the growth rate of strength. From the previous analysis, it is known that when the curing humidity is 98%, the compressive strength growth rate, within the interval of each test age designed in this study, would gradually decrease from

2.52 MPa/day to 0.92 MPa/day, 0.43 MPa/day, 0.28 MPa/day and 0.11 MPa/day. And when the curing humidity drops to 30%, the compressive strength growth rate in each age period decreases from 2.05 MPa/day to 0.38 MPa/day, 0.21 MPa/day, 0.17 MPa/day and 0.18 MPa/day. Comparing the above two groups, only in the last 30 days of age, from 61 to 90 days, the strength growth rate under a curing humidity of 30% is greater than one of 98% humidity. The main reason for this phenomenon is that, at the beginning, the water required for the hydration reaction of concrete cementitious materials, under low humidity conditions, may be insufficient, thus, delaying the growth of concrete strength. In the later stage, the hydration reaction of the cementitious material, under high humidity conditions, is near completion, so the strength growth rate is greatly reduced. However, although water is insufficient under low humidity conditions, there are still more cementitious materials that can participate in hydration, under conditions of insufficient water, resulting in its strength growth rate exceeding that of high humidity conditions. Within 7 days of the age of 7 to 14 days, the low-humidity curing condition, with a humidity of 30%, reduces the growth rate of compressive strength by 58.72%; hence, the influence of humidity on the strength of concrete cannot be ignored.

Figure 7. The variation of strength parameters of low-heat cement gallery concrete with age when the curing humidity is 98% and curing temperature is 10 °C, 20 °C and 40 °C: (a) compressive strength; (b) splitting tensile strength; (c) axial compressive strength and (d) static compressive elastic modulus.

Figure 8. The variation of strength parameters of low-heat cement gallery concrete with age when the curing temperature is 20 °C and curing humidity is 30%, 60% and 98%: (a) compressive strength; (b) splitting tensile strength; (c) axial compressive strength and (d) static compressive elastic modulus.

When comparing the various strength parameters of concrete under different curing humidity conditions (as in Figure 8), at the age of 7 days, the difference in concrete strength parameters, caused by the variation in curing humidity, is almost negligible. This is mainly because at an early age, the moisture inside the concrete can already meet the hydration reaction demand of the cementitious material. However, as the age increases, the adverse effects of insufficient maintenance humidity on the strength parameters gradually appear. As the age increases, the difference between the strength parameters, under different humidity conditions, shows a law that first expands and then reduces. Taking Figure 8a, the compressive strength test results of differing curing humidity, as an example, when the curing humidity is 30% and 60%, the compressive strength at the age of 7 days is reduced by 18.68% and 12.04%, respectively, than when the humidity is 98%. The reduction at 28 days expanded to 33.81% and 16.64%, while the reduction in compressive strength at 90 days, due to insufficient humidity, was reduced to 26.89% and 13.63%.

3.1.2. Fracture Test Results

Figure 9 shows the failure modes of the fracture surface of the low-heat cement concrete fracture test specimens, under different ages and curing conditions. Figure 9a–e are the fracture surfaces of low-heat cement gallery concrete at 7 days, 14 days, 28 days, 60 days, and 90 days, when the curing temperature is 20 °C and the curing humidity is 98%. Figure 9f,c,g are the fracture surfaces at 28 days, when the curing humidity is 98%

and the curing temperature is 10 °C, 20 °C and 40 °C. Figure 9c,h,i are the fracture surfaces of the low-heat cement gallery concrete, when the curing temperature is 20 °C and the curing humidity is 98%, 60% and 30%, respectively. Comparing the fracture surfaces of different ages under the same curing conditions (as in Figure 9a–e), the aggregates on the fracture surface of the wedge–stretch specimen, at the early age, are basically undamaged, and the fracture surface is relatively rough. This means that the early-age wedge-splitting specimens are mainly damaged by the interface between the aggregate and the mortar. As the age increases, the bond strength between mortar and aggregate gradually increases, and the failure mode of the fracture surface gradually changes, from interface failure to aggregate splitting. As shown in Figure 9e, the fracture surface of the 90 days specimen is relatively flat, and it can be clearly observed that there are more aggregate splits on the fracture surface.

