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Hot spot stress approach for Tsing Ma Bridge fatigue evaluation under traffic using finite element method

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Abstract. The hot spot stress approach is usually adopted in the fatigue design and analysis of tubular welded joints. To apply the hot spot stress approach for fatigue evaluation of long span suspension bridges, the FEM is used to determine the hot spot stress of critical fatigue location. Using the local finite element models of the Tsing Ma Bridge, typical joints are developed and the stress concentration factors are determined. As a case for study, the calculated stress concentration factor is combined with the nominal representative stress block cycle to obtain the representative hot spot stress range cycle block under traffic loading from online health monitoring system. A comparison is made between the nominal stress approach and the hot spot stress approach for fatigue life evaluation of the Tsing Ma Bridge. The comparison result shows that the nominal stress approach cannot consider the most critical stress of the fatigue damage location and the hot spot stress approach is more appropriate for fatigue evaluation.

Key words: long suspension bridge; fatigue life evaluation; hot spot stress; finite element method; traffic loading.

1. Introduction

Fatigue design rules commonly used in practice are mainly based on the data generated from tests on either beams or small-scale specimens incorporating the weld detail of interest, subject to unidirectional loading. Based on the above, the fatigue test can be expressed in terms of the nominal stress and the S-N curve can be obtained in terms of the nominal stress approach. This approach is then adopted for fatigue design and fatigue damage calculation. BS5400 Part 10 (1982), which gives guidance for steel bridge fatigue design in practice, adopts the nominal stress approach for fatigue evaluation. In BS5400 Part 10 (1982) the classification method is used to classify the welded details according to the direction of the fluctuating stress relative to the detail, the location of the possible crack initiation of the detail and the geometrical arrangement of the detail. The corresponding

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coefficient of the S-N relationship can then be determined from the class type and the damage accumulation is obtained using the linear Miner's law. The fatigue behavior of welded offshore structure joints usually fails in welded joints, not the member. The nominal stress approach for fatigue design is not suitable for steel structures with complicated geometry because the stress state in such structures is complex and the definition of nominal stress is difficult. An alternative method for the complicated welded steel joints fatigue design is the hot spot stress approach. The hot spot stress approach, which considers the structure stress concentration at the welded vicinity, has been widely used in hollow steel tubular structure fatigue design and analysis in practice. However, little effort has been made on the hot spot stress approach in the application of long suspension steel bridge fatigue evaluation.

Highway bridges are subjected to a large number of repetitive loadings of different magnitudes caused mainly by the passage of vehicles. For long span bridges, vehicles produce stress cycles in members and fatigue of the member is accumulated with the daily traffic loadings. Highway bridges undergo different and complicated traffic loadings everyday and, therefore, they are subjected to variable amplitude stress cycles that give rise to fatigue damage accumulation. Fisher (1984) has made extensive investigations of the fatigue and fracture behavior of steel bridges and bridge failure accidents. He has discovered that fatigue is one of the most important bridge failure modes and that fatigue of even the secondary member can sometimes cause a whole steel bridge to collapse. As welded steel structural elements are widely used in bridges, the fatigue accumulation evaluation of steel bridges should be paid much attention.

The influence factor of joint geometry should be considered in the calculation of stress for fatigue analysis as pointed out by BS7608 (1993). The corresponding applied stress for fatigue design and evaluation should be hot spot stress, which also takes joint geometry into account while the nominal stress range cannot consider the joint complicated geometry. BS7608 (1993) suggests using hot spot stress analysis for such tabular nodal joints and the stress range for such joints should be in the hot spot stress range. Hot spot stress analysis, therefore, is used in fatigue resistant design and the durability estimation of welded offshore tubular joints. During the last 30 years a large amount of research has been carried out on the fatigue design of hollow steel tubular structural welded joints adopting hot spot stress analysis for fatigue design and durability analysis (IIW, CISM courses and lectures No.394 1998, Maddox 2000, Puthli and Herion 2001, Savaidis and Vormwald 2000).

Savaidis and Vormwald (2000) used ABAQUS to undertake hot spot stress analysis of various welded joints of city bus floor structures. They also carried out an experiment on welded joints under cyclic constant amplitude loading to examine the fatigue failure behavior. The numerical results give the correct fatigue failure location - the hot spot stress area, as the experiment. The result also coincides with the experimental result, which shows that the hot spot stress approach is suitable for welded steel joint fatigue evaluation.

The hot spot stress method has been included in the CIDECT (2001) Part 8 design guide for circular and rectangular hollow section welded joints under fatigue loading. Recently, Mashiri *et al.* (2002) applied the hot spot stress method for fatigue design for thin walled welded joints, which is not included in the fatigue design rule of the current Canadian Standard.

Macdonald and Haagensen (1999) used hot spot stress analysis for fatigue design of welded rectangular hollow section joints and identified the hot spot stress analysis for fatigue design as one of the most promising method in terms of providing a coherent and comprehensive approach to fatigue design. The recommendation on the use of the hot spot stress analysis for fatigue design is discussed in their report.

Although much work has been carried out on offshore steel tubular welded joint fatigue analysis, little research has been done relating to fatigue damage evaluation of large-scale welded steel structures, especially for long span steel suspension bridges. This paper aims at the application of the hot spot stress approach for large welded steel structure fatigue evaluation and provides a basis for the remaining life estimation of large suspension steel bridge. As a case study, a global FEM model of the Tsing Ma Bridge is firstly developed to obtain the global structure response under traffic loading, which determines the critical fatigue location of the whole bridge. To investigate the critical fatigue locations determined from the global dynamic response, the local finite element models of these critical fatigue regions are developed to determine the corresponding hot spot stress to determine SCF (Stress Concentration Factors). With the general-purpose commercial software ABAQUS, the local finite element models of critical fatigue locations are set up. Different load case combinations and the appropriate boundary condition is applied to these models, given by the global FEM analysis of the Tsing Ma Bridge response under the traffic loadings. The hot spot stress in the vicinity of the welded joints can be determined for the fatigue life evaluation of the Tsing Ma Bridge. With the representative nominal stress block cycle obtained from the online data health monitoring system, the corresponding hot spot stress block cycle could be determined multiplied by the SCF and the nominal stress block cycle. A comparison is made to evaluate the fatigue accumulation between the nominal stress approach and the hot spot stress approach.

