Technology issues in developing structural health monitoring systems for long-span bridges

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ABSTRACT: Recent advances in sensing, data acquisition, computing, communication, data and information management have made increasing applications of long-term structural health monitoring systems in bridges. Successful implementation and operation of structural monitoring systems on bridges have been reported worldwide. An issue of great concern with bridge health monitoring systems is how to use the monitoring data for health and condition assessment of the instrumented bridges. In this paper, we explores the methodologies for bridge structural health and condition assessment and their applications to the cable-supported bridges in Hong Kong by use of long-term monitoring data. Due to the limited space, this presentation is focused on the assessment techniques rather than vibration- or modal-based approaches, including: (i) fatigue life assessment using monitoring data; (ii) condition assessment of expansion joints using monitoring data; and (iii) component failure probability assessment using monitoring data and its linkage with bridge maintenance. State-of-the-practice in bridge monitoring systems is also outlined in the present paper.

1 INTRODUCTION

The significance of implementing long-term structural health monitoring systems for large-scale bridges to secure structural and operational safety and issue early warnings on damage or deterioration prior to costly repair or even catastrophic collapse has been recognized by bridge administrative authorities. Successful implementation and operation of long-term structural health monitoring systems on bridges have been reported. So far over 40 long-span bridges (more than 100 m main span) worldwide have been instrumented with structural health monitoring systems (Ni and Hua 2004), including the Great Belt Bridge in Demark (Andersen and Pedersen 1994), the Confederation Bridge in Canada (Cheung *et al.* 1997), the Commodore Barry Bridge in United States (Barrish *et al.* 2000), the Akashi Kaikyo Bridge in Japan (Sumitro *et al.* 2001), the Seohae Bridge in Korea (Koh *et al.* 2003), the Tsing Ma Bridge in Hong Kong (Wong 2004), and the Jiangyin Bridge in Chinese mainland (Ko *et al.* 2005). In China (including Hong Kong), more than 10 long-span bridges have been instrumented with real-time monitoring systems (Ko and Ni 2005).

Bridge structural health monitoring systems are generally envisaged with the objectives to (i) validate design assumptions and parameters with the potential benefit of improving design specifications and guidelines and designing better for future similar structures; (ii) detect anomalies in loading and response and possible damage and deterioration at an early stage to ensure structural and operational safety; (iii) provide real-time information for safety assessment immediately after disasters and extreme events (e.g. earthquake, ship collision); (iv) provide evidence and instruction for planning and prioritizing bridge inspection, rehabilitation, maintenance and repair; (v) monitor repairs and reconstruction with the view of evaluating the effectiveness of maintenance, retrofit and repair works; and (vi) obtain massive amount of in-situ data for leading-edge research in bridge engineering, such as wind- and earthquake-resistant designs, new structural type and smart material applications.

Development and implementation of a structural health monitoring system to fully achieve the above objectives and benefits is still a challenge at present, and needs well coordinated research at different levels. Among them an issue of great significance is how to use the monitoring data for health and condition assessment of the bridges and how to apply the monitoring data for instructing bridge inspection and maintenance. In this paper, after an overview of some recent trends and technical difficulties in implementing long-term monitoring systems for large-scale bridges, we address the above issue by referring to several health monitoring engineering paradigms and exploring real monitoring data.

2 CURRENT STATUS IN BRIDGE HEALTH MONITORING PRACTICE

The development of structural health monitoring technology for surveillance, evaluation and assessment of existing or newly built bridges has now attained some degree of maturity. On-structure long-term monitoring systems have been implemented on bridges in Europe, United States, Canada, Japan, Korea, China and other countries (Ni and Hua 2004). Some recent trends in structural health monitoring practice for large-scale bridges are summarized below: (i) For several recent bridges such as the Hong Kong–Shenzhen Western Corridor, the Stonecutters Bridge, the Shanghai Chongming Crossing (a cable-stayed with a main span of 1200 m) and the Messina Strait Bridge (a suspension bridge with a main span of 3300 m), the design of a monitoring system is required in the tender as part of the bridge design. Integration of bridge design and monitoring system design ensures that important design engineers' concerns are reflected in the monitoring system while civil provisions for implementing a monitoring system are considered in the bridge design; (ii) The implementation of long-term monitoring systems on new bridges such as the 4th Qianjiang Bridge, the Hong Kong–Shenzhen Western Corridor, the Stonecutters Bridge and the Sutong Bridge is accomplished in synchronism with the bridge construction progress. In this way some specific types of sensors, e.g., corrosion sensors, strain gauges and fiber optic sensors can be embedded into the structure during certain bridge erection stages; (iii) The recently devised long-term health monitoring systems emphasize multi-purpose monitoring of the bridge integrity, durability and reliability. In Hong Kong, an improvement from the Tsing Ma Bridge monitoring system to the Stonecutters Bridge monitoring system is including more environment-measuring sensors such as corrosion sensors, barometers, hygrometers and pluviometers to facilitate bridge safety/reliability assessment (Wong and Hui 2004). Another example is the Sutong Bridge monitoring system that will incorporate a majority of the embedded sensors currently belonging to a foundation stability and safety monitoring system (designed for construction monitoring only) to enhance the superstructure long-term monitoring system for bridge durability assessment (Ni *et al.* 2004).