Figure 9. Fracture surfaces of wedge-splitting specimens at different ages and under different curing conditions: (a) $T = 20\text{ }^{\circ}\text{C}$, $\text{RH} = 98\%$, Age = 7 days; (b) $T = 20\text{ }^{\circ}\text{C}$, $\text{RH} = 98\%$, Age = 14 days; (c) $T = 20\text{ }^{\circ}\text{C}$, $\text{RH} = 98\%$, Age = 28 days; (d) $T = 20\text{ }^{\circ}\text{C}$, $\text{RH} = 98\%$, Age = 60 days; (e) $T = 20\text{ }^{\circ}\text{C}$, $\text{RH} = 98\%$, Age = 90 days; (f) $T = 10\text{ }^{\circ}\text{C}$, $\text{RH} = 98\%$, Age = 28 days; (g) $T = 40\text{ }^{\circ}\text{C}$, $\text{RH} = 98\%$, Age = 28 days; (h) $T = 20\text{ }^{\circ}\text{C}$, $\text{RH} = 60\%$, Age = 28 days; (i) $T = 20\text{ }^{\circ}\text{C}$, $\text{RH} = 60\%$, Age = 28 days.

In addition to age factor, curing temperature and humidity would also affect the failure modes of the fracture surfaces of the specimens. Compare Figure 9c,f,g, the fracture surface under different curing temperature conditions at 28 days, and Figure 9c,h,i, the fracture surface under different curing humidity conditions at 28 days; with the increase in temperature and humidity, the failure mode of the fracture surface gradually changes, from the interface failure between the aggregate and the mortar to the splits of the aggregates. This is mainly because higher a curing temperature and humidity contribute to the growth of the bond strength of aggregate and mortar, which leads to a macroscopic change in the failure mode of the fracture surface.

Table 5 shows the fracture test results of low-heat cement concrete, at different ages and under different curing conditions. The equivalent fracture toughness K_{IC} and fracture energy G_F are calculated by Equations (4) and (9), respectively. The test results listed in Table 5 are the average of the test results of three specimens in the same group, and the numbers in parentheses are the standard deviations of the test results.

Table 5. Fracture parameters of gallery concrete at different ages and under different curing conditions.

Curing Conditions	Age (Days)	F_{Hmax} (kN) [Mean (Standard Deviation)]	$CMOD_C$ (μm) [Mean (Standard Deviation)]	a_c (mm) [Mean (Standard Deviation)]	K_{IC} (MPa·m ^{0.5}) [Mean (Standard Deviation)]	G_F (N/m) [Mean (Standard Deviation)]
$T = 10\text{ }^{\circ}\text{C}$, RH = 98%	7	5.65 (0.26)	87.45 (4.63)	161.21 (6.53)	1.03 (0.03)	148.18 (4.29)
	14	6.99 (0.15)	95.20 (0.85)	160.09 (2.72)	1.22 (0.02)	165.49 (3.99)
	28	7.66 (0.34)	125.67 (16.81)	155.98 (5.29)	1.29 (0.03)	195.44 (3.98)
	60	9.14 (0.33)	122.85 (6.24)	157.64 (3.87)	1.55 (0.04)	226.46 (6.89)
	90	10.19 (0.21)	132.31 (16.87)	153.46 (3.00)	1.66 (0.06)	259.27 (5.88)
$T = 20\text{ }^{\circ}\text{C}$, RH = 98%	7	6.70 (0.20)	108.36 (5.04)	157.20 (3.98)	1.18 (0.02)	167.50 (6.69)
	14	8.43 (0.46)	125.40 (9.95)	153.00 (9.10)	1.37 (0.05)	192.22 (4.29)
	28	9.48 (0.24)	88.80 (14.06)	145.63 (5.52)	1.43 (0.04)	240.82 (9.25)
	60	10.53 (0.48)	131.99 (16.95)	155.18 (6.33)	1.74 (0.03)	262.67 (3.75)
	90	11.53 (0.48)	130.49 (14.88)	154.08 (5.95)	1.86 (0.08)	282.20 (9.58)
$T = 40\text{ }^{\circ}\text{C}$, RH = 98%	7	7.46 (0.27)	123.01 (13.98)	157.68 (3.53)	1.28 (0.03)	185.92 (9.02)
	14	9.42 (0.28)	115.53 (10.41)	151.6 (5.94)	1.51 (0.05)	224.62 (2.26)
	28	10.25 (0.40)	136.50 (6.01)	155.24 (5.01)	1.70 (0.04)	257.95 (5.53)
	60	11.72 (0.54)	125.55 (9.90)	153.32 (8.12)	1.91 (0.07)	288.67 (6.76)
	90	12.57 (0.44)	115.59 (10.50)	154.12 (5.36)	2.06 (0.06)	295.30 (6.04)
$T = 20\text{ }^{\circ}\text{C}$, RH = 60%	7	6.20 (0.28)	110.26 (14.34)	158.59 (6.77)	1.08 (0.06)	158.53 (11.43)
	14	7.05 (0.12)	126.67 (29.80)	165.51 (4.85)	1.27 (0.02)	169.69 (5.60)
	28	7.79 (0.13)	141.92 (9.76)	160.29 (1.26)	1.36 (0.01)	204.82 (11.34)
	60	9.29 (0.07)	125.47 (37.99)	156.59 (2.30)	1.56 (0.05)	240.95 (0.67)
	90	9.71 (0.13)	141.17 (20.20)	159.07 (4.91)	1.67 (0.07)	249.95 (15.52)
$T = 20\text{ }^{\circ}\text{C}$, RH = 30%	7	5.95 (0.14)	86.95 (11.54)	150.43 (5.48)	0.98 (0.05)	153.92 (0.59)
	14	6.34 (0.25)	125.21 (18.87)	163.37 (5.10)	1.14 (0.02)	157.68 (13.01)
	28	7.30 (0.01)	106.82 (14.66)	154.08 (3.17)	1.19 (0.04)	174.26 (6.55)
	60	8.50 (0.45)	111.18 (20.77)	151.79 (9.67)	1.36 (0.06)	210.31 (12.15)
	90	8.73 (0.25)	108.65 (21.09)	157.29 (4.85)	1.48 (0.04)	212.25 (8.03)

