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ARTICLE

Maintenance Management Research of a Large-span Continuous Rigid Frame Bridge Based on Reliability Assessment by Using Strain Monitored Data

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ABSTRACT

When the bridge components needing maintenance are the world problem at present, and the health monitoring system is considered to be a very helpful tool for solving this problem. In this paper, a large number of strain data acquired from the structural health monitoring system (SHMS) installed on a continuous rigid frame bridge are adopted to do reliability assessment. Firstly, a calculation method of punctiform time-dependent reliability is proposed based on the basic reliability theory, and introduced how to calculate reliability of the bridge by using the stress data transformed from the strain data. Secondly, combined with "Three Sigma" principle and the basic pressure safety reserve requirement, the critical load effects distribution function of the bridge is defined, and then the maintenance reliability threshold for controlling the unfavorable load state which appears in the early operation stage of this type bridge is suggested, and then the combination of bridge maintenance management and health monitoring system is realized. Finally, the transformed stress distribution certifies that the load effects of concrete bridges practically have a normal distribution; as for the concrete continuous rigid frame bridge with C50 strength grade concrete, the retrofit reliability threshold should be valued at 6.13. The methodology suggested in this article can help bridge engineers do effective maintenance of bridges, which can effectively extend the service life of the bridge and bring better economic and social benefits.

1. Introduction

In the world, people have recognized the importance of the SHMS for monitoring large-span bridges during in construction and service. At present, the health monitoring system becomes an indispensable part of long-span bridges, of which the role is to supply safety evaluation of bridge construction and operation [1-8]. In recent years, the technology of the SHMS has got a significant improvement, mainly in sensing technology,

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sensing systems, innovative strategies for monitored data analysis etc. Ni Y.Q. [9] designed and implemented a sophisticated long-term SHMS consisting of more than 700 sensors with 16 types on the Canton Tower from construction to service. Kim M. H. et al [10] introduced a typical hull monitoring system (HMS) for ship structures, which adopts Long-based strain gauges (LBSG), motion sensors and pressure sensors. Habel W. R. and Rodrigues C. et al [11-12] presented an innovative monitoring method of the SHMS for concrete structures mainly based on the increasingly used Fibre-optic sensor technology. Although many scholars in the world have put forward many data analyzing methods and many structures are installed with SHMS, however, the real function of the health monitoring system has not been realized so far [13-17].

Due to many influence factors of the data collected from the SHMS, such as: environmental factors, large size of the data itself, instability of material properties and the structure shape, the structure load and the material resistance changing with time during the bridge construction and operation stages etc., it makes evaluate the bridge safety in bridge construction or operation stage by the data collected from the SHMS or other methods very difficult. At present, many international experts and scholars have done effective work in the area of using the information acquired from the SHMS for rational maintenance planning of gradually deteriorating structures. For developing advanced bridge management systems, taking RC bridges as an example. Thoft-Christensen [18] proposed to apply reliability theory in bridge management systems, but the paper is strongly based on the reports produced within the EU project. The international workshop [19-20] on structural reliability in bridge engineering has demonstrated the advantages of using reliability based methods in the key field of civil infrastructure systems, but the application of reliability techniques in bridge engineering has lagged behind application in other types of structural systems and there are some limitations of the reliability concepts using in bridge applications. In order to estimate the reliability distributions for bridges, Frangopol and Das [21], and Thoft-Christensen [22] defined the bridges' reliability states and proposed a procedure to estimate the reliability distribution for bridge maintenance, but they just suggested the maintenance reliability threshold of the steel- concrete composite bridge taking the value 4.6 mainly based on theory and experience. In order to shed some light on the past, present, and future of life-cycle management of highway bridges, Frangopol D. M. [23] systematically concluded the birth and growth of bridge management systems, and suggested that the limitations of current bridge management systems could be overcome by using the reliability-based approach. In the process of summary of the recent technology developments in the field of SHM and their application to large-scale bridge projects, Kim J. M. et al [24] suggested an idea of calculating the bridge reliability by using the basis reliability theory and the stress monitored data, but they lack long-term monitoring data and so they don't establish the linkage between structural health monitoring technology and the bridge inspection, maintenance and management exercises. In order to forecast the lifetime performance of a reinforced concrete bridge by the probabilistic framework, Akgul F. and Frangopol D. M. [25] explored general methods for the analysis of the bridge performance in the life cycle and applied their research achievements in more than a dozen concrete bridges located in American Crow Leader states, but there is lack of site-specific or lab test data to revise the deterioration models. For predicting the likelihood and extent of cracking for RC surfaces exposed to chloride ion attack, Stewart M. G. and Mullard J. A. [26] proposed a space and time related reliability analysis method to forecast the probability of crack and damage degree of concrete bridges under environmental erosion, but they achievements need to be used in conjunction with a life-cycle cost analysis to optimize maintenance strategies, inspection intervals and durability design specifications for RC structures. In the case that most of SHM applications focusing on damage detection, Mustafa G. and Catbas F. N. [27] paid attention to investigate the statistical pattern recognition for SHMS by using time series modeling of theory and experimental verification, but the tests are just conducted by using two different structures in laboratory conditions and didn't do a sensitivity analysis to examine the effects of different parameters of the methodology. For cost-effective monitoring planning of a structure system, Kim S. and Frangopol D. M. [28] provided an approach with a time-dependent normalized reliability importance factor (NRIF) of structural components, however, further studies are needed to develop a general framework for cost-effective life time optimal monitoring of structural systems taking into account the uncertainty. How to determine the best maintenance strategies under budget constraints, Orcesia A. D. and Frangopol D. M. [29] researched the optimal maintenance strategies based on monitoring information and shown the benefits of SHMS, however, the lifetime reliability of structures is characterized by survivor functions and the SHM data is only used to update the probability density function of time to failure through a Bayesian process.