2. Dynamic response of the Tsing Ma Bridge under traffic loading

The Tsing Ma Bridge, which is 2.2 kilometers in total length with a main span of 1377 meters, as shown in Fig. 1, is the longest suspension bridge in the world carrying both road and rail traffic. The bridge carries two three-lane carriageways on the upper deck, and twin railway tracks and two longitudinal as well as two single emergency road lanes on the lower deck. The Tsing Ma Bridge is the main connection between Tsing Yi Island and Lantau Island, and is a major part of the

Fig. 1 Landscape of the Tsing Ma Bridge

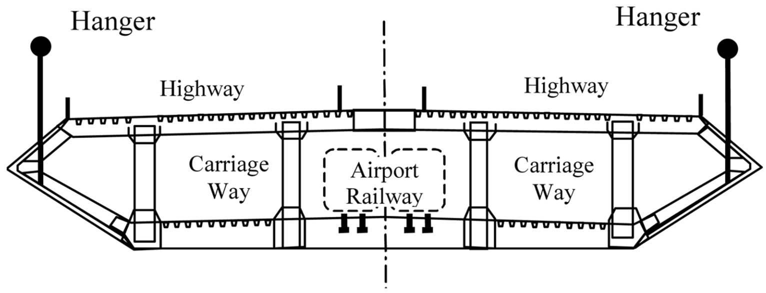


Fig. 2 Typical deck section of the Tsing Ma Bridge

transportation network for Hong Kong Airport. Due to its importance in the Hong Kong transportation network, a structural health monitoring system has been installed to monitor the integrity, durability, and reliability of the bridge (FNP 1998). With the information provided by the health monitoring system, some important work relating to fatigue damage analysis has been carried out in the Department of Civil and Structural Engineering of the Hong Kong Polytechnic University. The locations of the monitoring sensors installed in the bridge, however, were selected for general purposes and they are not necessarily located in the critical fatigue damage region. British Standards have recommended the finite element method as a rigorous method for structural fatigue stress analysis. The standard also gives guidance for the installation locations at the most critical fatigue damage location and the dynamic response under traffic loading. The global response of the Tsing Ma Bridge under traffic loading should be performed by the finite element method. As the first step of the fatigue damage evaluation, the global finite element model of the Tsing Ma Bridge is developed to simulate the response of the suspension bridge to provide necessary information for finding out the possible fatigue damage locations along the main span under traffic loadings (Chan *et al.* 2003).

As shown in Fig. 2, the Tsing Ma Bridge is a double deck suspension bridge, which contains about twenty thousand structural members including longitudinal trusses, cross frames, deck plates, tower beams, main cables, hangers, etc. The global finite element model of the Tsing Ma Bridge developed by using the general-purpose finite element package program ABAQUS (Wang *et al.* 2000) is mainly for modal analysis and such a model cannot be used as a dynamic response for stress and displacement output of the desired elements. Chan *et al.* (2003) have developed a Tsing Ma Bridge global FEM model that is suited for dynamic response output of the desired element. The 4-node shell element is adopted to model the orthotropic deck and the 3 dimensional 2-node beam element is used to model the longitudinal truss, the main cross frame, intermediate cross frame, the main cable and the hangers. More than 7,579 nodes and 17,677 elements are included in the Tsing Ma Bridge FE model. The verification of the global model shows that the global finite element model of the Tsing Ma Bridge could give reasonable stress results for both static and dynamic analysis (Chan *et al.* 2003).

With the developed FEM model, the dynamic characteristic of the Tsing Ma Bridge under traffic loading has been examined with the FEM model and the calculated dynamic characteristic agrees with the measured value well. The global analysis under railway traffic and highway traffic (Chan *et al.* 2003) shows that the most critical fatigue damage location is the outmost upper chord of typical cross frame and the corresponding longitudinal truss. Due to limitations in the computation cost and computer memory, the FEM model does not incorporate the typical cross frame joint in detail. However the stress concentration existing in the complex steel joint geometry, which could not be identified from the global Tsing Ma FE model and the local FEM analysis, is necessary to investigate the hot spot stress.

It is important to determine the critical fatigue damage locations in practical bridges. For long span bridges, individual vehicles produce very small stress cycles in members, but larger cycles may be produced when the entire bridge is subjected to lane loading during peak traffic hours. In order to consider both highway and railway traffic influence on the bridge dynamic response, the bridge dynamic responses under both kinds of traffic loading are studied separately with the finite element model developed. The critical fatigue damage location under both traffic loadings can be identified as the outermost zone of the highway lane, and has been analyzed in detail in Chan *et al.* (2003).

In order to simulate the moving load on the finite element model, the equivalent nodal forces and nodal moments, which are a function of time of all nodes along the Tsing Ma Bridge train rail, are applied on the nodes. The virtual work principle is used to transfer the moving concentrated loading on the nodes of element (Bathe 1982, Wu *et al.* 2000). Chan *et al.* (2003) applied the moving truck and moving train equivalent loads separately on finite element model to simulate the dynamic response. The single lane, double lane, triple lane truck loading and the train loading is analyzed separately. Stress distribution in typical deck has the same distribution trend, where the largest value of stress is near the outermost part of the upper chord and the bottom cross frame between the rail tracks. The dynamic response under train loading is also analyzed in Chan *et al.* (2003). The stress distribution along the bridge decks in the main span has the same trend, that is the maximum stress distribution occurs near the outermost lane of the upper chord and the bottom cross frame between the rail tracks. The critical locations due to fatigue damage under rail loading are identified as where the maximum stress value occurs, that is the outermost lane of the cross frame upper chord and the corresponding longitudinal truss.

FNP (1998) have made a review of the danger degree and the critical grade of different bridge members and pointed out that the most critical regions are the outermost lane which undergoes traffic loading. This agrees with the results of the dynamic response of the whole bridge under traffic loading (Chan *et al*. 2003).

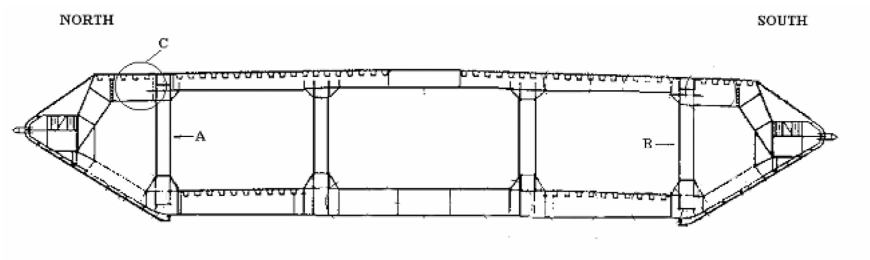
To apply the hot spot stress approach in the fatigue accumulation estimation of the Tsing Ma Bridge, it is necessary to investigate the critical fatigue region for the fatigue damage evaluation of the Tsing Ma Bridge under traffic loading. The local stress concentration due to the geometry changes, especially in the welded zones should also be considered. These factors are discussed in the following section.

