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Localized reliability analysis on a large-span rigid frame bridge based on monitored strains from the long-term SHM system

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Abstract. With more and more built long-term structural health monitoring (SHM) systems, it has been considered to apply monitored data to learn the reliability of bridges. In this paper, based on a long-term SHM system, especially in which the sensors were embedded from the beginning of the construction of the bridge, a method to calculate the localized reliability around an embedded sensor is recommended and implemented. In the reliability analysis, the probability distribution of loading can be the statistics of stress transferred from the monitored strain which covered the effects of both the live and dead loads directly, and it means that the mean value and deviation of loads are fully derived from the monitored data. The probability distribution of resistance may be the statistics of strength of the material of the bridge accordingly. With five years' monitored strains, the localized reliabilities around the monitoring sensors of a bridge were computed by the method. Further, the monitored stresses are classified into two time segments in one year period to count the loading probability distribution according to the local climate conditions, which helps us to learn the reliability in different time segments and their evolvement trends. The results show that reliabilities and their evolvement trends in different parts of the bridge are different though they are all reliable yet. The method recommended in this paper is feasible to learn the localized reliabilities revealed from monitored data of a long-term SHM system of bridges, which would help bridge engineers and managers to decide a bridge inspection or maintenance strategy.

Keywords: bridges; localized reliability; long-term health monitoring system; strain preprocessing; statistics

1. Introduction

Bridge accidents may cause great loss of people's lives and properties. Once a bridge is destructed, the consequences are very serious. Actually, serious bridge accidents occurred sometimes worldwide, such as: December 15, 1967, a bridge called Kanauga Bridge (Starritt *et al*.

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1995) located in Ohio collapsed, which resulted in 46 people’s death (Fig. 1); in the past 5 years at least 17 bridge collapsed in China, which resulted in the death of more than 150 people and the injury of 88 people.

Therefore, the bridge's construction and safety monitoring in operation are equally important. Bridges’ accidents make people realize the importance of long-term structure health monitoring (SHM) systems. A long-term SHM system is an integrated system, which assembles modern sensor technology, network communication technology, damage identification theory, signal processing and analysis techniques, computer technology and other disciplines. Houser *et al*. (1997) had given the definition of the health monitoring: "Use online non-destructive sensing and analysis of structure features, including structural response, to identify the damage occurrence and determine the damage locations, and estimate the severity of damage and evaluate the consequences caused by damage." In other words, a SHM system must have the capacity of providing damage monitoring and status appraisal of structures. Though long-term SHM systems have been installed on many bridges around the world (Wang 2004, Pines *et al*. 2002, Mufti *et al*. 2002, Fujino *et al*. 2004, Wu *et al*. 2003, Yun *et al*. 2003, Ou *et al*. 2004, Wong *et al*. 2004, Thomson *et al*. 2001), there is seldom bridge SHM system having the ability to identify bridge’s damages and evaluate safety status from the monitored data, because of the shortage of corresponding methods and practices.

At present, to develop a long-term SHM system for a large-span bridge, which is really able to provide information for evaluating structural integrity, durability and reliability and establishing optimal maintenance scheme during the whole bridge life cycle, is continuously concerned by bridge engineers and managers. Therefore, how to deduce the reliability of bridges from monitored information becomes a key technology. As for the development of the theory of structural reliability evaluation, it can be divided into three stages: the exploratory phase during 40's to 50's in the 20th century, focusing on the causes of the structural failure and repair methods; 60's to 70's in the last century, focusing on the overall assessment of the building structures etc; after 70's, mainly focusing on the damage assessment, the pattern recognition and the reliability evaluation on the built structures. Reddy *et al*. (1991) took the structural failure probability as fuzzy random events, and used the probability model of the fuzzy random events as a unified model of structural reliability analysis. Engelund *et al*. (1993) adopted an important sampling technique--Monte Carlo method to calculate the structural system reliability. Thoft-Christensen *et al*. (1995) proposed to apply reliability theory in bridge management systems. Enright *et al*. (1998, 1999) applied Bayesian technology for predicting the strength and reliability of structures, in which they used the test data obtained from the engineering structure diagnosis to predict the future reliability of the bridge. Strauss *et al*. (2008) predicted the performance of structure with Bayesian using monitoring data. Frangopol *et al*. (2001) pointed out that the reliability of a structure declines continuously in its life cycle and they divided the reliability of a structure into five states according to the value of a reliability index β. With the development of SHM technology, bridge management based on reliability calculated from monitored data will become possible. Ko *et al*. (2005) recommended the possibility to calculate the reliability index beta based on long-term health monitoring data. Based on the method of simple and two ranks quadrature, Frangopol *et al*. (2008a,b) evaluate the reliability of the chords and stringers of a bridge based on monitoring data, in which the standard deviation of the monitored load effect was presented theoretically. Akgul *et al*. (2005) explored a general method for the analysis of the bridge performance in the life cycle and applied their research fruits in more than a dozen concrete bridges located in American Colorado States. Stewart *et al*. (2007) proposed a method based on space and time related reliability analysis to predict the probability of crack and damage degree in concrete bridges under environmental erosion.