In order to do construction safety assessment and long-term prediction of prestressed concrete bridges, Helder S. et al [30] built revised Finite Element Models and set up a long-term monitoring system, and discussed the differences between the measurements and the results obtained with the numerical model, namely the trends due to shrinkage and creep and the variations due to the temperature, but they didn't do in-depth analysis of the long-term monitoring data for bridge construction assessment. How to build reasonable resistance prediction model and load effect prediction model to predict reliability of aging bridges, Liu Y. F. et al [31] adopted the Bayesian dynamic models (BDMs) to predict the structural load effects based on the monitored data, and calculated the structural reliability indexes with First Order Second Moment method (FOSM), however, the monitored load effect data is little and the monitoring duration is in short time, and so the predicted results could not be consistent with the actual condition. How to use different methods to obtain a fast and accurate evaluation result with the SHM, Dong and Yuan [32] presented a multi-agent fusion and coordination system to deal with the damage identification by the strain distribution and structure joint failure, however, the validation study of multi-agent system should be considered in the future.

As can be seen from the above literature, the limitations of current bridge management systems can be overcome by using the reliability-based approach, but there is still lack of long-term health monitoring data to validate the above idea. In addition, there is little monitoring data from SHMS for the assessment of bridge safety at the time, and the bridge maintenance strategy by the use of SHMS during construction or operation is mainly based on expert experience and theoretical analysis. Therefore, in this paper, combined with large amount of strain monitoring data of a bridge SHMS, we presented a methodology of calculating reliability, of which the main purpose is to make the association between SHM technology and bridge inspection, maintenance and management exercises. This method is useful for bridge engineers to do bridge maintenance. The background bridge will be used as a case study for this work.

2. Illustration of the Bridge SHMS and Initial Data Processing

2.1 SHMS of the Background Bridge

The background bridge is located in Zhaoqing city of The Pearl River Delta in Guangdong province. The superstructure of the bridge main beam is a continuous box-beam system with a total of eight main piers and 7 main spans. The first span is 145.4 m long and the sixth span is 87 m long, and the 4 center spans are all 144 m long. The cross section of box girder is a single-box and single-chamber. The width of box girder top plate is 12.5 and the width of the base plate is 6.8 m. The transverse slope of the bridge deck is 2.0% and the longitudinal slope of the bridge deck is 0.15%. The heights, thickness of base plate and thickness of web plate vary from 8 m to 2.8 m (change according to 1.6 order power parabola), 1 m to 0.32 m and 0.9 m to 0.45 m respectively in cross sections from the supporting base to the mid-span. The main beam is fully prestressed concrete structure with the arrangement of vertical, horizontal and longitudinal prestress, and the prestressing tendons are $15\Phi^j 15.24\,\mathrm{mm}$ steel strand (strength: $R_y^b = 1860\,\mathrm{MPa}$), $2\Phi^j 12.7\,\mathrm{mm}$ steel strand (strength: $R_y^b = 1395\,\mathrm{MPa}$) and high strength rebar respectively.

The measuring points of the SHMS in girder locate near piers, in mid-span and in 1/4 span. The section locations are illustrated in Figure 1. The embedded locations of strain variety sensor (The sensor is shown in Figure 1) in each section are illustrated in Figure 3 with given numbers. The manufacture of the sensors is CHANGSHA KINGMACH HIGHTECHNICS CO., LTD [33]. With the given name of cross section and number, a sensor in the SHMS can be located in the girder uniquely, such as a sensor is named 3-4MID-1, which means it locates in the top plate center of the mid-span cross-section between pier 3# and pier 4#. The measuring time interval of each sensor is 1 hour. The sampling parameters of JMZX-215 type strain gauge are listed in Table 1. So far, monitoring of the bridge is still continuing and data for the past few years has been acquired.

Table 1. Sampling parameters of JMZX-215 type strain gauge

Type	Range	Sensitivity	Length	Remarks
Intelligent digital vibrating strain gauge	$\pm1500\mu\varepsilon$	$1\mu\varepsilon$	157 mm	Strain gauge embedded in concrete

Figure 1. Gauge installed inside the bridge prestressed concrete beam before casting

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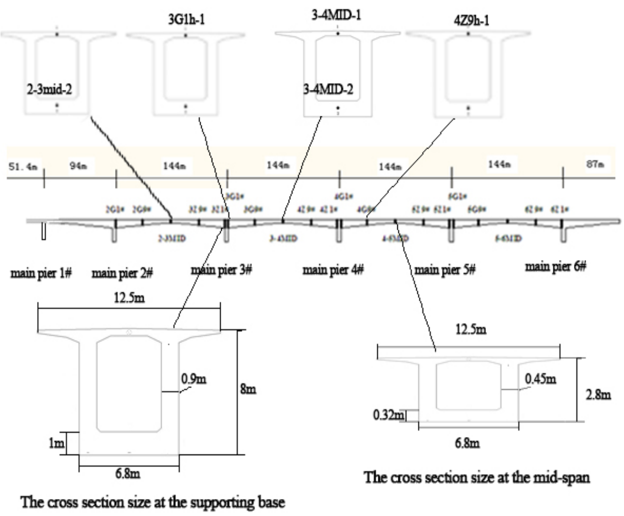