A recently emerging issue is the protection, maintenance and upgrade of the in-service monitoring systems themselves. Several bridge monitoring systems which were installed in the nineties last century have recently been found with malfunction in data acquisition units (DAUs) and sensors. Therefore, the first-generation bridge monitoring systems operating over ten years are now confronted with a problem on how to protect and maintain the systems for long-term operation. Lessons learned from the practice on existing bridge monitoring systems tell us that utmost care must be taken of the protection of DAUs, because some of them in existing bridge monitoring systems were out of operation only after a few years due to improper protection. Keeping this in mind, a special attention has been paid to designing the DAUs against a variety of environmental conditions in the Sutong Bridge monitoring project (Dong *et al.* 2005). Due to limited space of the carriageway, five DAUs in the Sutong Bridge have to be placed inside the box girders. Moreover, in order to prevent the steel girders from corrosion, structural design requires that humidity inside the sealed box is kept constant by a dehumidification system and the inner air is prohibited to circulate with the outer air. We have to meet this requirement in designing the protective system for DAUs. A stainless steel cabinet has been designed to house each DAU and to protect it from dust, temperature, humidity, and electromagnetic interference. An air conditioning system is settled inside the cabinet to accommodate severe temperature conditions in the interior of steel box girders, which may vary from below 0°C to over 60°C. To disable the air inside the cabinet from circulating with the air in the interior of the steel box, two holes of 150 mm in diameter are reserved at the lower deck flange in the vicinity of the main station and a refrigerant entry pipe and a refrigerant exit pipe are used to circulate the interior air of the cabinet with the air outside the deck box directly. Also, an isolation transformer is adopted at each DAU to protect it from lightning.

On-structure DAUs (or out-stations) are indispensable to long-term monitoring systems for long-span bridges such as the Stonecutters Bridge (with a main span of 1018 m) and the Sutong Bridge (with a main span of 1088 m). The DAUs are assigned at several locations of the bridge to collect the signals from neighboring sensors, digitize the analog signals and transmit the data into central room outside the bridge. They also have the function of short-term data storage and preliminary signal processing. For large-scale bridges with densely distributed sensors, optimal deployment of DAUs plays a significant role in assuring quality and fidelity of the acquired data. The number of DAUs required relies on the number of sensor channels in high sampling rate and in low sampling rate. Placement of DAUs is primarily dependent on the location of analog-type sensors, especially those with low-sensitivity voltage-signal output. The operation environment and the maximum allowable distance between the sensors and DAUs have to be considered to eliminate the transmission loss/noise and protect DAUs from interference. For the Sutong Bridge, the PXI/MXI-3 technique is envisaged to overcome the transmission cabling length limitation while minimizing the number of DAUs (Ni *et al.* 2004, Dong *et al.* 2005). In this design, each DAU includes a main station and

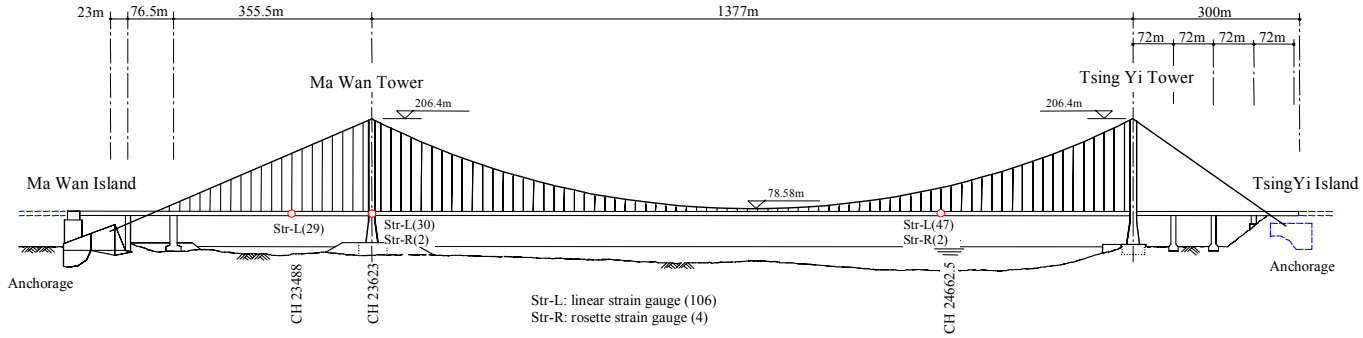


Fig. 1. Layout of strain gauges on Tsing Ma Bridge

optional sub-stations. The main station consists of a PXI instrument platform, a signal conditioning system and data acquisition modules, while the sub-station has no PXI instrument platform. Each sub-station is connected to its main station with MXI-3 interface kit and remotely controlled by the main station. Making use of the MXI-3 technique, the transmission cabling length limitation is released without employing additional DAUs.

3 FATIGUE LIFE ASSESSMENT USING LONG-TERM MONITORING DATA

Fatigue is among the most critical forms of damage that may occur in steel bridges, and therefore it is essential to assess the fatigue life of steel bridges using measurement data. A procedure for evaluating fatigue life of welded details based on long-term monitoring of strains has been proposed and applied to the suspension Tsing Ma Bridge. As shown in Fig. 1, the Tsing Ma Bridge with a main span of 1377 m is a suspension bridge carrying both highway and railway traffic in Hong Kong. After completing its construction in 1997, the bridge has been instrumented with a long-term health monitoring system comprising about 300 sensors permanently installed on the bridge (Wong 2004). As part of this monitoring system, 110 strain gauges were installed to measure strains at deck cross-sections and bearings as shown in Fig. 1. Most strain gauges are attached to the critical parts for fatigue which were identified during the design of the monitoring system.

Fig. 2 shows the location of strain gauges on a rail track section (CH 24662.50), where 42 linear strain gauges and two sets of rosette strain gauges are installed. Fig. 3 illustrates the daily strain time histories measured in two typical days by a strain gauge (SSTLS-13) on this section with a sampling rate of 51.2 Hz. It is seen that the strain time histories measured in different days are similar. After finding peaks and valleys (turning points) in the strain time history, a rainflow counting program is used to obtain the stress spectrum for each day as shown in Fig. 4. Because of similarity in the stress spectra obtained from different days, it is reasonable to average the stress spectra over a proper number of days to obtain a 'standard daily stress spectrum' which will be used to predict the fatigue life. Fig. 5 shows such obtained standard daily stress spectrum.