From the above brief literature reviews, it is possible to analyze and assess the reliability of bridges in service by the analysis of monitored data from long-term SHM systems of bridges. The authors has constructed a concentrated SHM system, and the information of overloaded vehicles passing through a bridge was identified based the monitoring data collected with the SHM system Li *et al*. (2012). In this paper, based on extensive data of the same bridge monitored in nearly five years, a method was recommended and applied for calculating the localized reliability around the locations of the embedded sensors on a continuous rigid frame concrete bridge, which could tell not only the reliability but also its trend with apt classification of monitored data for the use of external forces. Further, with the reliability information found by the method, it is possible for bridge managers to optimize the bridge inspection and maintenance.

2. The long-term SHM system installed on the background bridge

2.1. Architecture of the SHM system

The background bridge is called Zhaoqing West River Bridge (Fig. 2), which locates in Zhaoqing City, Gudangdong Province, China. It has two lanes and the vehicles travel from Gaoyao to Zhaoqing. The superstructure of the bridge 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, the sixth span is 87 m long, and the rest 4 central spans are all 144 m long (Fig. 2). The cross segment of box girder is a single-box and single-chamber. The heights, thickness of base plate and thickness of web plate vary from 8 m to 2.8 m, 1 m to 0.32 m and 0.9 m to 0.45 m respectively in cross segments from the supporting base to the mid-span (Fig. 3).

Fig. 1 Collapse of Kanauga Bridge (Starritt *et al*. 1995)

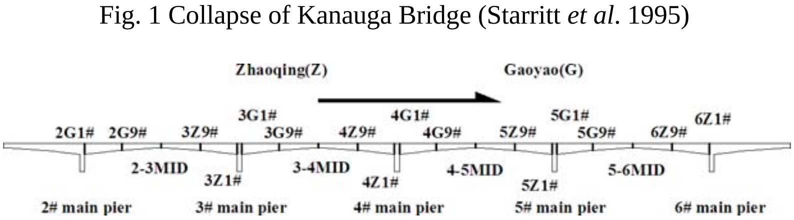


Fig. 2 The background bridge and cross sections with embedded sensors in the SHM

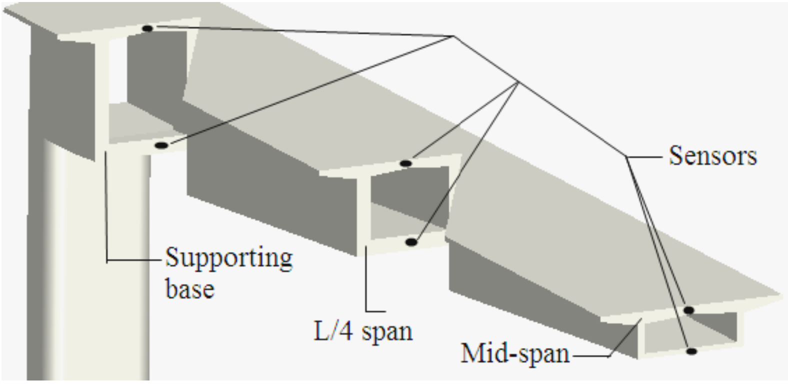


Fig. 3 Positions of the embedded sensors in monitored cross sections

The cross segments in the box girder of the bridge with the sensors locates near piers, mid-spans and 1/4 spans, and there are total 20 segments shown in Figs. 2 and 3 with given names. The embedded locations of strain sensors in each segment are illustrated in Fig. 3 with given numbers. With the given name of cross segment and number, a sensor in the long-term SHM 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#.

Sensors were installed inside of concrete during the construction of the bridge, and each sensor is apart from concrete surface at least 10 cm. The type of the sensor is a vibrating wire strain gauge called JMZX-215, in which a temperature sensor included. Fig. 4(a) is a photo of the sensor and the corresponding data acquisition module is shown in Fig. 4(b). The measurement parameters of the sensor are shown in Table 1. The sampling period of each sensor is 1 hour. So far, the monitoring of the bridge is still ongoing and data has been acquired for five years.

(a) JMZX-215 strain gauge (b) Data acquisition module

Fig. 4 Sensor and corresponding data acquisition module

Table 1 Basic parameters of JMZX-215 type strain gauge

Name	Range	Sensitivity	Gauge length	Remarks
Intelligent digital vibrating wire strain gauge	0~4000 $\mu\varepsilon$	1 $\mu\varepsilon$	157 mm	Strain gauge embedded in concrete

2.2 Basic features and data preprocessing of monitored strains with time

The bridge was put into service in October 2005. The bridge has been monitored more than 6 years until now. Here, the data during May 2006 to April 2010 collected from the sensors named 2-3MID-1 and 2-3MID-2 are selected as examples for the reliability analysis (Fig. 5(a)). Some gaps appear in the data shown in Fig. 5, because some data was not collected due to the fault of the data acquisition system.