Figure 2. Cross section size and locations with sensors in the bridge of SHMS

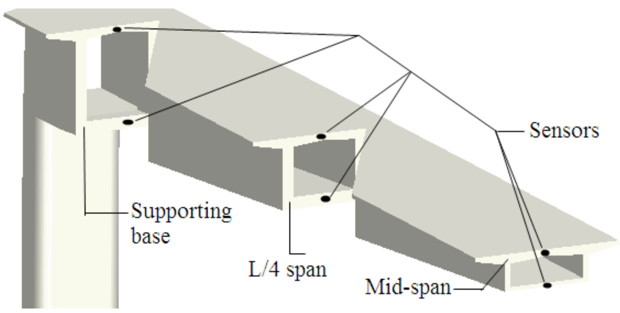

Figure 3. Typical positions of the sensors with the half-span bridge

2.2 Monitoring Data

At present, the bridge has been monitored for more than 4 years. In the paper, the data from the sensors named 3G1H-1, 3-4MID-1, 4Z9H-1 and 3-4MID-2 are used as examples, of which the monitoring time range is from March 2006 to April 2010. In fact, there are tens of thousands of data collected from the health monitoring system. The monitored data should be pre-processed firstly to delete some singular values which may be induced by strong thunders and other unexpected factors. The principle of deleting the singular values is: firstly, find out the difference between the values of each sampling point and its previous sampling point; then, if the value of the difference is greater than 200 micro-strains (engineering experience value [34]), the signal value of this sampling point is regarded as singular value and will be removed. Figure 4 shows the outline of the original data after the singular values are deleted. Some gaps appear in Figure 4, which signifies that some data are not collected due to data acquisition system fault.

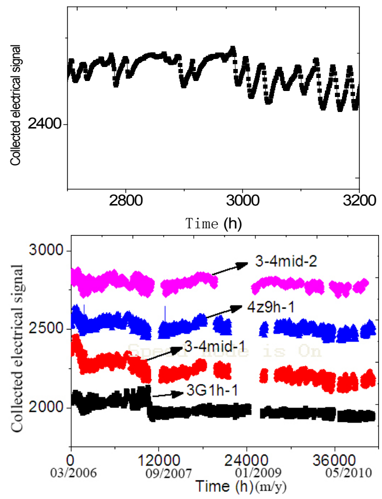

Figure 4. The data profile of the sensor

2.3 Data Pre-treatment

2.3.1 The Step of Pre-treatment

Because the monitored strain data can't be used for reliability calculation directly, and they should be implemented some processing for transforming into stress data, and then used to calculate the reliability index. The data processing method is as follows:

(1) Read the sensor initial setting value after the cast concrete is solidified. As the sensors were embedded before the concrete casting, the concrete hydration heat will produce initial strain in sensors. So, this value should be subtracted from the monitored strain value, of which the goal is to get setting values of the each sensor after the concrete is solidified.

(2) Subtract the shrinkage and creep strain values from the monitored strain value. As for those lacking of monitored data, they can use the finite element technology and build the finite element model of the bridge calibrated by field measured data to get the shrinkage and creep values. In this paper, we acquire the shrinkage and creep values corresponding to each sensor position from the long-term monitored strain data and then subtract this value from the measured strain values. The shrinkage and creep strain data extraction method is instructed in section 3.4 in detail. Of course, the extracted shrinkage and creep values are just approximate value.

(3) Subtract the thermal expansion strain value from the monitored strain value. As for the variation of environmental temperature, the monitored strain data include thermal strain. As we mentioned on the above paper, the sensor adopted in this paper can simultaneously monitor temperature. So, we can easily remove the thermal strain from the monitored strain, the elimination formula is as follows:

$$\varepsilon_r = \varepsilon - (T - T_0)(F - F_0) \tag{1}$$

In the above formula: F_0=10$\mu\varepsilon$/°C, which is the coefficient of linear expansion of the sensor steel wire as for concrete bridges; T_0 is the initial temperature; F is the coefficient of linear expansion of structure; ε is the measured strain; T is the measured temperature.

After processing, the stress data can be converted from the processed strain data by the following formula:

$$\sigma = E \cdot \varepsilon \tag{2}$$

In the formula: E is the concrete elastic modulus, and the value is: $E = 3.45 \times 10^4 MPa$ (28 days of age). This paper focuses on the performance of the bridge during early service, and the time lasts not long. During this time section, the change of modulus E tends to be stable, and so this paper neglect the effects caused by the concrete elastic modulus E.

In case of a limited number of measurements of SHMS, Bayesian methodology [31] can be used to update the structural resistance and load effects. Nevertheless, continuous monitoring over a long-term period can increase the reliability of the assessment and prediction of structural performance. In general, the damage development speed of bridges is very slow. Considering this reason and the data sample size etc., this paper determines each statistical time section of the monitoring data is 6 months, and so the data sample size of each statistical time section will reach 4000, and are enough for statistics. The derived load effects include the influence of environmental temperature, the bridge beam curve shape, the traffic loads (contain heavy loads) and resistance changing with time during the bridge operation etc. Due to many influence factors, the derived load effects are very hard to relate to absolute stresses.