Figure 10a–e is the load–crack opening displacement ($CMOD$) curves at 7 days, 14 days, 28 days, 60 days and 90 days, under different curing temperature and humidity conditions. Each curve in the figure is selected from the same group of three accompanying specimens. It can be seen from Figure 10 that for different curing conditions, the variation of the load– $CMOD$ curves are relatively similar. At the initial stage of loading, no macroscopic cracks were observed in the ligament zone of the specimen, and the load increased linearly with the increase in the crack mouth opening displacement, indicating that the concrete mainly undergoes linear elastic deformation. When $CMOD$ increases to a certain value, the load no longer increases linearly with the increase in the $CMOD$; that is, the relationship between the load and $CMOD$ changes from linear to nonlinear. In the double- K fracture theory, proposed by Xu et al. [41], this turning point is the crack initiation point, and the

corresponding load is the crack initiation load. With the further increase in the load, the crack tip enters a stable expansion stage. With the further increase in the load, the crack tip enters a stable propagation stage. At this time, the load increases nonlinearly with the increase in the opening displacement of the crack opening, until it reaches the peak load. Then, the load continues to decrease with the further increase in $CMOD$, which means that the crack has entered the instability propagation stage, and the testing machine begins to unload.

Figure 10. Load–crack mouth opening displacement curves of wedge-splitting specimens under different curing conditions: (a) $T = 10\text{ }^{\circ}\text{C}$, $\text{RH} = 98\%$; (b) $T = 20\text{ }^{\circ}\text{C}$, $\text{RH} = 98\%$; (c) $T = 40\text{ }^{\circ}\text{C}$, $\text{RH} = 98\%$; (d) $T = 20\text{ }^{\circ}\text{C}$, $\text{RH} = 60\%$; (e) $T = 20\text{ }^{\circ}\text{C}$, $\text{RH} = 30\%$.

From Figure 10a–c, it can be seen that, at a given age, the higher the curing temperature, the greater the peak load of the specimen. Taking the peak load at different curing temperatures, 90 days as an example, the curing humidity is 98% and the peak load, at a curing temperature of 40 °C, is 23.36% and 9.02% larger than the peak loads at 10 °C and 20 °C, respectively. Similarly, looking at Figure 10b,d,e, it can be seen that, at a given age, the higher the curing humidity, the greater the peak load of the specimen. When the curing temperature is 20 °C and the curing humidity is 98%, the peak load is 32.07% and 18.74% larger than the peak load at 30% and 60%, respectively. In addition, it can be seen from Figure 10 that, under a given curing temperature and humidity, the initial slope and peak load of the load–CMOD curves both increase with the increase in age. This is mainly because the higher the age, the more sufficient the hydration degree of the cement under the same curing conditions, which increases the mechanical properties and bearing capacity of the concrete.

Figure 11 shows the variation of equivalent fracture toughness and fracture energy of low-heat cement gallery concrete, with test age, when the curing humidity is 98% and the curing temperature is 10 °C, 20 °C and 40 °C. Similar to the strength test results, with the increase in age, the fracture parameters of concrete under different curing temperatures continued to increase, and none reached their final stable value at the maximum test age of 90 days, designed in this study. Taking the fracture energy at a curing temperature of 20 °C, using Figure 11b as an example, the growth rate of fracture energy is gradually reduced, from 23.93 (N/m)/day to 3.53 (N/m)/day, 3.47 (N/m)/day, 0.68 (N/m)/day and 0.65 (N/m)/day, in the designed test age intervals of 1–7 days, 8–14 days, 15–28 days, 29–60 days and 61–90 days. This indicates that the growth rate of the fracture parameter decreases greatly with the increase in age, so that the fracture parameters will gradually reach their final stable values.