With the standard daily stress spectrum, the fatigue life can be estimated by the Palmgren-Miner rule after determining the class of welded detail under monitoring, the S-N relationship and the Miner's summation. The classification of welded details can be obtained from specifications (BSI 1980, AASHO 1990, Cuninghame 1990). The S-N relationship is described by

$$N \times \sigma_r^m = K_0 \times \Delta^d \tag{1}$$

where N is the predicted number of cycles to failure for a stress range σ_r; K_0, Δ and m are constant parameters depending on the class of the detail; d is

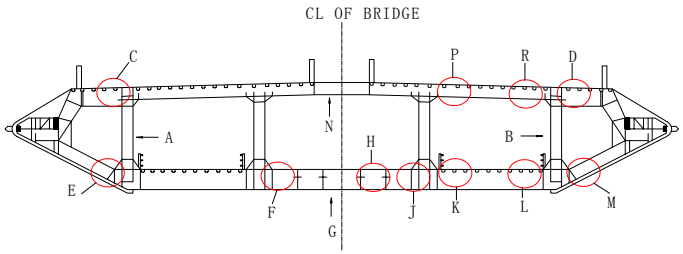
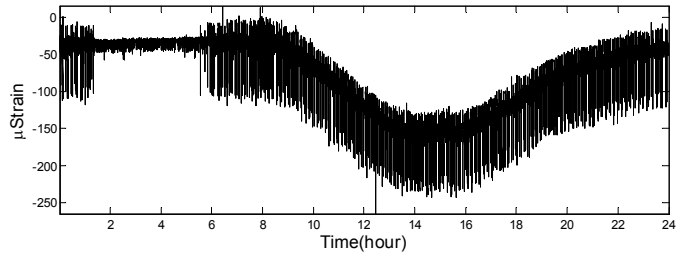
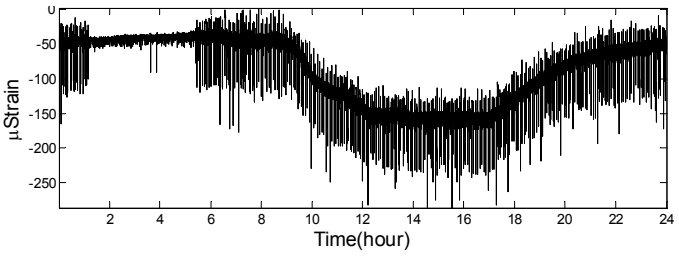


Fig. 2. Location of strain gauges on a deck cross-section

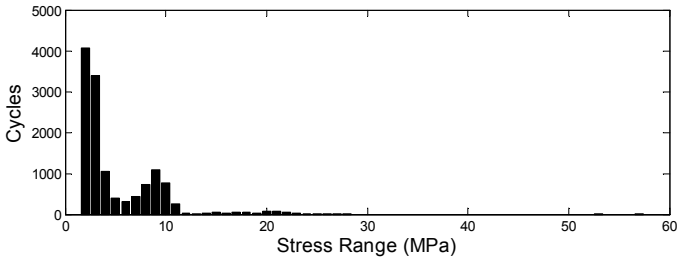


(a) 24 January 1999



(b) 8 August 1999

Fig. 3. Strain time history of detail SSTLS-13



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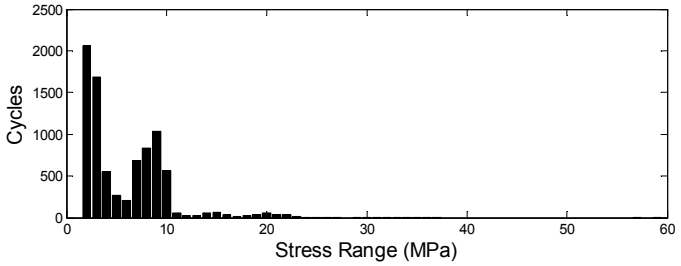


(b) 8 August 1999

Fig. 4. Measured daily stress spectrum

called the probability factor which is taken as 2 in the present study corresponding to a failure probability of 2.3% and a standard σ_r-N design curve. For the detail with its measured daily stress spectrum shown in Fig. 5, the parameters K_0, Δ and m are determined as K_0=1.23×10^{12}, Δ=0.592, m=3.0, and the corresponding S-N curve is obtained as illustrated in Fig. 6.

By combining the standard daily stress spectrum with S-N relationship and using the Miner's summation, the fatigue life F (years) for a specific detail is evaluated as

$$F = \frac{D_f}{\sum \frac{n_i}{N_i} \times 365} \tag{2}$$

where D_f is the threshold value of fatigue limit, n_i is the number of cycles for the ith stress range in the daily stress spectrum, and N_i is the number of cycles to failure for the ith stress range in the S-N curve.

The effect of stress distribution factor on the fatigue life has been studied. The real stress at the toe of a welded detail (hot-spot stress) may be much higher than the stress at the location monitored by the strain gauge (nominal stress) due to stress concentration and residual stress. Although the laboratory tests obtaining the parameters in σ_r-N formula