According to the climate characteristics of the bridge which locates in Chinese Pearl River Delta area, then, the time statistics section has two kinds: one is called summer section, from May to October; another is called winter section, from November to April of the next year. In this paper, the statistical time starting point is 2006 May and the end point is 2010 April, and each statistical time section is named in a series A, B, C, D, E, F, G, H.

2.3.2 Extraction of creep and shrinkage deformations from the monitoring data

The shrinkage and creep data extraction method is to extract data from the same time in a day during different periods, and assumes that the temperature field is the same at the moment, detailed steps are:

(1) Select the data monitored between the time 2:00 ~ 4:00, because the traffic flow is small at this time section and the elastic strain caused by vehicle load can be ignored.

(2) We assume that the temperature of the top and base plate of the box girder cross section is the same at the selected time section. Only in this way, the influence of the non-uniform temperature stress along the bridge longitudinal and vertical direction can be eliminated.

In addition, another data extraction principle is as follows: firstly, give the time series $(t_1, t_2, \cdots t_n)$; then, due to the creep and shrinkage of concrete growth rate gradually slows down, so, when determine the time series $(t_1, t_2, \cdots t_n)$, we should make the time series firstly dense and then sparse, and make the time interval increase gradually. Give the first calculation age t_1 and the second calculation age t_2. The other time point can be calculated in the following way [35]:

$$\frac{t_i - t_1}{t_{i-1} - t_1} = 10^{1/10} \tag{3}$$

Based on the above means, we can get the shrinkage and creep values corresponding to each sensor. Here, we show the shape of the extracted shrinkage and creep values of the sensors named 2G9h-1, 3Z9h-1and 3Z9h-2 in Figure 5. Then, based on the acquired shrinkage and creep values and Interpolation method, we can get shrinkage and creep values at any time. Figure 5 is just to illustrate the extracted results of creep and shrinkage strain values. From Figure 5, we can see that the extracted shrinkage and creep values change greatly and then become basically stable after a year or so. So, we subtract 1year shrinkage and creep strain values from the monitoring data, of which the main purpose is to delete the effect of concrete shrinkage and creep. Of course, these extracted values are just approximate values. As for the sensor 2-3M ID-2 adopted in Section 4.1, the concrete shrinkage and creep strain value about 1 year take 52.3 ($\mu\varepsilon$), and we deduct this value from the monitoring data of the sensor 2-3M ID-2.

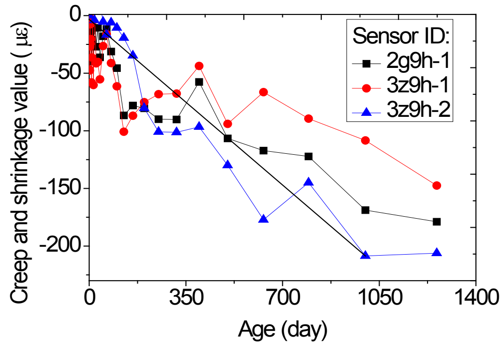

Figure 5. The measured trend of shrinkage and creep strains

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3. Main Idea of Reliability Calculation Based on Strain Monitoring Data

3.1 Calculation Procedure

According to the method adopted by Ko and Ni [24] (Ko, 2005), the failure probability P_f of the structural components can be calculated by using the member resistance R and the load effects S [36]. However, the derived result is accurate only if the random variables statistics are independent and obey the normal distribution, and then we can only get approximate results. If the probability density functions $f_R(r)$ of the resistance R and the probability density functions $f_S(S)$ of the load effects S both obey normal distribution respectively, the calculation formula of the reliability index β can be written as:

$$\beta = -\Phi^{-1}(P_f) = (\mu_R - \mu_S)/(\sigma_R^2 + \sigma_S^2)^{1/2} \tag{4}$$

In the formula: Φ^{-1} is the inverse function of the standard normal distribution; μ_R and μ_S are the mean of the resistance and load effects respectively; σ_R and σ_S are the standard deviation of the resistance and load effects respectively.

3.2 Structure Resistance Probability Density Function

For prestressed concrete bridges, as the applied stresses and stress capacities both are dependent on concrete material properties, and the correlation between the applied stresses and stress capacities is basically independent, so, the concrete strength probability distribution function is taken as the probability density function of the resistance R, which generally obeys Gaussian distribution and can be obtained by in situ material tests. As for the concrete tensile properties, An equation is applied to describe the concrete tensile strength distribution function:

$$f_{Rt}(r) = \frac{1}{\sqrt{2\pi}\sigma_t} e^{-\frac{(r-\mu_t)^2}{2\sigma_t^2}} \tag{5}$$

In the above: $f_{Rt}(r)$ are the Gaussian distribution function of the tensile strength of concrete; μ_t is the mean of the tensile strength of concrete; σ_t^2 is the variance of the tensile strength of concrete.