Figure 11. Fracture parameters of gallery concrete under different curing temperatures: (a) equivalent fracture toughness; (b) fracture energy.

In addition, comparing the fracture parameters of concrete under different curing temperature conditions, in Figure 11, it can be seen that the higher the curing temperature, the greater the fracture parameters of concrete at the same age. This is mainly because the higher curing temperature accelerates the rate of hydration of the cementitious material, thereby promoting the development of concrete material properties. However, with the increase in age, the growth rate of fracture parameters gradually decreases, and the higher the curing temperature, the more obvious the decline in the growth rate of fracture parameters. Therefore, the difference in the fracture parameters of the concrete, at the same age, caused by the curing temperature, gradually decreases with the increase in age. If the differences in early-stage pore characteristics and microcracks, that may be introduced by the difference in curing temperature, are neglected, as the age increases, the fracture

parameters of concrete under different curing temperatures will gradually approach their final stable values.

Figure 12 shows the variation of the equivalent fracture toughness and fracture energy of gallery concrete, with age, when the curing temperature is 20 °C and the curing humidity is 30%, 60%, and 98%. It can be seen from Figure 12 that the development of the fracture parameters of gallery concrete is closely related to its curing humidity. For different curing humidity conditions, the growth trend of equivalent fracture toughness and fracture energy, with age, is basically the same. Similar to the test results of strength parameters, at the age of 7 days, since the moisture inside the concrete can already meet the hydration reaction demand of the cementitious material, the curing humidity condition has little effect on the fracture parameters of the concrete. With the increase in age, the adverse effects of insufficient curing humidity on fracture parameters gradually appear. Taking the test results of fracture energy, under different curing humidity, Figure 12b as an example, the fracture energy at the 7-day age, when the curing humidity is 30% and 60%, is reduced by 8.11% and 5.36%, respectively, compared with a humidity of 98%. At the 28-day age, this reduction rate is increased, to 27.64% and 14.95%, and the reduction rate at the 90-day age is 24.79% and 11.43%, respectively.

Figure 12. Fracture parameters of gallery concrete under different curing humidity: (a) equivalent fracture toughness; (b) fracture energy.

3.2. Maturity Method Analysis

3.2.1. Maturity Index Calculation

To carry out a unified analysis of the strength and fracture parameters of gallery concrete, under different curing temperature and humidity conditions, the temperature–humidity maturity function (Equation (11)) was adopted in this study, to calculate the equivalent age of gallery concrete at different ages, from 7 days to 90 days. The calculation results are shown in Table 6. The results show that, under the same curing humidity condition, the higher the curing temperature, the higher the equivalent age of the concrete at the same age. Furthermore, under the same curing temperature condition, when the curing humidity is insufficient, the equivalent age of the concrete of the same age is smaller.

3.2.2. Strength Parameters

Figure 13a–d shows the relationship between the test results of compressive strength, splitting-tensile strength, axial compressive strength and elastic modulus of gallery concrete, and the equivalent age, under different curing temperature and humidity conditions. The figure also shows the best fit curves and the corresponding determination coefficients. It should be noted that this section only provides the logarithmic relationship between the strength parameter and the maturity index, because the logarithmic, exponential, and

hyperbolic function fitting results of equation 12 to 14 were compared in this study and it was found that the fitting result of the logarithmic function was the best. It can be seen from Figure 13 that the experimental data points are all located near the fitting curve, and the determination coefficients are all larger than 0.97, indicating that the temperature–humidity maturity method, considering curing temperature, humidity and age, and the logarithmic relationship of strength–maturity, can well describe the growth trend of the strength parameters of low-heat cement gallery concrete, under the curing temperature of 10 °C to 40 °C and humidity of 30% to 98%.

Table 6. Equivalent age of environmental chamber-cured concrete specimens.

Test Age (Days)	Equivalent Age (Days) (RH = 98%)			Equivalent Age (Days) (T = 20 °C)	
	T = 10 °C	T = 20 °C	T = 40 °C	RH = 60%	RH = 30%
7	3.48	7.00	16.85	4.72	3.00
14	6.96	14.00	33.71	9.15	5.51
28	13.92	28.00	67.42	17.79	10.13
60	29.82	60.00	144.46	37.26	20.20
90	44.73	90.00	216.70	55.40	29.46

Figure 13. Relationship between strength parameters and equivalent age: (a) compressive strength; (b) splitting tensile strength; (c) axial compressive strength and (d) static compressive elastic modulus.