For each monitored strain, the following preprocessing was done to insure the monitored strain can be related to the stresses directly:

(1) Firstly, extract the initial strain of concrete solidification. Before the liquid concrete pouring into mold, the initial state of the solid concrete is set when the concrete is just solidified. For the concrete hydration will produce an initial strain in each sensor, which is about 80 tensile micro strain nominally, each monitored strain will minus 80 micro strain to representative the concrete strain at that moment;

(2) Secondly, remove the effects of concrete creep and shrinkage in the monitored strain. Tang *et al*. (2007), Liu *et al*. (2008) have studied the creep and shrinkage of the concrete in the background bridge and found that the creep and shrinkage strains changed rapidly in the first year and then became stable. The creep and shrinkage strains found from the sensors named 2-3MID-1 and 2-3MID-2 are shown in Table 2;

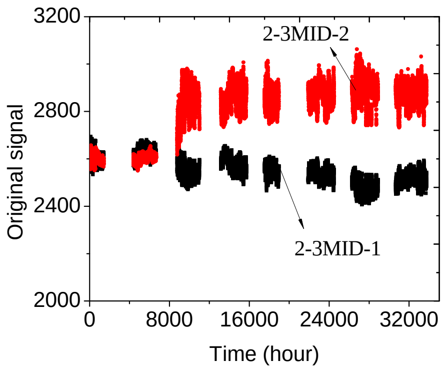
(3) Thirdly, remove the thermal strain in the monitored strain. For the long-term monitored strains, they should be coupled with the thermal strains when the temperature changed. Since a temperature sensor included in the select, and the thermal expansive coefficients of the concrete have been tested, it is easy to remove the thermal strain in each monitored strain; F and F_0 are the thermal expansive coefficients of concrete and the steel wire respectively. ε and T are the measured strain and temperature, then the strain after temperature compensation

$$\varepsilon_c = \varepsilon - \left((T - T_0)\right) \times \left(F - F_0\right) \tag{1}$$

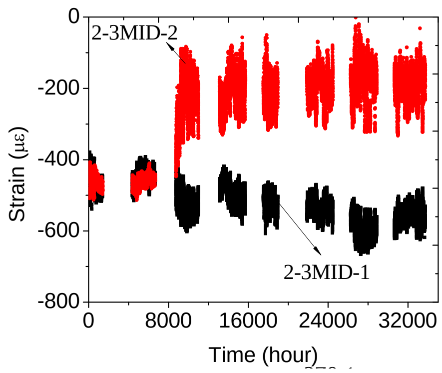
(4) Finally, eliminate the singular data. There existed singular data inevitably because of the strong thunders and other electromagnetic interference. Where if a strain has difference more than 200 micro-strain compare with the previous value, the strain will be regarded as an singular strain and will be removed from the series of data.

Table 2 Creep and shrinkage strains

Survey Time	2-3MID-1	2-3MID-2
2005	405.3	417.6
2006	510.7	469.9
Strain increment ($\mu\varepsilon$)	105.4	52.3



(a) Original data



(b) Strain

Fig. 5 Original data and real strain data collected from the SHM system

After above preprocessing, the monitored strains were transferred to stress-related strains (shown in Fig. 5(b)).

3. Theoretical considerations of localized reliability

3.1 Main Idea of calculating the reliability of the bridge

Currently, the reliability and its variation of bridges are of great concern in practical engineering application. Christensen *et al.* (1987), Mori *et al.* (1994) and Stewart *et al.* (1998) had made an attempt to explore the structural time-varying reliability, particularly in the issue of time-varying resistance. As recommended that it is possible to use monitored data from SHM system to calculate the reliability, the key is how to determine the probabilities of loads and resistance appropriately.

For the monitored data in the SHM system started from the beginning of the construction of the bridge, the data includes the effects of both dead and live loads, which can be used to deduce the probability features of load directly. The probability of resistance may be the statistics of strength of the material of the bridge accordingly.

Because the deduced reliability by the means can only reflect the reliability around the area where the sensor embedded, such reliability is defined as localized reliability in the present paper.