The concrete compressive strength distribution function alsoobeys Gaussian distribution. The mean compressive strength μ_c of concrete material is obtained by in situ test in this article. With regard to the variance σ_c^2 of the compressive strength of concrete, according to the highway reinforced concrete and prestressed concrete design specification [37], the variation coefficient can take δ_f=0.11, and then the variance σ_c^2 of the concrete compression strength used in the bridge can be obtained. In this paper, the initial compressive strength mean and standard deviation of the concrete used in the bridge can be acquired, seen in Table 2. Because of lacking concrete field test tensile strength data, so, we estimate tensile strength parameters theoretically. According to the specification [37], the relationship between the concrete mean axial tensile strength and the mean standard cube compressive strength is:

$$\mu_f = 0.88 \times 0.395 \mu_{f150}^{0.55} \tag{6}$$

Therefore, the concrete member axial tensile strength μ_{ft} can be obtained by the above formula. Also, according to the variation coefficient δ_f suggested in the specification [37], which can take the value 0.11, then, the variance $\sigma_t^2$of the concrete axial tensile strength can be acquired. Therefore, the concrete initial tensile strength mean and standard deviation used in the bridge can be derived, which is shown in Table 2.

Table 2. The concrete compressive and tensile strength mean and standard deviation values (28 days curing)

Parameters	Mean (units: MPa)	Standard deviation (units: MPa)
compressive	55.12	6.063
tensile	3.2783	0.361

In fact, due to the durability and fatigue and other factors, concrete strength changes over time. As for bridge structures, live load effects are also quite significant. In addition to the factor of durability, the material fatigue can also cause concrete strength decay, and its effect should not be ignored in practical engineering. Zhang J. L. et al [38] tested the concrete strength of more than 10 old bridges located in the Central South and the South China regions by means of hammer, core samples drilled and ultrasonic wave methods, and 703 useful data were obtained, and suggested the time-varying model of concrete compressive strength for concrete bridges given by:

$$\begin{cases} \mu_{fcu}(t) = \mu_{fcu0} \cdot \eta(t) = \mu_{fcu0} \cdot [1.378 e^{-0.0187(\ln(t)-1.7282)^2}] \\ \sigma_{fcu}(t) = \sigma_{fcu0} \cdot \zeta(t) = \sigma_{fcu0} \cdot [0.0347t + 0.9772] \end{cases} \tag{7}$$

In the above: μ_{fcu0} and σ_{fcu0} are the concrete mean and standard deviation of cube compressive strength respectively (28 days curing); $\mu_{fcu}(t)$ and $\sigma_{fcu}(t)$ are the concrete cubes mean and standard deviation functions of the compressive strength respectively after t years service. The symbols $\eta(t)$ and $\zeta(t)$ are the mean and standard deviation variation functions of the concrete compressive strength respectively. In fact, Equation (7) is revised by means of the bridge structure in situ measured data, which is under the dual roles of durability and fatigue and close to the actual bridge structure conditions. As the bridge adopted in this article lacks actual traffic statistical data and it also located in South China's Pearl River Delta region, Equation (7) is so adopted here to deduce the changing law of concrete tensile strength, combined with Equation (6) and Equation (7), this paper suggests:

$$\begin{cases}\mu_t(t)=\mu_{t0}\cdot\eta(t)^{0.55}=\mu_{t0}[1.378e^{-0.0187(\ln(t)-1.7282)^2}]^{0.55}\\ \sigma_t(t)=\sigma_{t0}\cdot\zeta(t)^{0.55}=\sigma_{t0}[0.0347t+0.9772]\end{cases} \tag{8}$$

In the above: μ_{t0} and σ_{t0} are the concrete cube mean and standard deviation of tensile strength with 28 days curing respectively; $\mu_t(t)$ and $\sigma_t(t)$ are the time-varying equations of the mean and standard deviation respectively after the concrete cube services t years.

3.3 Probability Density Function of Structural Load Effects

Jo K. M. and Ni Y. Q. [24] assumed that the probability density function of load effects of the bridge members also obey normal distribution. So, the load effects probability density function can be expressed as:

$$f_s(\mathrm{s})=\frac{1}{\sqrt{2\pi}\sigma_s}\exp\left(-\frac{(\mathrm{s}-\mu_s)^2}{2\sigma_s^2}\right) \tag{9}$$

In the above: $f_s(s)$ is the Gaussian distribution function of load effects of the bridge concrete members ; μ_s is the component load effects mean; σ_s^2 is the component load effects variance.

As the resistance and load effects of the bridge components are both obey normal distribution, therefore, the reliability index can be calculated according to Equation (4). Because the resistance R of the concrete has two probability density functions, therefore, according to Equation (4), there are two reliability indexes responding to the load effect probability density function $f_s(s)$. In view of this, the calculation methodology in this paper is: if there is $|\mu_s-\mu_c|<|\mu_s-\mu_t|$, we calculate the reliability index β_c by Equation (4), of which the meaning is that the load stress distribution is gradually close to concrete compressive with time; if not, we calculate the reliability index β_t, of which the meaning is that the load stress distribution is gradually close to tensile strength distribution with time. The calculation diagram is shown below:

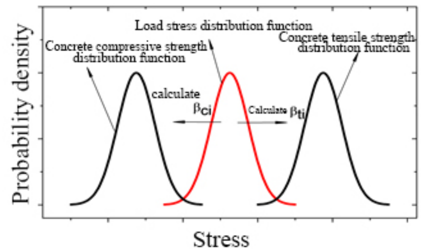


Figure 6. The reliability index calculation diagram

4. Calculation of the Reliability Index

4.1 Resistance and Load Effects Distributions

We take the monitored data collected from the sensor named 2-3MID-2 embedded in the mid-span section base plate between the bridge 2# and 3# pier for example, and we process the data according to the process method suggested in Section 3.3. The transformed strain data are presented in Figure 7.