3.2.3. Fracture Parameters

Figure 14 shows the relationship between the equivalent fracture toughness and fracture energy of the gallery concrete, under different curing temperature and humidity

conditions and the equivalent age. It can be seen from Figure 14 that the coefficients of determination, for the relationship equations between the fracture parameters and the equivalent age, are greater than 0.96, indicating that the temperature and humidity maturity method and the logarithmic function can well describe the development trend of the fracture parameters of low-heat cement gallery concrete.

Figure 14. Relationship between fracture parameters and equivalent age: (a) equivalent fracture toughness; (b) fracture energy.

3.3. Determination of Real Strength and Fracture Parameters of Dam Gallery Concrete

3.3.1. Maturity Index Calculation

In engineering, the maturity of gallery concrete is constantly changing, due to the combined influence of environmental temperature and humidity, cement hydration heat release and water-cooling, which determine the development process of concrete strength and fracture parameters. To obtain the actual temperature and humidity history of the surface and interior concrete of the dam gallery, in this study, small weather stations were set up inside and outside the gallery to collect the internal and external environmental temperature and humidity, and thermometers were embedded between the top of the gallery and the cooling water pipe, to collect the internal temperature of the gallery concrete. Figure 15 shows the design layout and photograph of the thermometers, buried in gallery concrete. It is shown in Figure 15a that a total of six internal thermometers were embedded between the top gallery arch and the cooling water pipes, arranged perpendicular to the top of the gallery, and the distances from the top of the gallery were 2 cm, 5 cm, 10 cm, 20 cm, 35 cm and 50 cm; the serial numbers are from No. 1 to No. 6, respectively.

Figure 16 shows the actual hourly temperature history of the gallery concrete, measured by the above-mentioned, pre-embedded thermometers. It can be seen from the figure that the temperature of the gallery concretes, at different locations, all presented a trend of first increasing and then decreasing. During the first 2.0 days or 2.5 days, the concrete temperature rose rapidly, from a low temperature state to the peak temperature, around 24°C , and then gradually decreased due to the combined effect of water cooling, hydration heat release and ambient temperature. Comparing the temperature data measured by thermometers at different positions, in the temperature rise stage, the temperature measured by thermometers No. 1 to 6 are basically the same; in the cooling stage, the temperature measured by thermometers No. 1 to 3 are similar, and the temperature measured by thermometers No. 4 to No. 6 are basically the same, which is lower than the temperature measured by No. 1 to No. 3. If it is assumed that the internal gallery concrete has sufficient moisture for hydration, the temperature maturity method can be used to calculate the equivalent age of the internal concrete, and the strength and fracture parameters can be predicted, based on the equivalent age. Evidently, the equivalent age and mechanical

properties of the concrete close to the top of the gallery are greater than those near the cooling water pipe.

Figure 15. Arrangement of thermometers in gallery concrete: (a) design layout drawing; (b) actual buried thermometers.

Figure 16. Hourly temperature history of the gallery concretes at different positions measured by the pre-embedded thermometer.

The concrete within 2 cm of the arch surface of the gallery is a shallow surface concrete, and the development of its material mechanical properties is greatly affected by the temperature and humidity of the environment in which it is located. During the construction process, it is generally necessary to install wind-proof and heat-insulating doors at the entrance of the gallery, to reduce the possible adverse effects of external wind speed, temperature and humidity changes on the concrete materials. To quantitatively analyze the influence of temperature and humidity changes, inside and outside the gallery, on the shallow surface concrete of the gallery, this study set up small weather stations, inside and

outside the gallery, to record hourly temperature and humidity data. Figure 17 shows the temperature and humidity data, outside the gallery and inside the gallery, respectively. Looking at Figure 17a,b, in the 110 days starting from 6 November 2020, the fluctuation range of the temperature outside the gallery is significantly greater than the temperature fluctuation range inside the gallery. This shows that the temperature in the gallery is relatively stable, which can help in avoiding the possible cracking of the concrete, caused by the sudden temperature drop. In addition, from Figure 17c,d, the relative humidity inside the gallery is significantly greater than the humidity outside the gallery, and the relative humidity reaches 100% during certain periods. This is mainly due to the fact that, during the construction process, the construction unit used artificial humidification to increase the relative humidity in the gallery during certain periods. From the above analysis, it can be seen that the temperature and humidity values, and the fluctuation range inside and outside the gallery, are quite different. It is necessary to calculate the equivalent age of the gallery concrete by using the temperature and humidity history inside and outside the gallery, respectively, so as to determine the most unfavorable conditions for the development of the mechanical properties of shallow surface concrete of the gallery.