The failure probability P_f of structural components can be evaluated by means of considering both the resistance R and the load effect S as random variables and can be written as (Ko *et al.* 2005)

$$P_f = \int_{-\infty}^{+\infty} f_S(s)\mathrm{d}s \int_{-\infty}^{s} f_R(r)\mathrm{d}r \tag{2}$$

where f_R and f_S are probability density functions of R and S. Eq. (2) is a fundamental formulation for structural reliability used since the 1940's. If $f_R(r)$ and $f_S(r)$ are both normal distribution, the reliability index β can be calculated as follows (Frangopol *et al*. 2008a)

$$\beta = -\Phi^{-1}(P_f) = (\mu_R - \mu_S) / (\sigma_R^2 + \sigma_S^2)^{1/2} \tag{3}$$

where Φ^{-1} is the inverse function of the standard normal distribution, μ_R and μ_S are mean resistance and mean load effect, respectively, and σ_R and σ_S are standard deviations of the resistance and the load effect, respectively.

3.2 Method of calculating the probability density function of loads

In this method, the probability density function f_S of the load S (stress, etc.) is derived directly from persistent measurement from the long-term SHM system.

Now the question is how to determine the time segment of monitored data to calculate the probability density function $f_S(s)$. If the time segment is too long, the reliability can not reflect the latest reliability of the bridge, and it can not estimate the variation trend of reliability appropriately; if the time segment is too short, the quantity of data is too small to reflect the statistics of the load effect. By considering the sampling time (1 hour) of each sensor in the long-term SHM system and the damage developing speed in a practical bridge, the time segment of the monitored data for calculating each probability density function of loads is six months. In each segment, there are more than 4000 monitored data which is enough to learn the statistics of load, and the reliability of such a concrete structure would not changes acutely in a half year in early stages. There are two kinds of segments, in which one is called summer segment which is from May to October in a year, and the other segment is called winter segment from November to April of the next year. The starting point of the statistical time is in May 2006 and the ending point is in April 2010. So, there are total 8 segments which will be named in the order a, b, c, d, e, f, g, h in the coming analysis.

The variation of the probability density function $f_S(s)$ with time depends on two factors: the change of the statistics of external loads on the bridge and the structural damage. For the second factor, though the statistics of external loads keep no change, the monitored strain may increase with the development of the structural damage, and the name stresses S (transferred from monitored strains) will increase accordingly, which will change the probability density function $f_S(s)$. In another word, the change of $f_S(s)$ includes the effect of the structural weakness.

3.3 Method of calculating the probability density function of resistance

The probability distribution of the material strength is used as the probability density function of the resistance R, which is normally regarded to meet the Gaussian distribution. For the compressive and tensile properties of concrete strength are different, two equations are used to represent the compressive and tensile strength distribution functions respectively

$$f_{Rc}(r) = \frac{1}{\sqrt{2\pi}\sigma_c} e^{\frac{(r-\mu_c)^2}{2\sigma_c^2}} \tag{4}$$

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

Where $f_{Rc}(r)$ and $f_{Rt}(r)$ are Gaussian probability density distribution functions of compressive and tensile strength of concrete, other parameters are consistent with Eq. (3).

The mean compressive strength of the concrete used in the background bridge is got by in situ material test. The standard deviation of the compressive strength can be got by multiplying the mean of the compressive strength with a proportional coefficient δ_f, and the coefficient δ_f is recommended to take 0.11 in the code of JTG D62-2004 (China Communications 2004). So, the mean and standard deviation of the compressive strength are as follows

$$\begin{cases}\mu_c=-55.12(\mathrm{MPa})\\ \sigma_c=6.063(\mathrm{MPa})\end{cases} \tag{6}$$

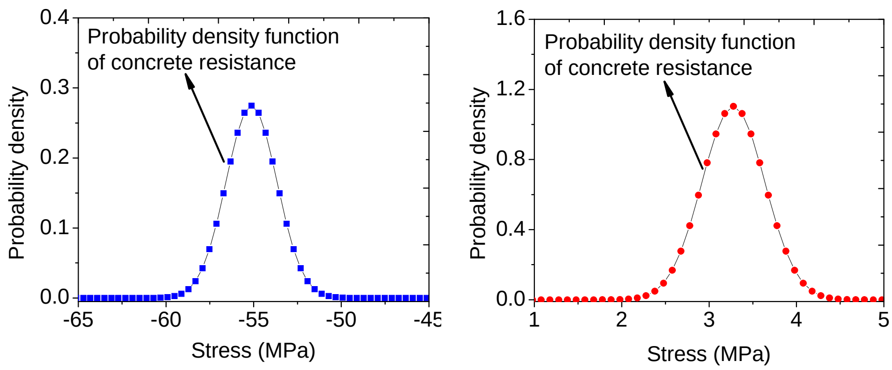
As the tensile strength of concrete has relation with the compressive strength, the tensile distribution of the concrete strength also meets the Gaussian distribution. According to "Code for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts, JTG D62-2004" (China Communications 2004), the mean value relationship between the axial tensile strength and the compressive strength is

$$\mu_t=0.88\times0.395\left|\mu_c\right|^{0.55}=3.28(\mathrm{MP}a) \tag{7}$$

As for the standard deviation of the tensile strength, it can also be got by multiplying the mean of the tensile strength with a proportional coefficient δ_f which also takes 0.11. So, the standard deviation of the tensile strength is calculated as follows

$$\sigma_t=0.11\times\mu_t=0.361(\mathrm{MP}a) \tag{8}$$

The Gaussian distributions of the concrete strengths can be seen in Fig. 6.