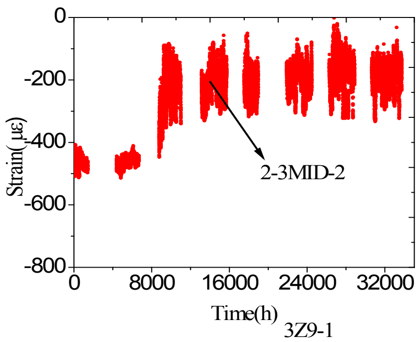
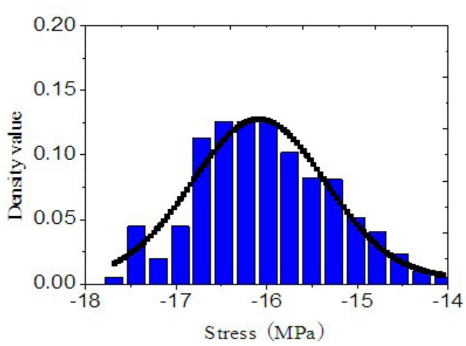


Figure 7. The strain data transformed from the monitoring data

We convert the above processed data into stress data according to Equation (2), and then do statistical analysis of the stress data, seen from Figure 7. We find that the stress data are basically normally distributed, and so we deal with the statistical data by Gaussian distribution fitting, seen in Figure 8. A vertical line in some pictures in Figure 8 with the value -3 MPa means that the stress distribution is close to the concrete tensile strength distribution.



(A) 2006.05 ～ 2006.10

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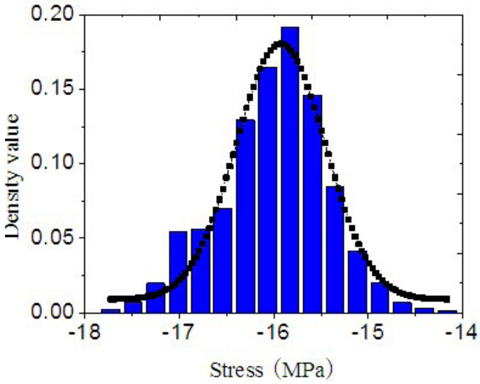

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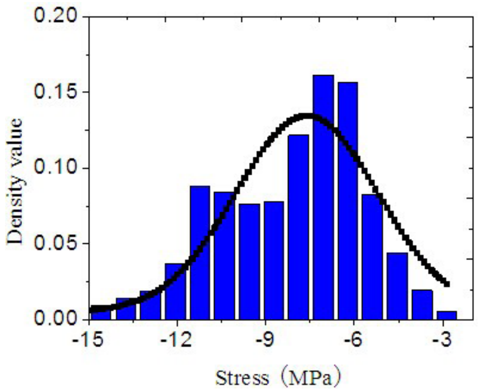

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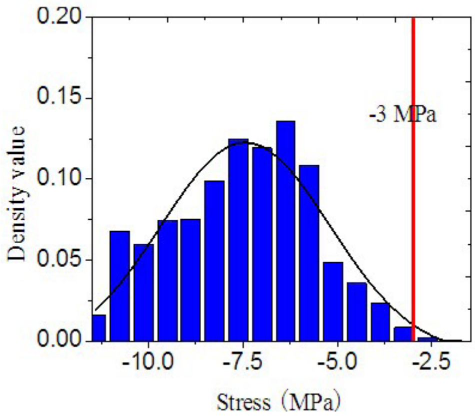

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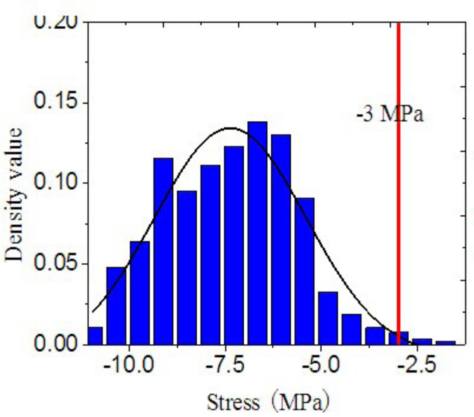

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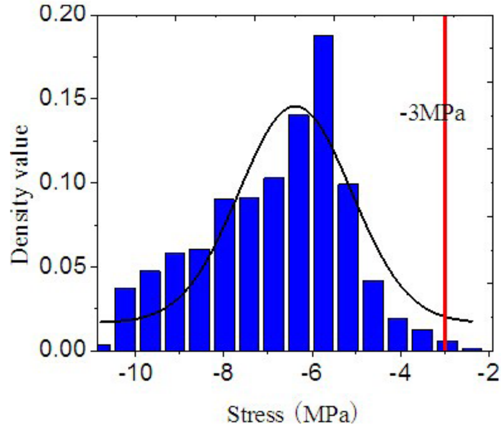