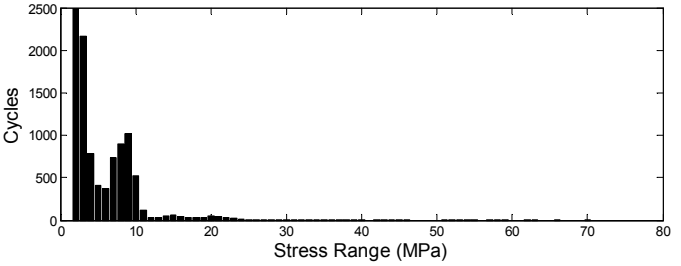


Fig. 5. Standard daily stress spectrum

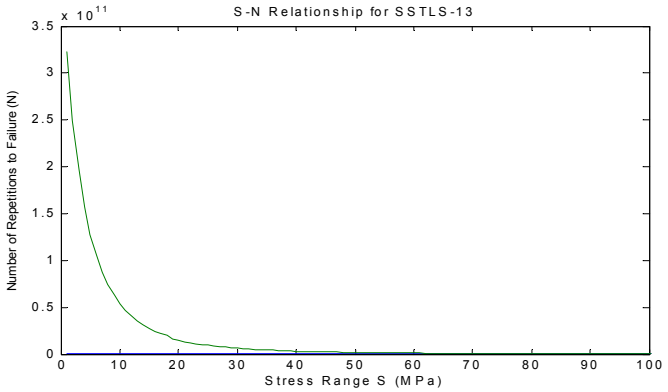


Fig. 6. S-N relationship of detail SSTLS-13

have considered nominal stress as reference stress, actual distance between the installation location of a strain gauge and the welded joint in a bridge may be much longer than that (e.g. 15mm) in the laboratory tests. Hence, a factor needs to be introduced to take into account the stress distribution in the region surrounding the strain gauge and the monitored weld joint. Table 1 shows the fatigue life prediction results of the aforementioned detail (SSTLS-13) without considering the stress distribution factor (γ_s=1.0) and under different stress distribution factors. It is observed that the predicted fatigue life is considerably influenced by the stress distribution factor. The value of γ_s for detail SSTLS-13 is estimated through finite element analysis to be about 1.15.

Table 1. Effect of stress distribution factor on fatigue life

Stress distribution factor γ_s	1.00	1.05	1.10	1.15	1.20
Estimated fatigue life (year)	679	541	435	355	293

4 ASSESSMENT OF EXPANSION JOINTS USING LONG-TERM MONITORING DATA

Expansion joints are important components in bridge structures which are used to accommodate bridge movements due to creep and shrinkage of concrete, temperature fluctuations, traffic loadings, and uneven settlement without imposing significant secondary stress to the superstructure (Dornfile 2000). A long-term structural health monitoring system for a large-scale bridge usually includes the measurement of displacement at expansion joints and temperature on one or several deck cross-sections (Cheung *et al.* 1997, Aktan *et al.* 2002, Wong 2004, Ku *et al.* 2005). A procedure for verification and assessment of expansion joints making use of long-term monitoring data has been proposed and applied to the cable-stayed Ting Kau Bridge. Following this procedure, a normal correlation pattern between the effective temperature and thermal movement is first established using the measurement data of expansion joint displacement and bridge temperature. Alarms will be

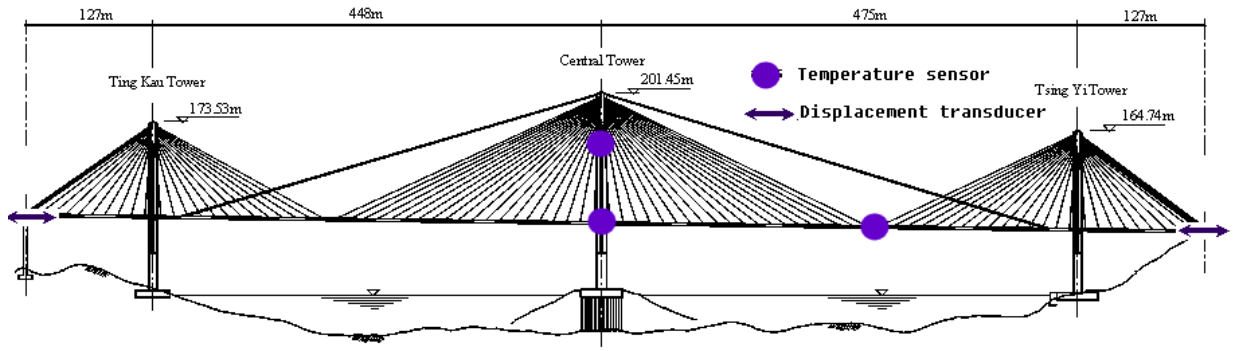


Fig. 7. Layout of displacement and temperature sensors on Ting Kau Bridge

raised if a future pattern deviates from this normal pattern. With the established correlation pattern, the expansion joint displacements under the maximum and minimum design temperatures are predicted and compared with the design allowable values for validation. The extreme temperatures for a certain return period are also predicted using the measurement data for design verification. Then the annual or daily average accumulative movements experienced by expansion joints are estimated from the monitoring data for comparison with the expected values in design. Because the service life and interval for replacement of expansion joints heavily depend on the accumulative displacements, an accurate prediction of the accumulative displacements will provide a robust basis for determining a reasonable interval for inspection or replacement of expansion joints.

The Ting Kau Bridge as shown in Fig. 7 is a multi-span cable-stayed bridge in Hong Kong with three towers supporting two main spans of 448 m and 475 m respectively and two side spans of 127 m each. After completing its construction in 1998, the bridge has been instrumented with a long-term structural health monitoring system including more than 230 sensors (Wong 2004). The displacement transducers have been used for the measurement of displacements at two expansion joints located at two ends of the continuous deck. A total of 83 temperature sensors have been installed at different locations of the bridge to measure: (i) steel-girder temperature, (ii) temperature inside concrete deck, (iii) temperature in tower legs, (iv) temperature in asphalt pavement, and (v) atmosphere temperature. The deployment of displacement transducers and temperature sensors on the bridge is illustrated in Fig. 7. Fig. 8 shows the location of temperature sensors on a deck cross-section nearly in the middle of the Tsing Yi main span and Fig. 9 shows the layout of a displacement transducer at the Tsing Yi abutment.