(a) Compressive strength (b) Tensile strength

Fig. 6 Distributions of the compressive strength and the tensile strength of concrete

In fact, due to the durability, fatigue and other factors, the concrete strength changes with time (Niu *et al*. 1995). Zhang *et al*. (2004) tested the concrete strength of more than 10 old bridges located in the Central South and the South China regions, the variations of mean and standard deviation of the compressive strength are recommended as

$$\begin{cases} \mu_c(t) = \mu_{c0} \cdot \eta(t) = \mu_{c0}[1.378 e^{-0.0187(\ln(t)-1.7282)^2}] \\ \sigma_c(t) = \sigma_{c0} \cdot \zeta(t) = \sigma_{c0}[0.0347t + 0.9772] \end{cases} \tag{9}$$

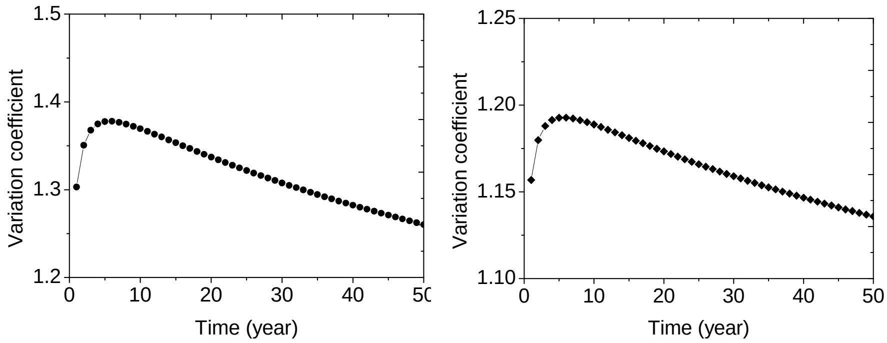
Where μ_{c0} and σ_{c0} are mean and standard deviation of the compressive strength of concrete (28 days curing) respectively and will take values shown in Eq. (6). Fig. 7(a) shows the variation of the mean of the compressive strength. The time unit in Eq. (9) is year. As for variations of the mean and standard deviation of the tensile strength, based on Eqs. (7) and (9), we assume that they obey the laws as follows

$$\begin{cases} \mu_t(t) = \mu_{t0} \cdot \eta(t)^{0.55} = \mu_{t0}[1.378 e^{-0.0187(\ln(t)-1.7282)^2}]^{0.55} \\ \sigma_t(t) = 0.11\mu_t(t) = \sigma_{t0}[1.378 e^{-0.0187(\ln(t)-1.7282)^2}]^{0.55} \end{cases} \tag{10}$$

Where μ_{t0} and σ_{t0} are mean and standard deviation of the tensile strength of concrete (28 days curing) respectively, and will take values shown in Eqs. (7) and (8). The trends of the mean of the tensile strength can be seen in Fig. 7(b).

Due to two probability densities $f_{Rc}(r)$ and $f_{Rt}(r)$ of resistance, there are two reliability indexes β_c and β_t for a loading probability density distribution $f_S(s)$ according to Eq. (2), they take expressions as

$$\begin{aligned} \beta_t &= (\mu_c - \mu_S)/(\sigma_c^2 + \sigma_S^2)^{1/2} \\ \beta_c &= (\mu_S - \mu_t)/(\sigma_t^2 + \sigma_S^2)^{1/2} \end{aligned} \tag{11}$$



(a) Compressive strength (b) Tensile strength

Fig. 7 Coefficient variation of the mean of the concrete compressive and tensile strength

4. Statistical results of monitored data and localized reliability analysis

As for large-span prestressed concrete bridges, due to the creep role of concrete and relaxation of prestressed steel strand, the dead load effect keeps changing with time in service. During the entire operation of the bridge, live load, dead load, or structural resistance, have shown a great deal of uncertainty. During the design basis, the ability of bridge structures to complete the intended function is unpredictable in the specific operating environment. Therefore, the load statistics is very useful, which can help bridge managers to learn bridge stress state and its trends in the future time.

The data collected from the sensors numbered 2-3MID-1 and 2-3MID-2 located between the main pier 2 # and the main pier 3 # (Positions of the embedded sensors shown in Figs. 2 and 3) are selected as examples to learn the localized reliability and its evolvement trends. As the monitored strain data has been preprocessed, which can be transferred to stress by

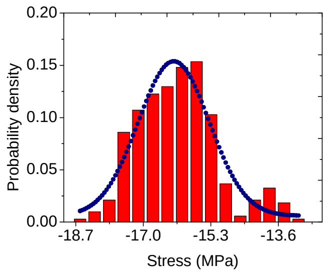
$$\sigma = E \bullet \varepsilon \tag{12}$$

where E is the final elastic modulus of concrete.