3. Critical fatigue damage region investigated

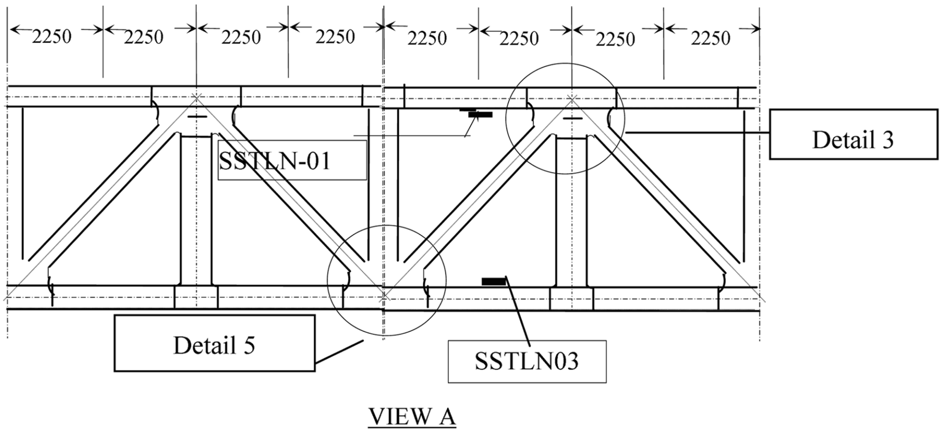
3.1 Necessity of hot spot stress analysis

Commonly used fatigue design rules are based mainly on the data generated from tests on small-scale specimens incorporating the welding detail of interest subjected to unidirectional loading. Fatigue test results can, therefore, be expressed in terms of the nominal applied stress in the region of the test detail and the same stress is usually specified for use with the design S-N curves. An alternative approach, which is widely applied and is already established for tubular connections, is based on the hot spot stress, where the stresses are near the fatigue failure of the weld toe.

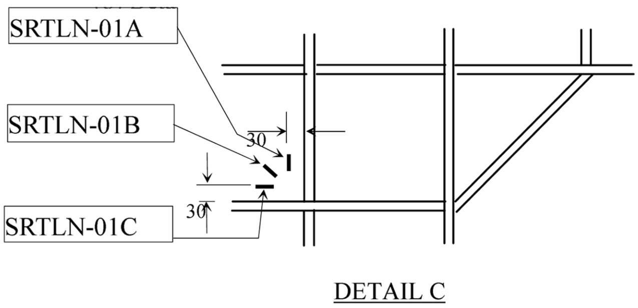
The hot spot stress method relates the fatigue life of complex steel structure joints with the hot spot stress, rather than the nominal stress. It considers the uneven stress distribution in the complex steel structure joints and incorporates geometry influences and excludes the effects related to the weld configuration and weld toe. The hot spot stress is the maximum stress occurring in the complex steel structures, where cracks are most likely initiated.



(a) The cross frame of TMB



(b) Details of the location of "SSTLN-1" and "SSTLN03"



(c) Details of the location of "SRTLN-01"

Fig. 3 Location of strain gauges in the cross frame of TMB (Li *et al.* 2001)

As mentioned earlier, strain gauges were installed on some of the steel members of the Tsing Ma Bridge as part of the structural health monitoring. Fig. 3 shows typical strain gauges or rosette locations in a cross frame. It can be seen that the strain gauges are located away from the weld toe, which gives only nominal strain. In order to know the hot spot stress near the weld toe, it is necessary to convert the nominal stress obtained from the online health monitoring system into the corresponding hot spot stress near the weld toe for fatigue life evaluation. Here the stress concentration factor (SCF) is defined as the ratio of the hot spot stress value with that of the nominal stress. With the SCF value obtained, the hot spot stress can be obtained by multiplication of the nominal stress with the SCF value. The SCF value of a welded joint is commonly obtained by experimental numerical methods. The welded joint of the Tsing Ma Bridge is very big and complex; it is almost impossible to obtain the SCF value in a laboratory. In the current research, the numerical finite element method is chosen as an alternative method for obtaining the SCF value, which is widely used in the determination of the SCF value in tubular welded joints (CIDECT 2001). In order to apply the hot spot stress approach to the fatigue evaluation of the Tsing Ma Bridge, it is necessary to carry out local stress analysis to obtain the hot spot stress of the most critical fatigue damage locations.

3.2 Finite element model of critical welded region

With the critical fatigue damage region determined from the global response of the Tsing Ma Bridge, the hot spot stress analysis is performed in the vicinity of the welded joints corresponding to the critical fatigue region. With the response determined from the global FEM analysis, the local FEM model is developed with the general-purpose commercial software ABAQUS. The stress concentration factor of the welded joints is used to convert the nominal stress into the hot spot stress. The welded joints hot spot stress is then obtained to provide the basis for the critical fatigue damage evaluation.

The effect of the notch stress concentration is negligible at a distance from the weld toe exceeding about 0.4 t, where t is the thickness of the welded plate (IIW). Thus the element dimensions of

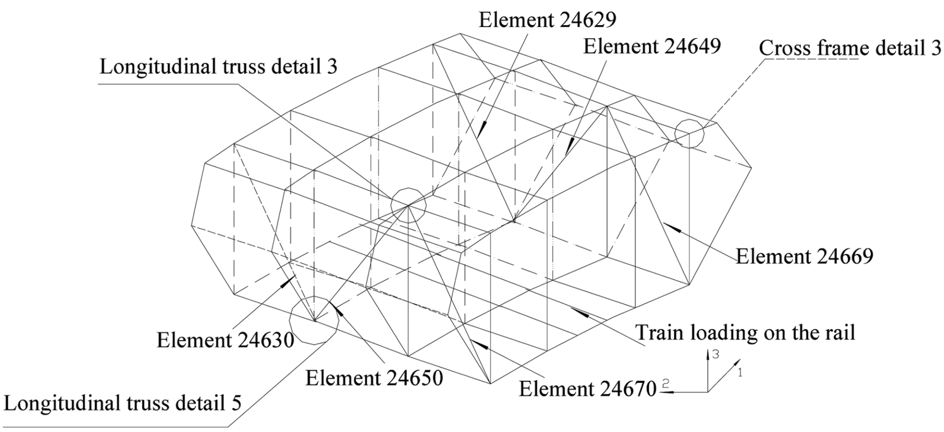
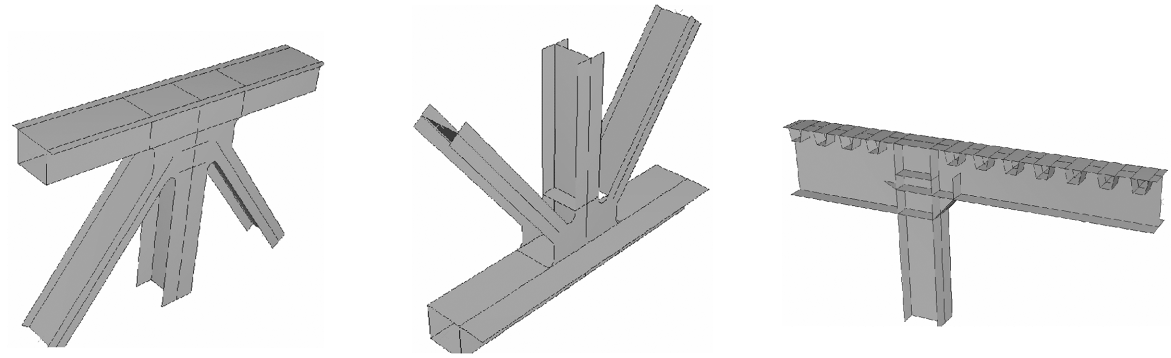