Figure 17. The variation of temperature and humidity inside and outside the gallery: (a) hourly and daily average temperature outside the gallery; (b) hourly and daily average temperature inside the gallery; (c) hourly and daily average humidity outside the gallery and (d) hourly and daily average humidity inside the gallery.

Table 7 shows the calculation results of equivalent ages for the shallow surface, surface and deep concrete of the top arch of the gallery. Among them, the equivalent age of the surface and deep concrete is calculated based on the temperature history, measured by the embedded thermometers, according to Equation (10). The equivalent age of the shallow surface concrete is calculated based on the hourly temperature and humidity data, recorded

by small weather stations, located outside and inside the gallery, according to Equation (11). It can be seen from the calculation results in Table 7 that when the actual ages are the same, the equivalent age, calculated based on the temperature data of thermometers No. 1 to No. 3, is almost the same, and greater than the equivalent age calculated based on No. 4 to No. 6. In addition, the equivalent age of the shallow surface concrete is significantly lower than the equivalent age of the surface and deep concrete, and the equivalent age, considering the influence of temperature and humidity, is smaller than the equivalent age considering only the influence of temperature.

Table 7. Equivalent age calculated by hourly temperature and humidity history.

Actual Age (Days)	Equivalent Age (Days) Calculated by Temperature History						Equivalent Age (Days) Calculated by Temperature and Humidity History			
	Measured by Thermometer						Measured by Weather Station Installed		Measured by Weather Station Installed	
	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6	Outside the Gallery	Inside the Gallery	Outside the Gallery	Inside the Gallery
3	3.25	3.23	3.22	2.99	3.08	3.01	2.53	2.58	2.16	2.42
7	7.69	7.64	7.62	7.20	7.28	7.12	5.38	6.11	4.61	5.94
14	14.79	14.65	14.56	13.80	13.80	13.44	9.68	12.27	8.24	12.04
28	27.72	27.43	27.24	25.85	25.75	25.04	19.63	23.28	16.69	22.11
60	53.60	53.08	52.81	50.36	50.38	49.28	41.53	46.91	35.07	43.21
90	75.28	74.65	74.40	71.02	71.43	70.07	61.48	68.89	51.51	64.15

3.3.2. Determination of Strength and Fracture Parameters

After the equivalent ages, shown in Table 7, are calculated, the strength and fracture parameter prediction equations, presented in Figures 13 and 14, can be substituted to obtain the compressive strength, splitting tensile strength, axial compressive strength, elastic modulus, equivalent fracture toughness and fracture energy of the gallery concrete, respectively. The calculation results are summarized in Table 8. In traditional dam structure analysis, the strength and fracture parameters of concrete, at a specific age, under standard curing conditions, are generally taken as the basic material parameters. Therefore, in order to compare the difference between the proposed method, presented in this study, and the traditional method, Table 8 shows the strength and fracture parameters of gallery concrete, under standard curing conditions, from 3 to 90 days. For the concrete within the range of 2 cm to 50 cm, for the arch of the gallery, in order to consider the most unfavorable working conditions, only the minimum equivalent age, calculated by the temperature data measured by thermometer No. 6, and its corresponding predicted strength and fracture parameters are presented. Similarly, for the shallow surface concrete, Table 8 gives the equivalent age and the prediction results of strength and fracture parameters, considering the effects of temperature and humidity. Since the temperature and humidity data for the equivalent age calculation in this study are all collected from the actual dam gallery structure, it can be considered that the predicted strength and fracture parameters calculated, based on these equivalent ages, are the real strength and fracture parameters of the gallery concrete. While the strength and fracture parameters of specific age concrete, under the standard curing conditions used in the traditional analysis process, are theoretical parameters, the deviation between the two is considered as the error between the theoretical parameter and the real parameter.

Table 8. Comparison between concrete strength and fracture parameters given by prediction model and traditional method.