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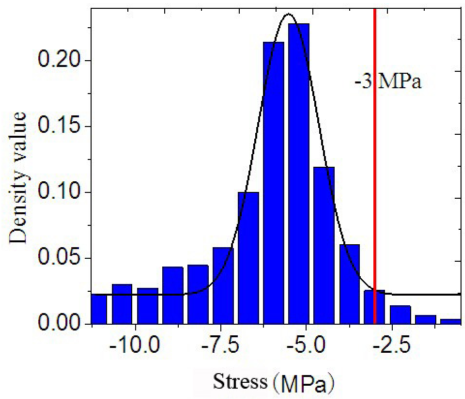
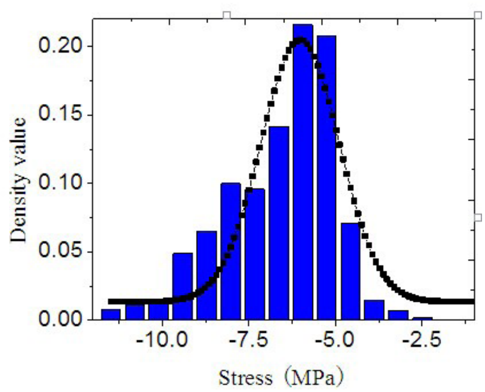


(G) 2009.05 ～ 2009.10



(H) 2009.11 ～ 2010.04

Figure 8. Gaussian distribution fitting of the stress distribution statistics

Through the above statistics analysis of the converted data, the load effect mean and standard deviation of probability distribution can be obtained for each time section, of which the standard deviation is shown in Table 3. We can see that the results of statistics analysis in Table 3 are variable, and the main reason may be that it is caused by the climate change, data loss etc.

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Table 3. The load effect mean and standard deviation of probability distribution of each time section (units: MPa)

Time series	A	B	C	D	E	F	G	H	Mean
Mean values	-16.09	-15.93	-7.62	-7.44	-7.37	-6.38	-5.52	-6.01	
Standard deviation	0.733	0.944	2.403	2.24	1.97	1.27	0.886	1.13	1.447

As can be seen in Figure 7, the load effect distribution $f_s(s)$ is gradually close to the tensile strength distribution $f_{Rt}(r)$ and then basically keep stable. Therefore, this paper only calculates β_t of the position in the mid-span base plate. Based on the methodology in the above, we can calculate the punctiform time-dependent reliability index around the sensor embedded position in the bottom plate of the mid-span cross-section, which is illustrated in Figure 9, and it shows that the reliability index β_t reduced significantly after the bridge was in service about a year's time or so. Fortunately, it remains stable a year later. As for this phenomenon, the main reason may be that the concrete creep and shrinkage strain values change rapidly in the first few years and lead to the mid-span sinking of the concrete bridge, which can be reflected in section 3.3.2, and the design of the bridge doesn't consider this factor precise enough.

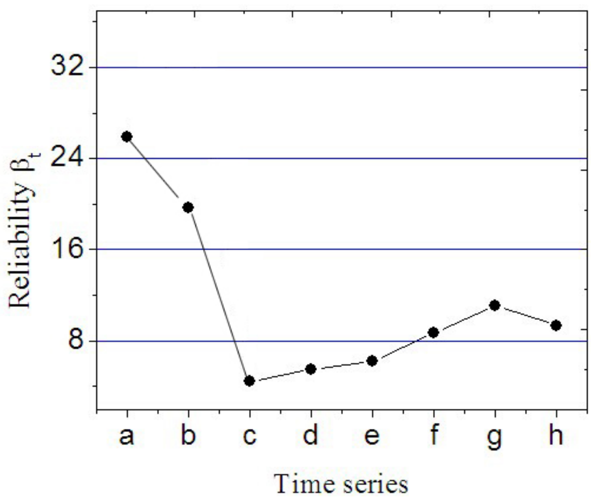

Figure 9. The reliability β_t change over time which is calculated by $f_{Rt}(r)$

As the reliability index calculated by the above proposed methodology only reflects the state around the sensors, so, we name it punctiform time-varying reliability.

4.2 Brief Introduction of "Three Sigma" Principle and Maintenance Reliability Threshold

Since the last twentieth century, the human productivity continuously develops, and the product and quality are continuously improved. In twenty-first century, the quality becomes the theme of the new century. There is firstly "Three Sigma" principle in quality management in the past, and now "Six Sigma principle" is suggested. At present, the procedure guarantee capability and management level of the majority enterprises (including construction enterprises) in the world is in the range about "Three Sigma" to "Four Sigma" [39].

"Three Sigma" principle itself is generated from normal distribution of the statistics. The normal distribution is determined by two important parameters: the mean and standard deviation. In total quality management, there is:

$$P(\mu-3\sigma < x < \mu+3\sigma) = \Phi(3) - \Phi(-3) = 0.9973 \qquad (10)$$

The above formula shows that the probability of the quality characteristic values falling without the confidence interval (μ-3σ, μ-3σ) is only 0.27%.

4.3 Determination of Maintenance Reliability Threshold

Bridge construction project is inherently a planned or under construction building products, and obsess the same quality connotation with other products, namely a set of natural characteristics to meet the need, which includes: safety, adaptability, reliability, economy and environmental suitability etc, of which the main influence factors are: the human factors, technical factors, management factors, environmental factors and social factors etc. Therefore, the idea of total quality management can also be applied on the bridge from design, construction, operation, to maintenance.