Fig. 4 The typical deck element



(a) Longitudinal truss detail 3 (b) Longitudinal truss detail 5 (c) Cross frame detail 3

Fig. 5 Joint geometry of typical deck investigated

interest should be defined as such that the center of the first element from the highest hot spot stress location should be no more than 0.4 *t*. The longitudinal truss shown in Fig. 3(b) has two types of local details, that is detail 3 and detail 5, as shown in Figs. 5(a) and 5(b) separately. The main cross frame of the deck shown in Fig. 3(a) has typical local detail 3 in the outermost of the highway lane, as shown in Fig. 5(c). Three different types of typical joints in the Tsing Ma Bridge are considered located in the outermost lane, which is recognized as the most critical fatigue damage region under traffic loading (Chan *et al*. 2003). These three kinds of details are representative of the details along the main span and are shown in Fig. 5 namely: (a) longitudinal truss detail 3, (b) longitudinal truss detail 5 and (c) main cross frame detail 3.

For analysis undertaken using the finite element method, the hot spot stress ranges can be obtained directly from the analysis to determine the stress concentration factor. The S4R element, 4 node reduced integration shell element in the ABAQUS element library, is used to model the local finite element model. The element size near the weld toe is adequate to fulfill the requirement of capturing hot spot stress as mentioned by IIW.

3.3 Loading applied on the model

For long suspension bridges, the longitudinal truss deformation mainly undergoes in-plane deformation and the out-of-plane deformation is negligible compared with that of the in-plane deformation, which is given by the global finite element analysis result under dead load. Fig. 6 shows a typical longitudinal truss element number designation of four deck segments between two neighbourhood hangers. The magnitude of the axial force and the in-plane moment the diagonal undergoes under dead loading is almost the same among the digonals between the neighbourhood hangers. The equivalent dynamic loading applied on the local model is determined by the calculated results of the internal forces obtained from the global analysis of TMB under train loading.

The analysis on dynamic force response under train loading shows that the diagonal force response behaves in in-plane deformation and the out-of-plane deformation is negligible when

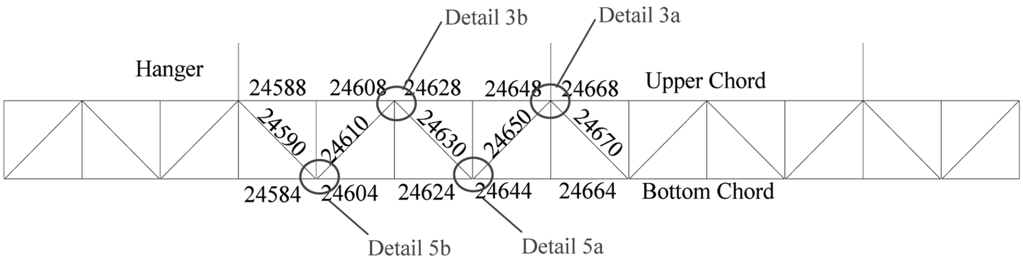
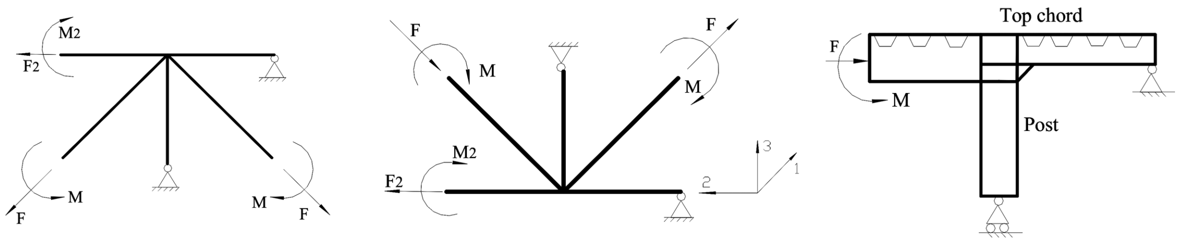


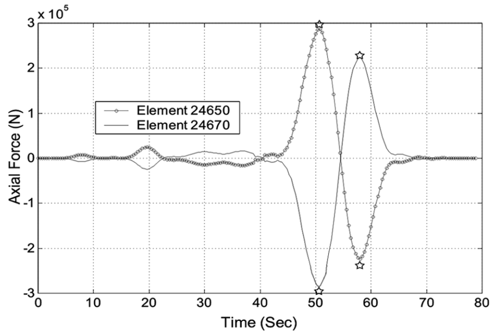
Fig. 6 Longitudinal truss element number of a typical longitudinal truss



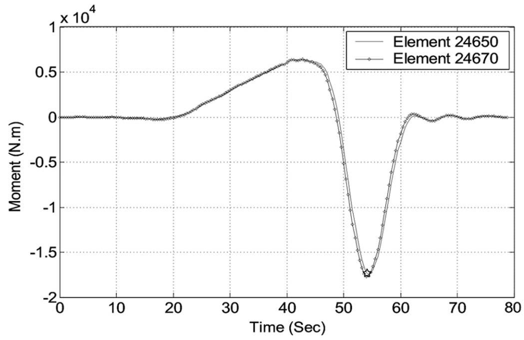
(a) Longitudinal truss detail 3 (b) Longitudinal truss detail 5 (c) Main cross frame detail 3