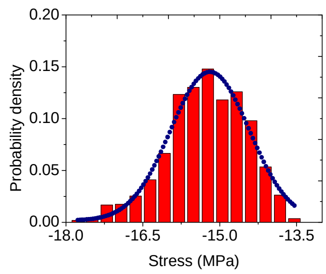
Though the probability density function of loads is not required to be a Gaussian function, here we still chose Gaussian function to depict the probability density function of loads.

4.1. The statistics of loads and the variation of the reliability on the top plate

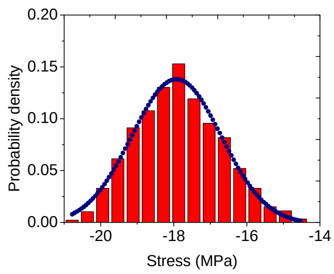
The stress distribution statistics of the measured data collected from the sensor 2-3MID-1 (Embedded in the mid-span base plate) and the Gaussian distribution fitting of the monitored data are illustrated in Fig. 8.



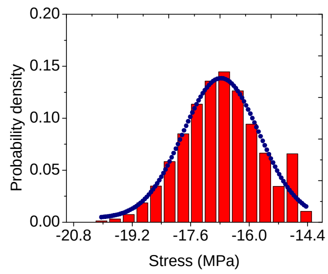
(a) Segment a: 06.05 ~ 06.10



(b) Segment b: 06.11 ~ 07.04

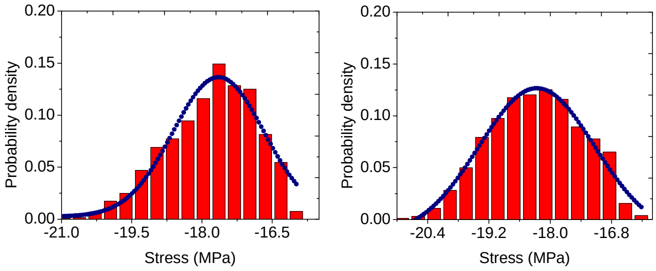


(c) Segment c: 07.05 ~ 07.10

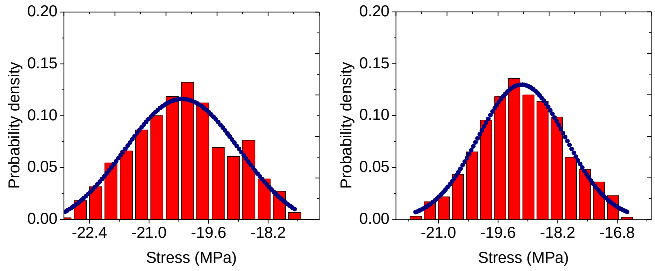


(d) Segment d: 07.11 ~ 08.04

Continued-



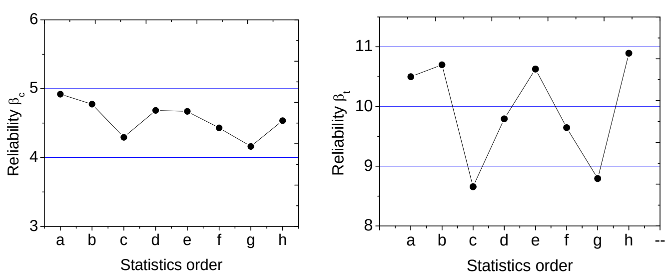
(e) Segment e: 08.05 ~ 08.10 (f) Segment f: 08.11 ~ 09.04



(g) Segment g: 09.05 ~ 09.10 (h) Segment h: 09.11 ~ 2010.04

Fig. 8 Stress statistics distribution and Gaussian distribution fitting of the data collected from sensor 2-3MID-1