Parameters	Standard Curing Conditions		Temperature Measured by Thermometer No. 6			Temperature and Humidity Inside the Gallery			Temperature and Humidity Outside the Gallery		
	Actual Age (Days)	Theoretical Values	t_e (Days)	Model Predictions	Error of Theoretical Value (%)	t_e (Days)	Model Predictions	Error of Theoretical Value (%)	t_e (Days)	Model Predictions	Error of Theoretical Value (%)
f_c (MPa)	3	10.20	3.01	10.23	−0.29%	2.42	8.22	24.09%	2.16	7.19	41.86%
	7	17.90	7.12	18.06	−0.89%	5.94	16.41	9.08%	4.61	14.11	26.86%
	14	24.20	13.44	23.83	1.55%	12.04	22.83	6.00%	8.24	19.38	24.87%
	28	30.50	25.04	29.49	3.42%	22.11	28.35	7.58%	16.69	25.80	18.22%
	60	37.43	49.28	35.64	5.02%	43.21	34.44	8.68%	35.07	32.55	14.99%
f_t (MPa)	90	41.11	70.07	38.84	5.84%	64.15	38.04	8.07%	51.51	36.04	14.07%
	3	0.97	3.01	0.98	−1.02%	2.42	0.85	14.12%	2.16	0.79	22.78%
	7	1.44	7.12	1.45	−0.69%	5.94	1.35	6.67%	4.61	1.21	19.01%
	14	1.82	13.44	1.80	1.11%	12.04	1.74	4.60%	8.24	1.53	18.95%
	28	2.20	25.04	2.14	2.80%	22.11	2.07	6.28%	16.69	1.92	14.58%
f_{acs} (MPa)	60	2.62	49.28	2.51	4.38%	43.21	2.44	7.38%	35.07	2.33	12.45%
	90	2.84	70.07	2.71	4.80%	64.15	2.66	6.77%	51.51	2.54	11.81%
	3	7.19	3.01	7.22	−0.42%	2.42	5.52	30.25%	2.16	4.65	54.62%
	7	13.71	7.12	13.85	−1.01%	5.94	12.46	10.03%	4.61	10.50	30.57%
	14	19.05	13.44	18.74	1.65%	12.04	17.89	6.48%	8.24	14.97	27.25%
E_c (GPa)	28	24.39	25.04	23.53	3.65%	22.11	22.57	8.06%	16.69	20.40	19.56%
	60	30.26	49.28	28.74	5.29%	43.21	27.73	9.12%	35.07	26.12	15.85%
	90	33.38	70.07	31.45	6.14%	64.15	30.77	8.48%	51.51	29.08	14.79%
	3	21.75	3.01	21.77	−0.09%	2.42	20.49	6.15%	2.16	19.84	9.63%
	7	26.67	7.12	26.77	−0.37%	5.94	25.72	3.69%	4.61	24.25	9.98%
K_{IC} (MPa·m ^{0.5})	14	30.69	13.44	30.45	0.79%	12.04	29.81	2.95%	8.24	27.61	11.16%
	28	34.71	25.04	34.06	1.91%	22.11	33.34	4.11%	16.69	31.71	9.46%
	60	39.13	49.28	37.99	3.00%	43.21	37.22	5.13%	35.07	36.01	8.66%
	90	41.48	70.07	40.03	3.62%	64.15	39.51	4.99%	51.51	38.24	8.47%
	3	0.95	3.01	0.96	−1.04%	2.42	0.90	5.56%	2.16	0.87	9.20%
	7	1.17	7.12	1.17	0.00%	5.94	1.13	3.54%	4.61	1.06	10.38%
	14	1.34	13.44	1.33	0.75%	12.04	1.30	3.08%	8.24	1.21	10.74%
	28	1.51	25.04	1.49	1.34%	22.11	1.45	4.14%	16.69	1.38	9.42%
	60	1.70	49.28	1.65	3.03%	43.21	1.62	4.94%	35.07	1.57	8.28%
	90	1.80	70.07	1.74	3.45%	64.15	1.72	4.65%	51.51	1.67	7.78%

Table 8. Cont.

Parameters	Standard Curing Conditions		Temperature Measured by Thermometer No. 6			Temperature and Humidity Inside the Gallery			Temperature and Humidity Outside the Gallery		
	Actual Age (Days)	Theoretical Values	<i>t_e</i> (Days)	Model Predictions	Error of Theoretical Value (%)	<i>t_e</i> (Days)	Model Predictions	Error of Theoretical Value (%)	<i>t_e</i> (Days)	Model Predictions	Error of Theoretical Value (%)
G _F (N/m)	3	136.52	3.01	136.66	−0.10%	2.42	128.01	6.65%	2.16	123.57	10.48%
	7	169.75	7.12	170.44	−0.40%	5.94	163.34	3.92%	4.61	153.38	10.67%
	14	196.93	13.44	195.34	0.81%	12.04	191.01	3.10%	8.24	176.16	11.79%
	28	224.12	25.04	219.74	1.99%	22.11	214.86	4.31%	16.69	203.82	9.96%
	60	254.01	49.28	246.29	3.13%	43.21	241.14	5.34%	35.07	232.95	9.04%
	90	269.91	70.07	260.10	3.77%	64.15	256.63	5.17%	51.51	248.03	8.82%