Fig. 7 The loading and the boundary applied on the model

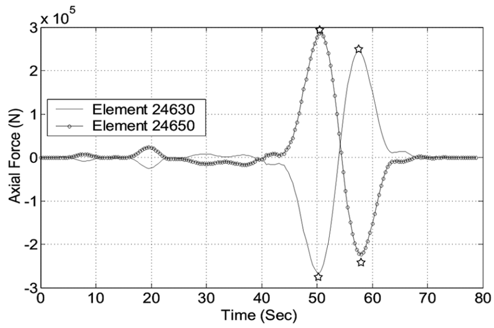
compared with that of the in-plane deformation. Fig. 7 shows the equivalent dynamic forces to be applied on the local FEM model and the corresponding boundary conditions. The axial force of the diagonal is much greater than that in the upper or bottom chord, and the axial displacement of the upper or bottom chord and the post is negligible. The displacement boundary of upper chord or bottom chord and the post is therefore considered as fixed, and the force is applied to the diagonal and one side of the upper or bottom chord. As mentioned above, the magnitude of the equivalent dynamic forces showed in Fig. 7 could be obtained from the calculated results of the internal forces in the corresponding structural elements of longitudinal truss under train loading by the global analysis, and the results are shown in Fig. 8, in which t starts from the time that the train enters the Tsing Ma Bridge and ends when it leaves the bridge. From Fig. 8, it can be seen that the the forces in the elements concerned of the longitudinal truss reach their maximum values, when the train is approaching near the corresponding digonal. The amplitude of the force in diagonals caused by the train traffic almost has the same value with that of the neighbourhood diagonals and the tendency is almost the same. In addition, the axial force in the neighbourhood diagonal has different directions i.e., one positve and the other negative, while the in-plane moment of the neighbourhood diagonal has the same direction. Several cases of load combination, as shown in Table 1 are chosen and applied to the local model of the critical welded regions. In Table 1, the considered details 3a, 3b, 5a and 5b are shown in Fig. 6, and each of the load combination, except No.1 and 5, is the superposition of internal forces generated by the dead load and the maximum absolute value of the equivalent dynamic forces at the moments with the asterisk marks shown in Fig. 8.



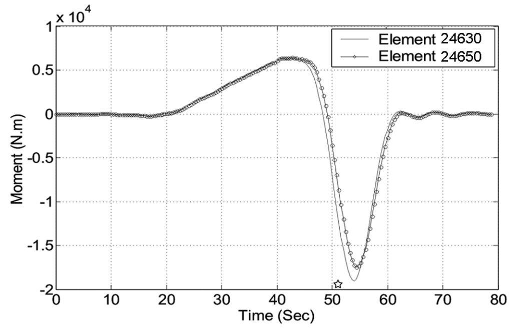
(a) Longitudinal truss detail 3 axial force



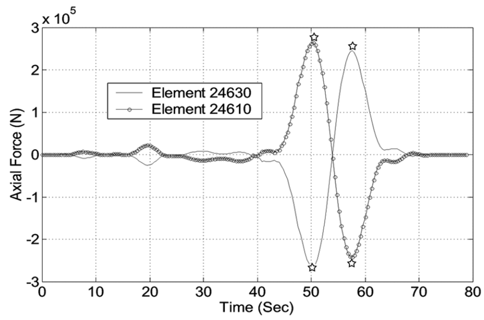
(b) Longitudinal truss detail 3 moment



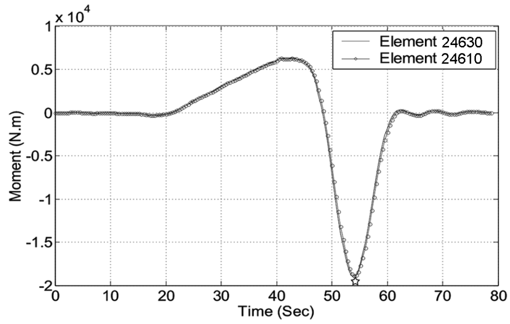
(c) Longitudinal truss detail 5 axial force



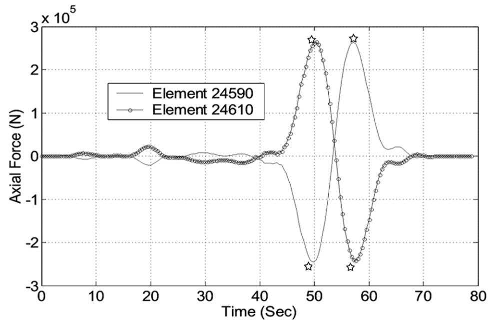
(d) Longitudinal truss detail 5 moment



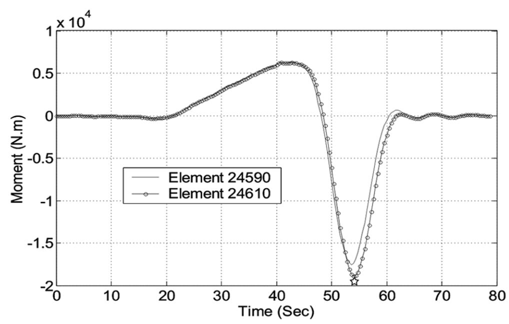
(e) Longitudinal truss detail 3 axial force



(f) Longitudinal truss detail 3 moment



(g) Longitudinal truss detail 5 axial force



(h) Longitudinal truss detail 5 moment

Fig. 8 Internal forces of longitudinal truss under train loading

Table 1 The load case combinations

1) Longitudinal truss detail 3 load case combinations

Load case combinations	Detail/No. of involved element	Load combinations
1	Detail 3a / 24670; 24650; 24648	Internal Forces due to Dead Load (IFDL)
2	Detail 3a / 24670; 24650; 24648	IFDL+IFTL* at 51.2 sec
3	Detail 3a / 24670; 24650; 24648	IFDL+IFTL* at 58.4 sec
4	Detail 3a / 24670; 24650; 24648	IFDL+IFTL* at 54.8 sec
5	Detail 3b / 24630; 24610; 24608	IFDL
6	Detail 3b / 24630; 24610; 24608	IFDL+IFTL* at 51.2 sec
7	Detail 3b / 24630; 24610; 24608	IFDL+IFTL* at 58.4 sec
8	Detail 3b / 24630; 24610; 24608	IFDL+IFTL* at 54.8 sec

2) Longitudinal truss detail 5 load case combinations

No. of load case combinations	Detail/No. of involved element	Load combinations
1	Detail 5a/24650; 24630; 24644	Internal Forces due to Dead Load (IFDL)
2	Detail 5a/24650; 24630; 24644	IFDL + IFTL* at 51.2 sec
3	Detail 5a/24650; 24630; 24644	IFDL + IFTL* at 58.4 sec
4	Detail 5a/24650; 24630; 24644	IFDL + IFTL* at 54.8 sec
5	Detail 5b/24610; 24590; 24604	IFDL
6	Detail 5b/24610; 24590; 24604	IFDL + IFTL* at 51.2 sec
7	Detail 5b/24610; 24590; 24604	IFDL + IFTL* at 58.4 sec
8	Detail 5b/24610; 24590; 24604	IFDL + IFTL* at 54.8 sec

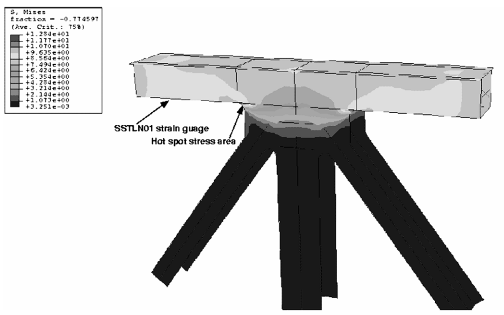
*Note: IFTL at 51.2, 58.4 and 54.8 sec means Internal Forces due to Train Load (IFTL) in the involved components at t = 51.2, 58.4 and 54.8 sec respectively, marked with the asterisk shown in Figs. 8(a~h).