Available for the present study are one-year measurement data from the displacement and temperature sensors. Hourly average values of the temperatures and displacements are first obtained. Temperature in a deck cross-section can be divided into effective temperature and differential temperature (Li *et al.* 2004). The differential temperature refers to temperature differences between the top surface and other levels in a cross-section, which results in the temperature-induced internal forces and bending stresses in continuous structures. The effective temperature is obtained by weighted average of the temperatures measured at various levels in a cross-section, which produces to axial elongation/ shortening if the cross-section is unrestrained. The weighting is a ratio of the area of cross-section at various levels to the area of whole cross-section. When the temperatures over a cross-section are measured, the effective temperature can be obtained by

$$T_e = \frac{1}{A}\iint_A T(x,y)dxdy \tag{3}$$

where A is the area of the cross-section; $T(x, y)$ is a two-dimensional temperature over the cross-section. According to the provision (HKHDSD 1997), the effective temperature on a deck cross-section should be estimated by using only the measured temperatures in structural components exclusive of those in asphalt and air. Fig. 10 illustrates such obtained effective temperature for the deck cross-section shown

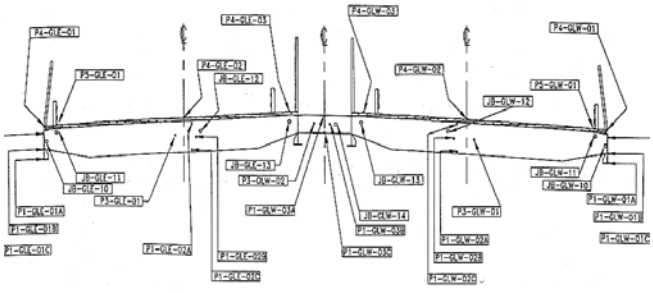


Fig. 8. Temperature sensors on deck cross-section

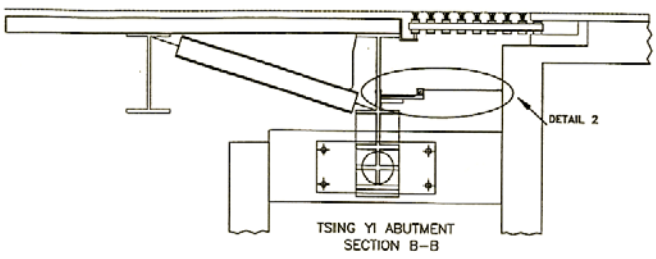


Fig. 9. Displacement transducer at Tsing Yi abutment

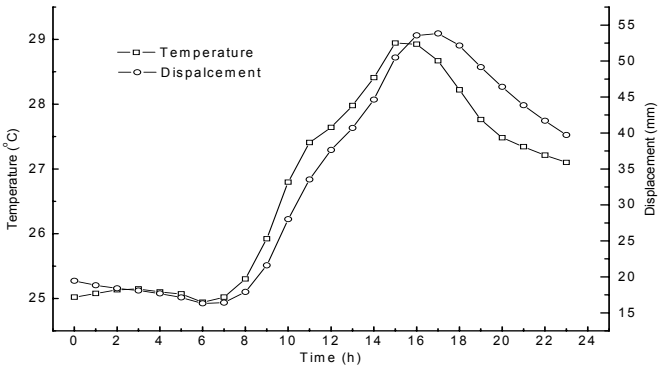


Fig. 10. Displacement and effective temperature

in Fig. 8 and the expansion joint displacement at the Tsing Yi abutment. A good correlation between them is observed and it can be concluded that the temperature fluctuations mainly account for movements of the expansion joints.

With the measurement data of the displacement and effective temperature, a regression analysis is carried out by assuming the following linear relationship

$$\Delta = \beta_1 T - \beta_0 \tag{4}$$

where Δ is the displacement of an expansion joint; T is the effective temperature at a deck cross-section. The regression coefficients β_1 and β_0 can be obtained by the least-squared method as

$$\beta_1 = \frac{S_{\Delta T}}{S_{TT}} \tag{5}$$

$$\beta_0 = \overline{\Delta} - \beta_1 \overline{T} \tag{6}$$

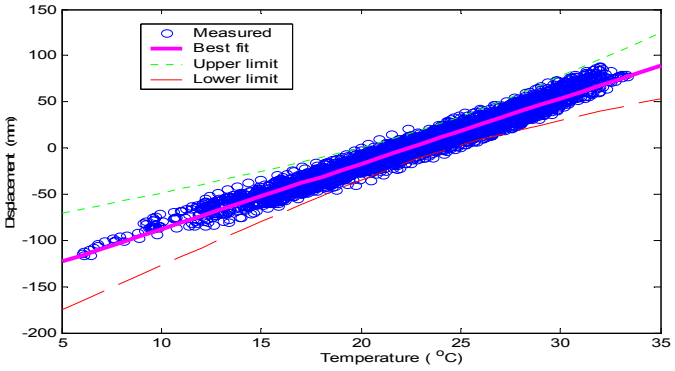
where S_{TT} is variance of the measured temperature sequence; $S_{\Delta T}$ is covariance between the measured displacement and temperature sequences; $\overline{T}$ and $\overline{\Delta}$ are means of the measured temperature and displacement sequences, respectively.

For real applications, an interval under a certain confidence level is necessary to account for the inherent uncertainties due to randomness in the measurement data, the incompleteness of the model and so on. The upper and lower bounds of an interval under a confidence of $1-\alpha$ for the predicted output Δ at a given input T are described by

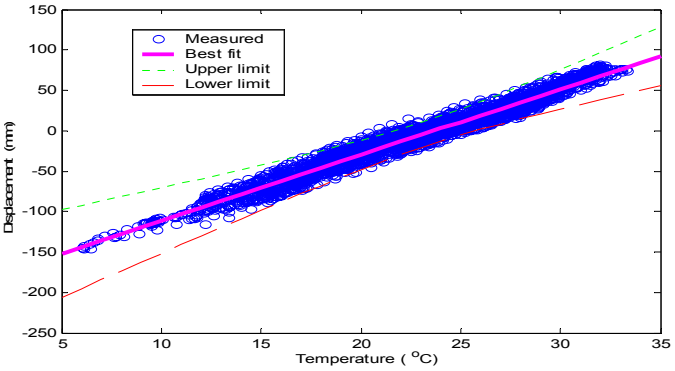
$$\Delta \pm t_{n-2,\alpha/2}\sigma^2 \sqrt{1+\frac{1}{n}+\frac{(T-\overline{T})^2}{S_{TT}}} \tag{7}$$

where σ is the variance of residuals for the regression model, and $t_{n\text{-}2,\alpha/2}$ is a critical value which can be obtained from the t-table (Kottegoda and Rosso 1997).