From the comparison results in Table 8, it can be seen that for the deep concrete of the gallery, its real strength and fracture performance were relatively close to the theoretical values used by traditional methods. This is mainly because the calculation result of the equivalent age of deep concrete is close to the actual age of concrete. Taking compressive strength as an example, when the actual age is 7, 28 and 90 days, the equivalent age calculated by the temperature history of thermometer No. 6 is 7.12 days, 25.04 days and 70.07 days, respectively. The deviations between the theoretical compressive strength and the real compressive strength, predicted by the proposed method, are all less than 10%, at -0.89% , 3.42% , and 5.84% , respectively. For the shallow surface concrete of the gallery, when the measured temperature and humidity history inside the gallery is used to calculate the equivalent age of the concrete, the actual ages of 3 days, 7 days, 28 days, and 90 days, correspond to equivalent ages of 2.42 days, 5.94 days, 22.11 days, and 64.15 days, respectively. This will result in a deviation of 24.09% between the theoretical compressive strength and the real value at 3 days, and the maximum deviations at 7, 28, and 90 days are reduced to 9.08% , 7.58% , and 8.07% , respectively. This means that when the actual age of concrete is used to determine the theoretical strength or fracture parameters of the shallow concrete of the gallery, it may lead to an overestimation of the theoretical value, resulting in insufficient structural design safety reserves. In addition, if the gallery entrance is not completely closed during the construction process, the temperature and humidity outside the gallery may affect the development of the concrete strength and fracture parameters. Table 8 also shows the equivalent ages calculated by using the external temperature and humidity data and the corresponding predicted values of the concrete mechanical parameters. Consequently, because the external temperature and humidity is lower than the internal temperature and humidity of the gallery, when the actual age is the same, the equivalent age calculated from the external temperature and humidity is lower, which leads to a greater difference between the theoretical strength and fracture parameter and their real values. For example, at 3 days, 7 days, 28 days, and 90 days, the deviations of compressive strength are 41.86% , 26.86% , 18.22% and 14.07% respectively. In actual engineering, when the strength and fracture parameters of the gallery concrete, determined by the method proposed in this article, are relatively small compared with the strength and fracture properties of the gallery concrete obtained by the traditional method, special attention should be paid to whether the strength and fracture parameters of the dam gallery concrete meet the requirements of the bearing capacity.

4. Conclusions

A determination method for the strength and fracture parameters of the dam gallery concrete, considering the environmental temperature and humidity, is proposed in this study. Based on the strength and fracture test results of gallery concrete, at different ages and under various curing conditions, the evolution equations of the mechanical parameters of gallery concrete are established, and then combined with the measured temperature and humidity data of the dam gallery structure, meaning the real strength and fracture parameters of gallery concrete at any age are determined. The main conclusions obtained in this study are as follows:

- (1) In the range of $10\text{ }^{\circ}\text{C}$ to $40\text{ }^{\circ}\text{C}$, the higher the curing temperature, the greater the strength and fracture parameters of the gallery concrete, and the faster the growth rate of the strength and fracture parameters at the same age. As the age increases, the strength and fracture parameters of concrete, under different curing temperatures, will gradually approach their final stable values.
- (2) The curing humidity has a negligible effect on the strength and fracture parameters of gallery concrete at early ages, but as the age increases, the adverse effects of insufficient curing humidity on the strength and fracture parameters gradually appear, and the difference between the material parameters under different humidity conditions demonstrates a law that first expands and then reduces. The low-humidity curing condition may result in a reduction in the strength growth rate, up to 58.72% , and

strength up to 33.81%; hence, its effect on the mechanical parameters of gallery concrete cannot be neglected.

- (3) The coefficients of determination for the relationship equations between strength and fracture parameters and equivalent ages are all greater than 0.96, indicating the temperature–humidity maturity function and logarithmic relationship between strength and maturity index can be used to quantitatively describe the growth law of strength and fracture parameters of low-heat cement gallery concrete, under conditions of curing temperatures of 10 °C to 40 °C, and relative humidity of 30% to 98%.
- (4) The method of embedding thermometers in the gallery concrete and setting up weather stations inside and outside the gallery can determine the most unfavorable environmental conditions for the development of the gallery concrete's strength and fracture parameters, which can help to correct a maximum deviation of 54.62% on mechanical parameters, calculated using actual ages.
- (5) A case study based on the real temperature and humidity data of a dam gallery shows that the development of the strength and fracture parameters in the shallow surface concrete, within 2 cm of the gallery top arch, is greatly affected by the ambient temperature and humidity.
- (6) In engineering, when the strength and fracture parameters of the gallery concrete, determined by the method proposed in this study, are much lower compared with the values obtained by the traditional method, special attention should be paid to whether the strength and fracture parameters of the dam gallery concrete meet the requirements of bearing capacity.

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