4. Stress concentration factors of typical joints and fatigue life evaluation of the Tsing Ma Bridge

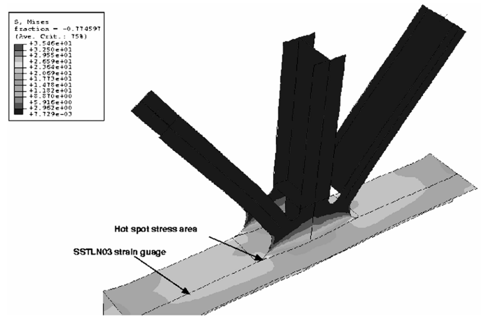
4.1 SCF of Tsing Ma Bridge typical joints

With the load combination case applied on the developed local finite element models, the stress distribution under different load combinations can be obtained. In the case of the longitudinal truss detail 3 and detail 5 shown in Fig. 5, the maximum stress under different load case combinations is found to be acting on the welding toe between the intersection of the diagonal and the gusset plate covering the post and the upper chord, or the bottom chord. The hot spot stresses are then defined as the maximum stress occurring for longitudinal truss detail 3 and 5. For main cross frame detail 3, the hot spot stresses are taken as the stress acting on the location of the weld toe between a post of the longitudinal truss and the beam of the cross frame.

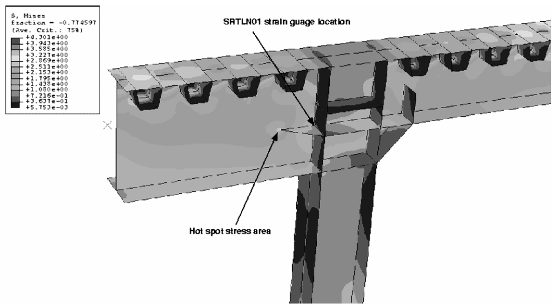
The computation results show that hot spot stress for longitudinal truss occurs at the intersection of the diagonal with the upper chord or bottom chord. The typical stress distributions for joints investigated are shown in Fig. 9. From the stress distribution of the three joint details shown in Fig. 9, it can be confirmed that stress concentration exists. The hot spot stress is taken as the nodal stress at the weld toe and the nodal stress should not be smoothed among the neighbourhood element, which is suggested in the hot spot stress analysis of welded bus frame joints (Savaidis and Vormwald 2000). The nodal stress is taken as the principle stress. The stress concentration factors are defined as the ratio of the hot spot stress to the nominal stress obtained from the corresponding location of the strain gauge of the health monitoring system. The calculated results under different load case combinations show that the maximum stress arises at the welded region between the box chord and the joint for longitudinal truss detail 3 and detail 5. The calculated SCF values with different load case combinations are shown in Table 2. In order to consider the unfavorable tendency of the stress concentration, it is suggested to use the maximum SCF among the different load case combinations to obtain the hot spot stress. With the calculated SCF value, it is possible to convert the nominal



(a) Longitudinal truss detail 3



(b) Longitudinal truss detail 5



(c) Main cross frame

Fig. 9 Stress distribution for various joint details

Table 2 Stress concentration factor of the Tsing Ma Bridge typical joints

Joint type	SCF value under load combination case								Max SCF value
	1	2	3	4	5	6	7	8	
Longitudinal truss detail 3	1.51	1.47	1.63	1.73	1.56	1.53	1.70	1.71	1.73
Longitudinal truss detail 5	1.81	1.9	1.8	1.95	1.82	1.80	1.75	1.84	1.95
Main cross frame	1.41		1.41		1.41		1.44		1.44

stress obtained from the structural health monitoring system into the hot spot stress. Extensive FEM modeling of longitudinal truss detail 3, longitudinal truss detail 5 and main cross frame detail 3, using denser mesh, shows that the stress distribution under load case combinations has the same trend and almost has the same result. In addition, the hot spot stress is found to be in the same location as the analysis result in the current calculation result. The upper chord of the longitudinal truss mainly undergoes compressive force under different load case combinations, the magnitude of which is much greater than that of the diagonal and the post of the longitudinal truss. The maximum hot spot stress occurring at the welded intersection of the upper chord is reasonable. The typical stress distribution away from the weld toe and the strain gauge location away from the weld toe are as shown in Figs. 10, 11 and 12. From the figures, the ratio of the hot spot stress to the nominal stress can easily be identified, from which the SCF can be determined. From the figures, it can be seen that the curves follow the same trend and that the stress increases from the location of the strain gauge to the hot spot stress area. When approaching the weld toe, the stress increases slowly for longitudinal truss details 3 and 5. This may be due to the fact of the gusset plate welded