Frangopol [21] put forward 5 kinds of bridge reliability status, and assume that the bridge life can be seen as a reliable state process from the intact state (β≥9.0) to the unacceptable state (β<4.6). However, Frangopol just suggested the maintenance reliability threshold 4.6 (corresponding to the failure probability 1e-6) of the steel-concrete composite bridge mainly according to theory and experiences. Ko J. M. and Ni Y. Q. [24] just suggested an idea of calculating the bridge reliability based on the basis reliability theory and the stress monitored data, and not propose how to decide the maintenance reliability threshold with the monitored data. As for this problem, combined with the monitoring data, this paper puts forward a method to determine the maintenance reliability threshold of the prestressed concrete bridge during early operation stage.

As can be seen in Figure 8, the stress state of the mid-span base plate is gradually changed from compression to tension, and then the pressure safety reserve becomes small. Seen from Figure 8 (d) - (g), there is compressive

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stress between 2 ~ 3 MPa of the unfavorable load state which is unfavorable on the bridge, which means that the pressure safety reserves is too low and is inconsistent to the general engineering experience value requests that the pressure safety reserve is at least 2 ~ 3 MPa under the most unfavorable load conditions.

According to the above description, at present, the procedure guarantee ability and management level of most enterprises in the world are about in the range from "Three Sigma" to "Four Sigma". Therefore, this paper adopts "Three Sigma" standard management level to determine the maintenance reliability threshold of the bridge. According to the request that the pressure safety reserve is at least 2 ~ 3 MPa under the most unfavorable load condition, this paper takes the value -2 MPa. Then, the bridge maintenance reliability threshold is calculated as follows:

Firstly, according to "Three Sigma" standard and the minimum -2 MPa of pressure safety reserve requirement, this article defines a critical load effects distribution function for calculating the maintenance reliability threshold, and the calculation diagram is shown in Figure 10, in which we only consider the probability of the abnormal load effects fall in the right confidence interval [$-2MPa$, $+\infty$], and the reason is: taking into account the compressive properties of the concrete, the probability of the abnormal load effects not complying with the design requirements of falling in the left confidence interval [$+\infty$, μ-3σ] is too small and can be basically neglected. Among them, the standard deviation σ_{th} of the defined critical load distribution function is obtained by the monitoring data, and this paper takes the mean σ_{th}=1.447 from Table 3 in Section 4.1. So, based on Figure 10 and σ_{th}=1.447, we can get the mean value of the defined critical load effects distribution function, which is μ_{th}=-5.86MPa.

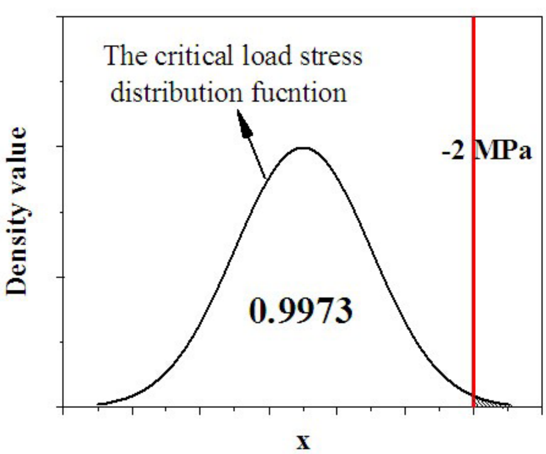

Figure 10. Diagram of the determination of the critical load effect distribution function

Secondly, based on the mean μ_{th} and standard deviation σ_{th} of the defined critical load effects distribution function, using the mean and the standard deviation of the tensile strength shown in Table 2, combined with Equation (4), we have calculated and found the corresponding critical reliability value β_{tth}=6.13, and this value is taken as the maintenance reliability threshold of the background bridge.

Actually, the maintenance reliability threshold is calculated by the concrete tensile strength in the bridge early operation. Therefore, the maintenance reliability threshold suggested in this paper is mainly aimed at the regulation of the early appeared unfavorable internal force state because of the early concrete shrinkage and creep, prestress loss etc.

However, the critical reliability value β_{tth}=6.13 should be revised, of which the main reason is that the traffic loads of each bridge is different and so leads to the calculated values of the Equation (7) and Equation (8) not precise enough.

5. Conclusions

As for the difficulties to make the bridge maintenance strategy, based on the monitored data collected from the SHMS of the background bridge, this paper put forward a calculation methodology of punctiform time-varying reliability and maintenance reliability threshold of this type bridge, and the main conclusions are as follows:

● A method is suggested for the strain monitoring data processing of SHMS (elimination of creep, shrinkage and thermal effects) to get stress data.

● The statistical analysis of the transformed stress data shows that the load effects of concrete bridges basically obey Gaussian distribution, and should be further proved based on hypothesis tests in the next research work plan.

● Based on "Three Sigma" management principle and the transformed stress data, the critical load effects distribution function of this kind bridge is suggested in this manuscript. Then, we suggest that the maintenance reliability threshold of the bridge should be valued 6.13, and finally the combination of bridge maintenance management and health monitoring system is realized.

● The next research project should focus on the bridge maintenance reliability threshold study taking into account the bridge material strength degradation.

● Some important parameter values should be revised by field test data. The methodology suggested in this paper can provide a reference for bridge engineers doing rational bridge maintenance in bridge operation.

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