Fig. 11 shows the identified regression models as well as the upper and lower bounds with a confidence level of 0.95. The slopes of the linear regression functions are 8.13 mm/℃ and 7.08 mm/℃ for the expansion joints at the Ting Kau and Tsing Yi abutments, respectively. The bounds corresponding



(a) Ting Kau abutment



(b) Tsing Yi abutment

Fig. 11. Establishment of normal correlation patterns

to an appropriate confidence level should be set to avoid false alarming. It is generally necessary to adjust the bounds after trial operation with the measurement data obtained in the first few years. Once the normal correlation pattern has been determined, new measurement data can be fed into it for anomaly detection of the expansion joints. By substituting the maximum and minimum design effective temperatures into the regression models, the displacement range of expansion joints due to temperature variations can be predicted and a design validation can be conducted by comparing the predicted displacement range with the corresponding design value.

In the design of long-span bridges, it is of importance to estimate, as accurately as possible, the extreme temperatures with a certain return period. Overestimation of the extreme temperatures will lead to economical over-design, whereas underestimation may pose danger to bridge safety and serviceability. With the long-term monitoring data, a reasonable estimation of the extreme temperatures can be made by means of extreme value analysis (EVA). Given the samples of maxima and minima from measurement, EVA attempts to parametrically fit the data to one of the three limiting distributions (Castillo 1988). The Gumbel distribution is employed in the present study which is defined by

$$F(T) = \exp\left(-\exp\left(-\frac{T-\lambda}{\delta}\right)\right) \qquad -\infty < T < +\infty \tag{8}$$

where λ, δ are constants to be determined. The formulated distribution model is then used to extrapo-

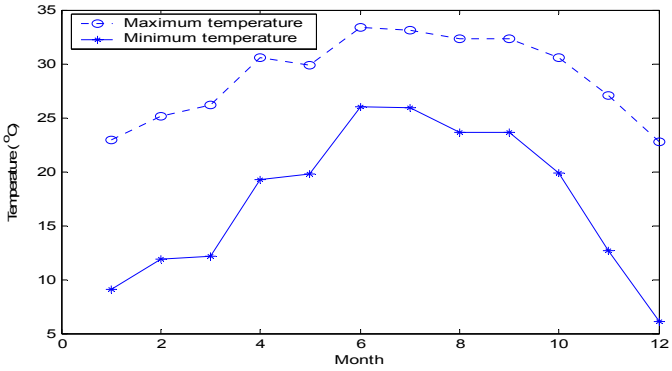


Fig. 12. Monthly maxima and minima of temperature

late extremes with a specified return period (the return period is taken equal to bridge design life in this study). Rearranging Eq. (8) leads to

$$T = \lambda - \delta \ln\left(-\ln F(T)\right) \tag{9}$$

Monthly maxima and minima of the measured effective temperatures for the Ting Kau Bridge deck cross-section are illustrated in Fig. 12. Denoting the sample sequence for monthly maxima or minima as $T_1, T_2, \cdots, T_k$ where T_i's are increasingly ordered with $T_1 \le T_2 \le \cdots \le T_k$ and k = 12 is the number of total samples, the cumulative probability of sample T_i is estimated as

$$p_i = \frac{i}{k+1} \tag{10}$$

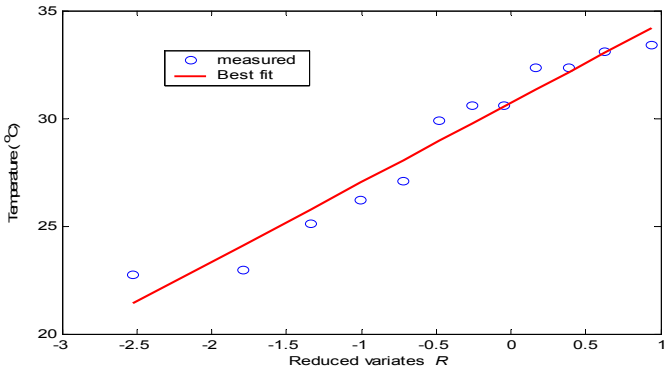
As p_i is a rational estimate of the limiting distribution function $F(T_i)$, the coefficients λ and δ can be best fitted from Eq. (9) with the use of the temperature samples T_i and the corresponding cumulative probabilities p_i (i=1, 2, ···, k). As the cumulative probability of extreme values with a return period of y years is $1-1/ky$, the extreme values for this return period can be estimated by

$$T_R = \lambda - \delta \ln\{-\ln[1-1/(ky)]\} \tag{11}$$

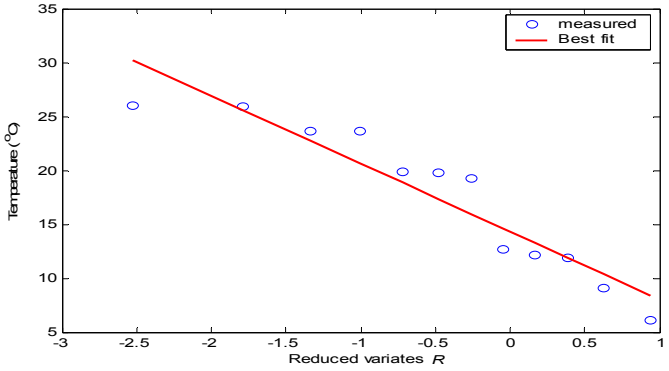
or

$$T_R \approx \lambda + \delta R \tag{12}$$

where $R = \ln(ky)$ is termed as reduced variate. Fig. 13 shows the measured and fitted relations between the extreme temperature and the reduced variate. Based on the established model, the maximum and minimum effective temperatures for a return period equal to the bridge design life can be predicted. Verification is then conducted by comparing the predicted extreme temperatures with the corresponding design values.