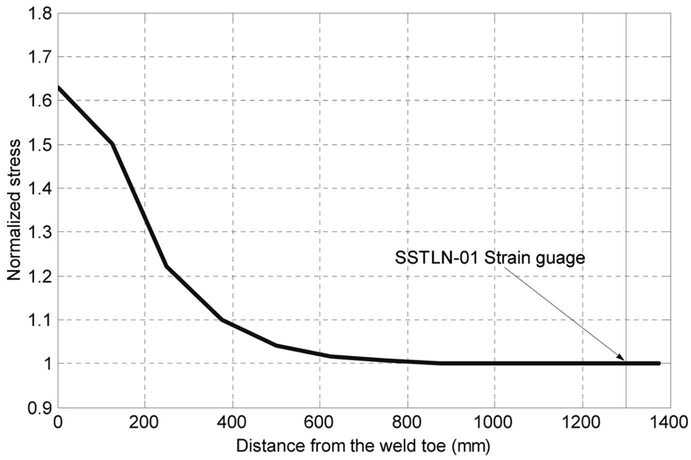


Fig. 10 Stress distribution along the longitudinal truss detail 3

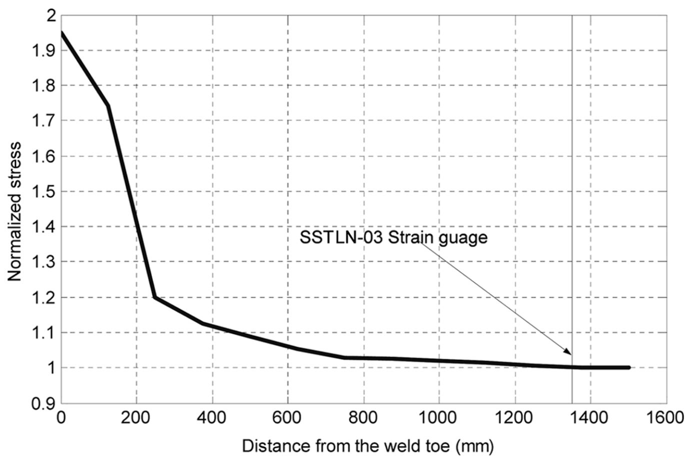


Fig. 11 Stress distribution along the longitudinal truss detail 5

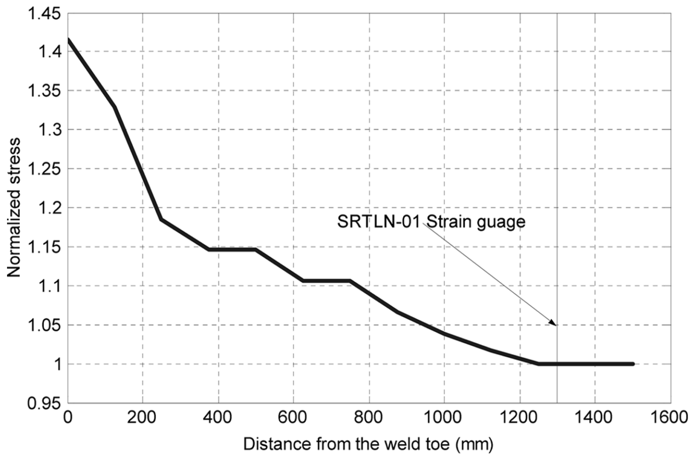


Fig. 12 Stress distribution along the cross frame investigated

to the upper chord, which transfers the stress to the diagonal lessening the stress in upper chord for longitudinal truss detail 3 (bottom chord for longitudinal truss detail 5). For the main cross frame detail 3, it does not have other components connected to the outmost beam to undergo the stress.

4.2 Fatigue life evaluation of Tsing Ma Bridge using hot spot stress approach

Bridge fatigue is a high-cycle fatigue problem where stress fluctuations are low so that the deformation in the structure is elastic except at the notch and welds where stress concentration exists. As a result, the material in the vicinity of welds is locally yielded and plastic deformation exists. For accurate evaluation of fatigue damage accumulation, the fatigue crack initiation and growth in the vicinity of welds should be properly modeled. Fatigue damage in the region of the fatigue crack growth initiation and growth of cracks in micro-scale can be well modeled by the CDM (Continuum Damage Mechanics) concept. To better evaluate and estimate the fatigue damage accumulation under traffic loading of the Tsing Ma Bridge, the CDM based fatigue damage model is adopted to determine fatigue damage accumulation.

Miner's rule for fatigue damage evaluation is adopted in BS5400 Part 10 (1982), and is a linear damage accumulation with the service year increasing. The continuum damage mechanics is concerned with the deteriorating mechanical behavior of a continuum at the macro scale and the fatigue damage evaluation based on CDM provides a more reasonable description of the damage accumulation that found in Chan *et al.* (2001). In order to make a comparison between the fatigue evaluations using the linear Miner's law, the CDM based fatigue damage accumulation rule is adopted to evaluate the damage accumulation.

According to thermodynamics and the potential of dissipation, the damage evolution of high cycle fatigue damage can be written as Chan *et al*. (2001):

$$\dot{D} = \begin{cases} \dfrac{R_v \sigma_{eq}^2 |\sigma_{eq} - \bar{\sigma}_{eq}|^{\beta}}{B(1-D)^{\alpha}} \langle \dot{\sigma}_{eq} \rangle & \text{for} \quad \sigma^* \geq \sigma_f \\ 0 & \text{for} \quad \sigma^* < \sigma_f \end{cases}$$

Where B and β denote the material constants. σ_{eq} is the effective Von Mises equivalent stress, defined by: $\sigma_{eq} = \left[\frac{3}{2}\sigma_{ij}^{D}\sigma_{ij}^{D}\right]^{1/2}$. R_v is a triaxiality function, which can be reduced to the simple form under pure bilateral conditions, $R_v = \left[\frac{\sigma^*}{\sigma_{eq}}\right]^2 = \frac{2}{3}(1+\mu) + 3(1-2\mu)\left(\frac{\sigma_H}{\sigma_{eq}}\right)^2$. The symbol < > denotes the McCauley brackets where $< x > = x$ for $x > 0$ and $< x > = 0$ for $x < 0$.

The principal stress is used to obtain the representative stress block cycle from the structural health monitoring system. To apply the CDM based fatigue damage model to the structural health monitoring online data system, it is necessary to simplify the general expression of CDM based fatigue damage model into a one-dimensional situation.

The fatigue damage evolution can be reduced to the form under the one-dimensional condition.

$$\dot{D} = \begin{cases} \dfrac{\sigma^2|\sigma - \bar{\sigma}|^{\beta}}{B(1-D)^{\alpha}}\langle\dot{\sigma}\rangle & \text{for} \quad \sigma^* \geq \sigma_f \\ 0 & \text{for} \quad \sigma^* < \sigma_f \end{cases}$$

Assuming $\bar{\sigma} = \sigma_m = 0$ and neglecting the variation of $(1 - D)^{\alpha}$ over one block stress cycle, the damage increment can be obtained as:

$$\frac{\partial D}{\partial N} = \sum_{i=1}^{m_{rb}} \frac{\sigma_{ari}^{\beta+3}}{B(1-D)^{\alpha_i}(\beta+3)}$$

It is clear to see that the fatigue damage accumulation under a stress block cycle depends on the damage states and the stress representative spectrum and the damage accumulation exhibit nonlinear behavior.

Based on the monitoring data, the daily representative block of stress cycle can be obtained with the rain-flow counting method and statistical analysis (Chan *et al.* 2001). As a case study, the representative block of stress cycle from SSTLN-01 is used to evaluate the fatigue life of the Tsing Ma Bridge under traffic loading, as shown in Fig. 13.

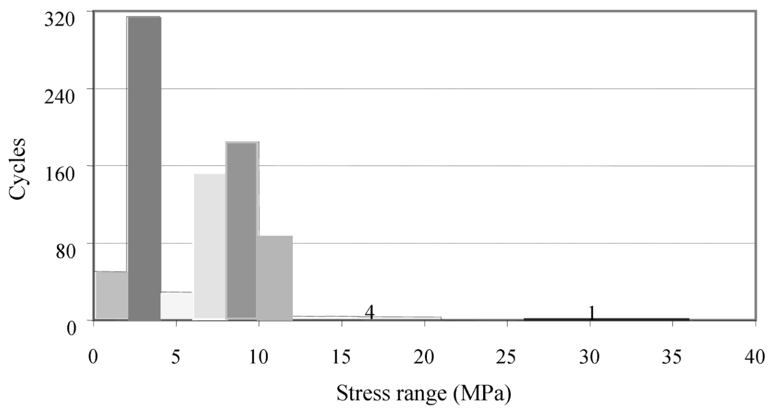


Fig. 13 Stress cycles from SSTLN-01

Highway bridges undergo a large number of repetitive loadings of different magnitudes that are caused mainly by the passages of vehicles. For long span bridges, bridges undergo variable amplitude stress cycles that generally occur in a random sequence. To take the variable amplitude stress cycles into consideration, the effective stress range is used.