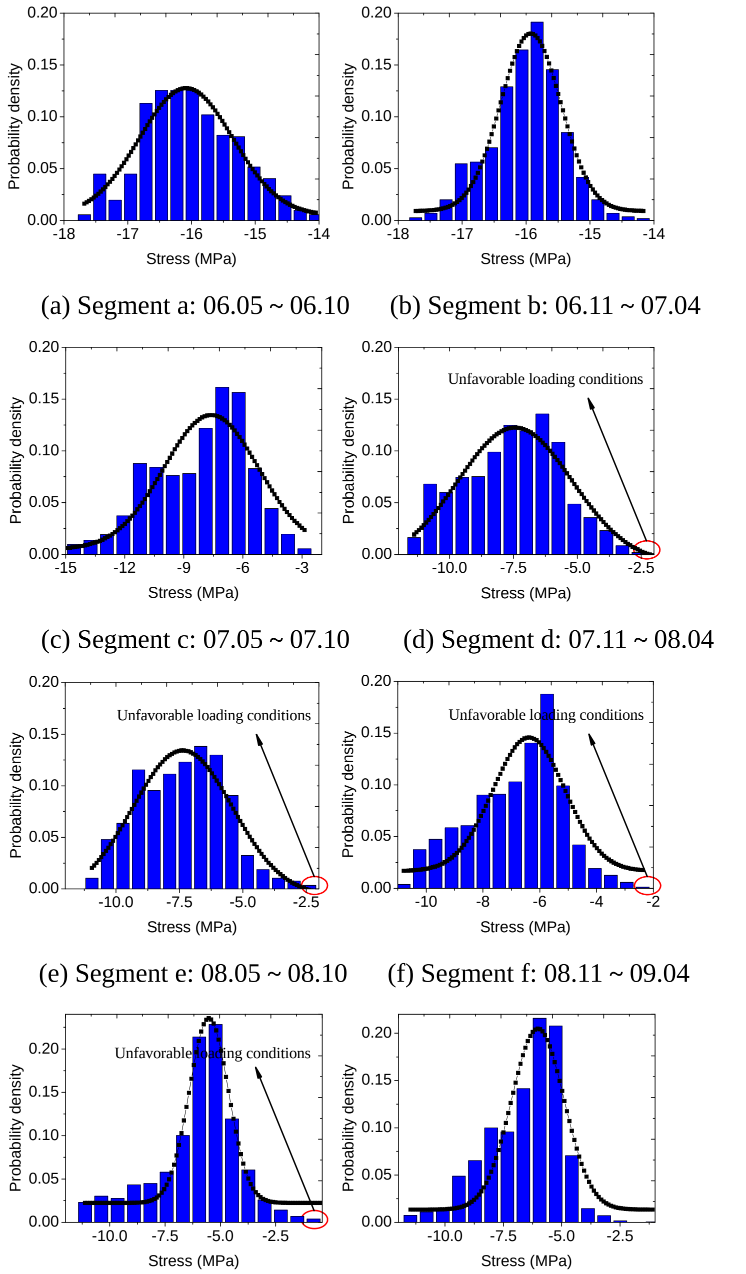
By statistical analysis of the monitored data with Gaussian distribution and strength's Gaussian distribution shown in Fig. 8, $f_S(s)$ basically obeys normal distribution. The reliability in each time segment can be calculated by Eqs. (2) and (3). Fig. 9(a) indicates that the localized reliability on the top plate in the mid-span of the bridge do decrease with time generally, and the reliability in summer is lower than that in winter, because of the thermal stress effect in summer is greater than that in winter. Fig. 9(b) illustrates that it is not necessary to consider concrete tensile failure in top plate of the bridge. However, the reliability values shown in Fig. 9(a) are a little small, for which the standard deviation of the compressive strength in Eq. (6) may be larger than the actual value.



(a) Calculated by $f_{Rc}(r)$ (b) Calculated by $f_{Rt}(r)$

Fig. 9 Localized reliability evolvement at top plate in the mid-span

4.2 The statistics of loads and the variation of the reliability on the base plate



(a) Segment a: 06.05 ~ 06.10 (b) Segment b: 06.11 ~ 07.04

(c) Segment c: 07.05 ~ 07.10 (d) Segment d: 07.11 ~ 08.04

(e) Segment e: 08.05 ~ 08.10 (f) Segment f: 08.11 ~ 09.04

(g) Segment g: 09.05 ~ 09.10 (h) Segment h: 09.11 ~ 2010.04

Fig. 10 Stress distribution statistics and Gaussian distribution fitting

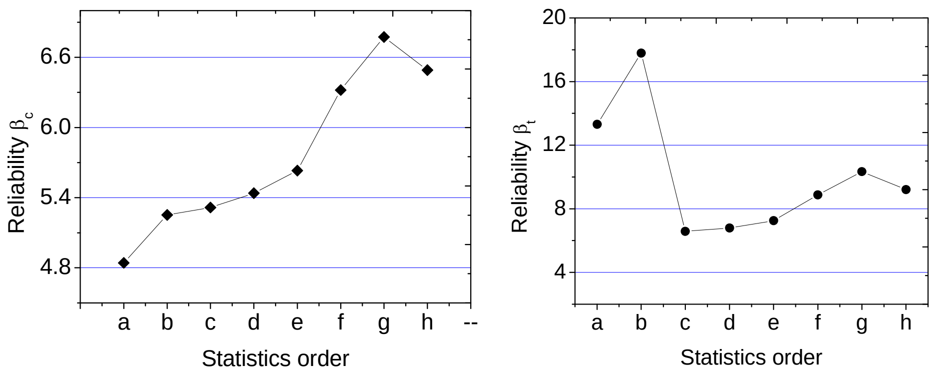
The stress statistics distribution and the fitting Gaussian distribution of the monitored data collected from the sensor 2-3MID-2 embedded in the mid-span base plate are shown in Fig. 10.