(a) Maximum temperature



(b) Minimum temperature

Fig. 13. Extreme temperature versus reduced variate R

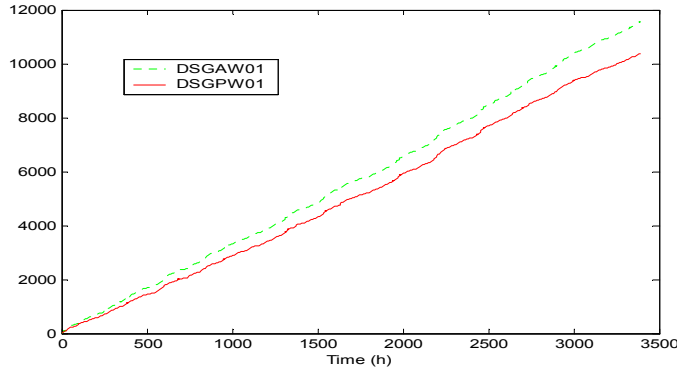


Fig. 14. Accumulative displacements of expansion joints

As the service life and interval for replacement of expansion joints mainly depend on the accumulative displacements that the expansion joints experienced, it is of significance to obtain actual daily and annual accumulative displacements. Upon the assumption of linear evolution of the accumulative displacement with time, annual and daily average accumulative displacements can be readily estimated from the continuous monitoring data. Suppose that the design value of daily accumulative displacement is D_d from which the interval for replacement of expansion joints is determined as N_d years. When the actual daily average accumulative displacement is measured to be D_m, a reasonable alteration of the interval for replacement of expansion joints is

$$N_m = \frac{D_d}{D_m} N_d \tag{13}$$

Fig. 14 shows the measured accumulative displacements of the expansion joints. It is seen that the accumulative displacements increase with time in a nearly linear proportion, with a rate of approximately 3.41 mm per hour for the Ting Kau abutment and 3.06 mm per hour for the Tsing Yi abutment, respectively. Hence the monthly or yearly accumulative displacements can be easily estimated. The daily average accumulative displacements are obtained

from the measurement data as 81.82 mm and 73.48 mm for the Ting Kau and Tsing Yi abutments, respectively, which are much less than the design value of 120 mm. Following Eq. (13), we obtain

$$N_m = \frac{120}{(81.82+73.48)/2} N_d \approx 1.5 N_d \tag{14}$$

It is therefore expected that the service life of the expansion joints is longer than the design specification and the interval for replacement can be appropriately prolonged.

5 FAILURE PROBABILITY EVALUATION USING LONG-TERM MONITORING DATA

The development of structural health monitoring methods for detection of damage occurrence, location and severity has now attained some degree of maturity. However, the application of monitoring data/results for instructing bridge inspection, maintenance and management is still in its infancy. A gap between health monitoring technology and bridge inspection and maintenance exercises exists currently which impedes bridge managers to benefit from the monitoring system. From the monitoring data the bridge managers want to get answers to the serviceability and reliability issues: (i) has the load capacity or resistance of the structure changed? (ii) what is the probability of failure of the structural members and the whole structure? Indicators of these performance issues are needed to enable the owners to allocate resources toward inspection, maintenance and rehabilitation of their structures.

A method for evaluating failure probability (or safety index) of bridge components based on long-term monitoring data and time-dependent reliability analysis is proposed. This method is targeted to provide quantitative information to bridge managers for decision making on optimizing and prioritizing bridge inspection and maintenance. According to the reliability theory, the failure probability P_f (or safety index β) of a structural component can be evaluated by considering both the member resistance (capacity) R and the load effect S as random variables

$$P_f = \iint_{r-s<0} f_R(r) f_S(s) dr ds \tag{15}$$

where $f_R(r)$ and $f_S(s)$ are probability density functions of R and S. In this method, the probability density function of the load effect S (stress, bending moment, shear force, etc.) is obtained directly from continuous measurement or derived from continuously measured strain. The probability density function $f_S(s)$ is then estimated from this measured distribution which is not required necessarily to be Gaussian. When the measured strain/stress distribution varies due to structural damage or loading condition, the probability density function $f_S(s)$ thus obtained will also be changed accordingly.

The probability density function of the resistance (capacity) R for intact (healthy) member is calculated using the mean value and standard deviation of material strength prescribed in standards or obtained by in-situ material tests. When damage-induced change in member stiffness or structural properties is detected by the health monitoring system, the probability density function $f_R(r)$ will be changed accordingly to account for the structural damage. For instance, the moment resistance will be reduced by $\alpha/(1-\alpha)$ when a fractional loss α (=1-EI_d/EI_u) in bending stiffness is detected. In this way, the effect of detectable structural damage (expressed in either deterministic or probabilistic manner) on the failure probability is accounted for.

Corrosion and fatigue are two important factors affecting structural performance of bridges. With the measured chloride data and stress history from long-term monitoring systems, the effects on the failure probability caused by corrosion and fatigue are estimated using the corresponding reliability models and interpreted in terms of 'condition factor' f_c. The overall failure probability is obtained by multiplying the 'base probability of failure' given in Eq. (15) with the 'condition factor' f_c as well as 'loading factor' f_l and 'redundancy factor' f_r. That is

$$P_f^o = P_f \cdot f_c \cdot f_l \cdot f_r \tag{16}$$

When the failure probability (or safety index) of structural components or sections under monitoring is obtained at regular intervals, it is easy to decide bridge inspection/maintenance strategy because the correspondence between the safety index value and the required maintenance action has been established (Frangopol *et al.* 2001, Lark and Flaig 2005). Table 2 shows such a relation. For unmonitored structural components, the proposed method is applied for safety index evaluation with the help of the statically calibrated structural (finite element) model, measuredly determined loading spectrum and measurement data of neighboring monitored members.