The effective stress range is defined by Shilling and Klippstein (1978) $S_{re} = \left[\sum_i \alpha_i S_{ri}^B\right]^{1/B}$

Where $\boldsymbol{S_{ri}}$ is the midwidth of the *i*th bar or interval, in a frequency-of-occurrence bar graph defining the variable-amplitude spectrum and α_i is the fraction of stress ranges with that interval. If B is taken as 2, S_{re}, from Eq. (4), is equal to the root mean square (RMS) of the stress ranges in the spectrum. If B is taken as the reciprocal of the slope of the constant-amplitude S-N curves, the equation is equal to Miner's Law. The fatigue test results show that both the RMS method and Miner's Law effective stress ranges represent the variable-amplitude spectrum satisfactorily, but the RMS method provides a slightly better representation variable-amplitude spectrum (Shilling and Klippstein 1978).

In order to compare the evaluation using the hot spot stress approach, fatigue evaluation using the nominal stress approach has been carried out using both the linear Miner's law and the CDM based fatigue damage model. The fatigue evaluation result is shown in Fig. 14.

The hot spot stress at the weld toe is obtained from the nominal stress multiplied by the SCF calculated from the FEM method. The effective stress range of a representative block of daily cycles for SSTLN-01 location can be obtained with the RMS method with the value of 13.8 MPa. With the assumption that the representative stress cycle block under current traffic loading does not change, the fatigue damage can be calculated using the linear Miner's law and the calculated result shows that the fatigue life is approximated as 300 years under the current traffic loadings. This indicates that the Tsing Ma Bridge is sufficiently safe. Fatigue damage using the CDM (Continuum Damage Mechanics) based model is also carried out to compare with that using the liner Miner's law and the fatigue damage accumulation calculated for the two cases is shown in Fig. 15.

It can be seen from Fig. 15 that the CDM based damage model describes the fatigue damage accumulation as a nonlinear phenomenon. This phenomenon is almost linear during the early

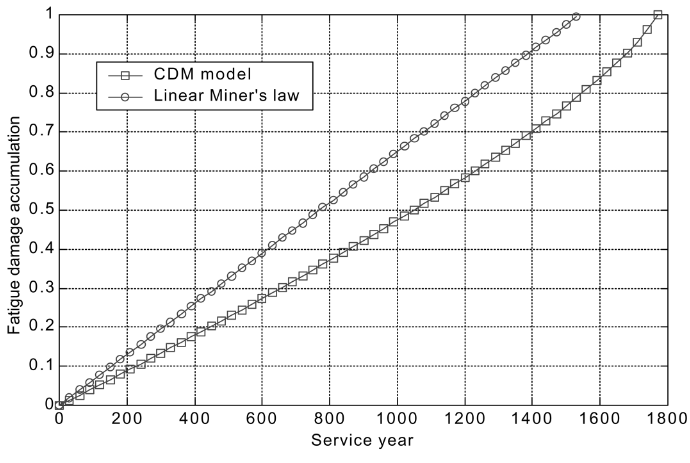


Fig. 14 Fatigue damage accumulation at the location of SSTLN-01 using nominal stress approach

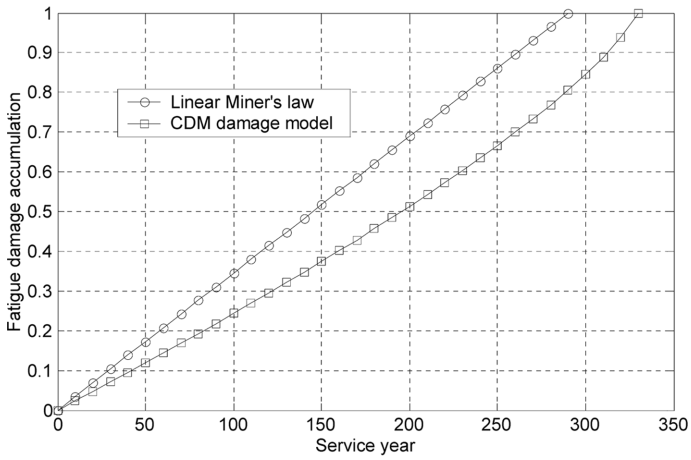


Fig. 15 Fatigue damage accumulation at the location of SSTLN-01 using hot spot stress approach

service years but during the later stages of service, fatigue damage accumulates dramatically. Miner's law adopted in BS5400 (Part 10) determines the stress cycle for fatigue design for welded details and uses the nominal stress range that each member undergoes. The CDM based damage model shows more service years than that by the linear Miner's law and the result given by liner Miner's law shows that the fatigue accumulation has the linear accumulation behavior. Results from both models show that the Bridge under current traffic loading is quite safe at the current stage.

It can be seen from Figs. 14 and 15 that the estimated fatigue damage accumulation is less using the nominal stress approach than using the hot spot stress approach. This is due to the fact that the nominal stress approach does not take the critical fatigue location into consideration while the hot spot stress approach takes the hot spot stress into consideration which is recognized as stress at the most likely failure location. Therefore, the hot spot stress approach is more appropriate for fatigue damage evaluation.

5. Conclusions

The hot spot stress approach method relates the fatigue life of complex steel structures to the real stress distribution, which includes the uneven stress distribution around large steel structural joints. As a case study, the typical cross frame steel joints of the Tsing Ma Bridge have been studied in this paper to determine the SCF (stress concentration factor) in order to provide a basis for fatigue life evaluation based on the hot spot stress approach. Based on the study, the following conclusions can be drawn:

- Local finite element models of the typical joints in the Tsing Ma Bridge have been developed. Extensive numerical analysis has been carried out using different kinds of meshes and the numerical analyses show that results are almost the same. With the load combination case from the dead load and the dynamic load, the local stress analysis has been carried out to derive the SCF (Stress Concentration Factor).
- The load combination case is selected to be representative and sufficient to represent the dynamic response of the member. The SCF under different load case combinations is derived

and the general SCF value is given. With the help of the SCF value, the nominal stress range can be converted into the hot spot stress range.

- As a case study the SCF is applied to convert the representative block of stress cycle into the hot spot stress range. The fatigue life evaluation is made between the hot spot stress approach and the nominal stress approach. The result of the fatigue damage and life prediction based on hot spot stress gives a more appropriate fatigue damage accumulation result compared to that based on the nominal stress approach.

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