Similarly, the localized reliability at the mid-span bottom plate of the bridge can be calculated (Fig. 11). As shown in Fig. 11(b), after one year or so, the localized reliability has some obvious reduction, but luckily it kept stable after the year. In the same way, Fig. 11(a) illustrates that it is not necessary to consider compressive failure of the concrete in base plate of the bridge.

4.3 Analysis of the statistical results

Figs. 8 and 10 illustrate that the fitted curves and the measured data are in good agreement. According to the previous statistics analysis, it can be seen that the top and base plates at the mid-span between the main pier 2 # and the main pier 3 # basically work under pressure. The maximum compressive stress on the top and base plates in the mid-span is up to 23 MPa while the minimum in situ test compressive strength of C50 concrete used in the bridge is 53.6 MPa. Hence, it can be concluded that the mid-span top plate is in safe working conditions.

However, the statistical mean compressive stress of the mid-span base plate gradually decreases with time. It can be seen from Figs. 10(d)-10(g) that the actual reserved pressure of the mid-span base plate is among 2 ~ 3 MPa, which reaches the reserved minimum value of concrete. The general require is that the actual reserve pressure should be maintained with the minimum value of at least 2 ~ 3 MPa under the most unfavorable load combinations. Especially after the bridge in service a year or so, the pressure of the mid-span base plate rapidly decreased, and then changed slowly. At present, the mechanical properties of high performance concrete are still not fully grasped in engineering field. In fact, bridges are under the long-term roles of repeated loading, poor environment and erosion etc, and cracks may appear in the mid-span base plate of the bridge due to load, fatigue and corrosion effects, etc.



(a) Calculated by $f_{Rc}(r)$ (b) Calculated by $f_{Rt}(r)$

Fig. 11 Localized reliability evolvement at bottom plate in mid-span

4.4 Linkage with bridge maintenance and management

When the failure probability of structural components or segments under monitoring is obtained at regular intervals, it is easy to decide a bridge inspection or maintenance strategy because the relation between the safety index value and the required maintenance action has been established. Frangopol *et al*. (2001) proposed five kinds of bridge reliability state, and suggested that the service life of bridges could be seen as a reliable state process from the good status ($\beta > 9$) to the unacceptable status ($\beta < 4.6$). The beta 4.6 proposed by Frangopol is corresponding to failure probability 1e-6. However, the bridge-type adopted in Frangopol's article is Steel / concrete composite bridge. Zhaoqing West River Bridge is concrete bridge, of which the discreteness of material strength is larger than Steel / concrete bridge. Simultaneously, the paper mainly focuses on tensile failure at the mid-span base plate. In other words, we need to define reliability threshold β_{tth} Calculated by $f_{Rt}(r)$. As for concrete bridges, the reliability threshold β_{tth} in this article takes the value 5.2 (Corresponding to failure probability 1e-7).

In this paper, we assume that maintenance operations should be chosen to respond to significant changes of reliability states. Finally, the bridge reliability and management are combined directly. According to previous localized reliability analysis of the mid-span section of the bridge, comparatively, the reliability of the top plate is greater than of the bottom plate, which can be explained as the bending effect in the mid-span of the bridge. The reliability β_t on the base plate in the mid-span after the bridge in service about 1 year decreased rapidly with remarkable reduction as much as 8 and then changed slowly with time, which means that abnormal events happened early on the bridge structure. At the same time, the localized reliability β_t of the base plate in mid-span is low. Therefore, some special inspection should be done for the base plate in the mid-span between main pier 2 and 3.

5. Conclusions

Based on the idea of mining the hidden information from the long-term monitored data of a bridge, a method is recommended to learn the localized reliability of the bridge with the monitored strains from the long-term SHM. From the analysis, some conclusions can be drawn as:

(1) After applying the method on the background bridge, it shows that it is possible to learn the localized reliability with the long-term monitored data from a SHM system of a bridge. If the monitored data is strain related, it is necessary to preprocess the monitored data before it is used for reliability analysis;

(2) With the adapted division of the time range and load statistics, it is also possible to learn not only work stress state and the localized reliability but they evolvement trends, which will do great help to the optimal design of similar bridges, health assessment of bridges and setting good maintenance policy for bridges;

(3) By means of the reliability analysis of a cross-segments between the main pier 2 # and the main pier 3 # of the bridge, the localize reliability β on the top plate in the mid-span deceased gradually, but it keeps on a high value; the actual reserve pressure and the localized reliability on base plate becomes low, though it keeps stable in recent years, which means that some special inspection should be done for the base plate in the mid-span between main pier 2 and 3. The

reliability analysis is a little simple indeed, but it is convenient to the engineering applications.

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