Table 2. Relation between safety state and maintenance action

Safety State	5	4	3	2	1
Safety Index	$\beta > 9.0$	$9.0 > \beta > 8.0$	$8.0 > \beta > 6.0$	$6.0 > \beta > 4.6$	$4.6 > \beta$
Attribute for Safety	Excellent	very good	good	fair	unacceptable
Maintenance Action	No action	Preventive inspection	Detailed inspection	Possible strengthening	Rehabilitation

The proposed method and the influence of structural damage on safety index are demonstrated using finite element simulation of a steel truss bridge. In this structure, the mean value of yield stress is 210 MPa, and the coefficient of variation (ratio of standard deviation to mean value) is equal to 0.1. The damage is simulated by 10%, 30%, 50%, 70% and 90% reduction of stiffness in a member. The sequences of response and 'measured' internal force (stress) of the intact (healthy) and damaged structure are obtained by finite element analysis under random traffic loads. Fig. 15 shows the distribution of resistance and load effect of the concerned member before damage and under different damage severities, from which the failure probability versus damage extent and the safety index versus damage extent are obtained as shown in Figs. 16 and 17, respectively. It is seen that the failure probability of the member successively increases with the increasing damage, while the safety index of the member successively decreases with the increasing damage. The influence of structural damage on safety index is noticeable in this example and the safety index obtained from the monitoring data (stress or other internal force) can be used to instruct maintenance action.

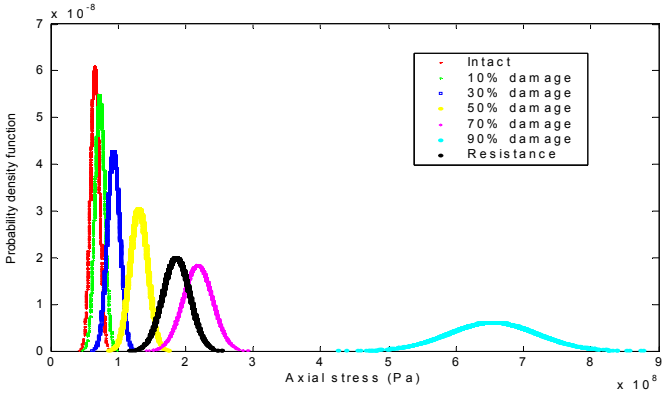


Fig. 15. Distribution of resistance and load effect

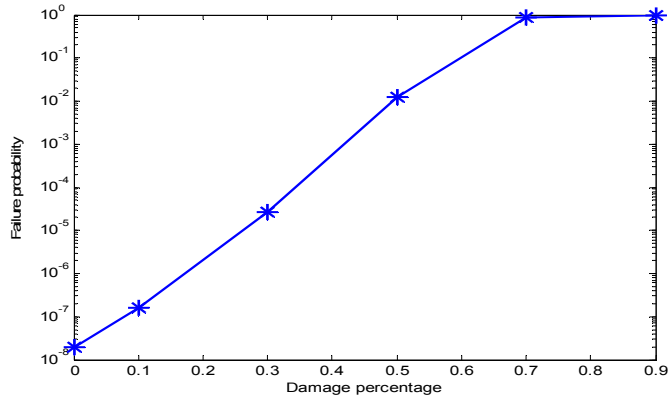


Fig. 16. Failure probability versus damage extent

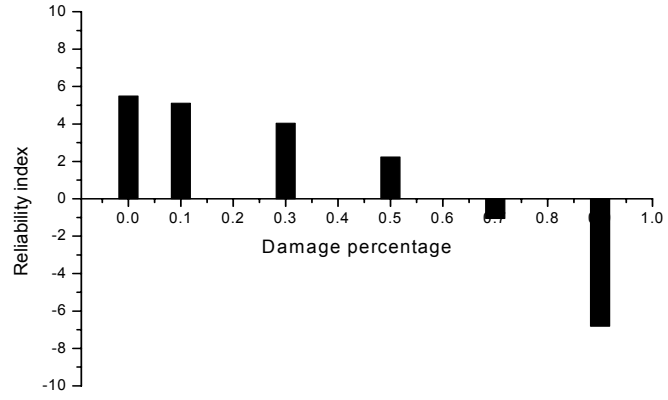


Fig. 17. Safety index versus damage extent

6 CONCLUSIONS

Recent trends and technical difficulties in implementing long-term structural health monitoring systems for large-scale bridges are first addressed in this paper. Some structural health and condition assessment methods which are not based on vibration or modal measurement are then explored by use of real measurement data from the long-term monitoring systems installed on the cable-supported bridges in Hong Kong. These methods involve: (i) fatigue life assessment using monitoring data, (ii) condition assessment of expansion joints using monitoring data, and (iii) bridge component failure probability assessment using monitoring data and its linkage with bridge maintenance.

On the current status in bridge health monitoring practice, it has been pointed out that: (i) the recently devised long-term monitoring systems emphasize multi-purpose monitoring of the bridge integrity, durability and reliability, (ii) utmost care must be taken of the protection of on-structure data acquisition units from dust, temperature, humidity, and electromagnetic interference, (iii) a recently emerging task is on the protection, maintenance and upgrade of the first-generation bridge monitoring systems that have operated over ten years. On the fatigue life assessment using monitoring data, it is shown that the measured stress spectra from different days are similar and a standard daily stress spectrum can be obtained for fatigue life prediction of welded details. The stress distribution factor may have a considerable effect on the predicted fatigue life.

On the assessment of expansion joints, a procedure for design verification and condition assessment of expansion joints based on long-term monitoring of the expansion joint displacement and bridge temperature has been presented. It pursues the establishment and check of the temperature-displacement pattern and the prediction and verification of the maximum displacement range, extreme temperature and accumulative movement. In order to link structural health monitoring with bridge inspection and maintenance, a method for evaluating failure probability (or safety factor) of bridge components based on long-term monitoring data and time-dependent reliability analysis has been proposed. This method possesses several merits: (i) the load effect and its distribution are determined from field measure-

ments, (ii) damage-induced change in member stiffness or structural dimensions can be reflected in the resistance variable, and (iii) the effects caused by material deterioration or environmental loading are accounted for in the evaluation of safety index.

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