Fatigue Cracking in Modular Bridge Expansion Joints

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Single support bar modular expansion joints with 1200 mm of movement capability were used at each end of the Third Lake Washington Bridge between Seattle and Mercer Island on Interstate 90. Within 18 months after the bridge was open to traffic, cracks were observed in the tubular centerbeams of these large modular systems. Additional cracks have occurred since that date. Research was performed to determine the causes of the observed cracking and included an evaluation of existing methods for fatigue design of modular joint systems, finite element analyses of the large modular joints, and correlation of the results with observed behavior. The results show that the cracking is caused by fatigue due to repeated wheel loading. However, existing design methods may not be reliable indicators of the fatigue behavior because the behavior is influenced by the stiffness and dynamic response of the individual joint system. The variable span lengths complicate the evaluation process. The edge centerbeams have the longest and shortest alternating spans and have the highest percentage of fatigue cracks. The dynamic response of the modular joints is complicated because hundreds of vibrational modes contribute to the response, but theory suggests that the response is affected by joint type and loading. The single support bar system amplifies horizontal loads that are applied slowly, but it amplifies vertical loads through a wide range of vehicle speeds. The 1200-mm movement joints will require replacement before the expected design life of 25 to 30 years is achieved.

The Washington State Department of Transportation (WSDOT) uses modular expansion joints on bridges whose expected movements are larger than 127 mm. The Third Lake Washington Bridge has two 1200-mm modular expansion joints at opposite ends of 1.75 km of floating pontoons. These joints, which were open to traffic in June 1989, are believed to be the largest modular expansion joints in the world (*1*). As shown in Figure 1, these joints use the single support bar swivel design, which was developed in Germany. Steel tubes were substituted for the I-shaped centerbeams used in the original design because domestically produced centerbeams were unavailable and FHWA's Buy American steel requirements for federally funded bridge construction would not permit the use of foreign steel. Figure 2 shows the extruded steel rail that was welded to the top of the tubes to grip the strip seals.

Approximately 6 months after the bridge was opened to traffic, WSDOT received complaints of expansion joint noise. Inspection of the joints showed that some elastomeric bearings used to cushion the traffic impact between the centerbeams, stirrups, and support bars were loose. Shims were added, but within a year cracks in the tubular centerbeams were observed. Most of these cracks started at the toe of the stirrup to centerbeam fillet weld and progressed through the centerbeam, as shown in Figure 3. One crack occurred at the end of a reinforcing bar. The manufacturer repaired seven of these cracks in April 1991 by rewelding the cracked metal. Additional cracks were noted in the centerbeams after this first repair, and seven more cracks were repaired in November 1991. Additional cracks were noted after this second repair, and some of the previously repaired cracks reappeared.

WSDOT had concerns about the observed cracking and initiated two courses of action. First, a specification was developed to improve the quality and durability of bridge modular expansion joints. This specification requires fatigue design and testing of joint components to a minimum of 100 million cycles (*2*). Second, a research study was started to evaluate the cause of cracking because there were many special conditions for the Third Lake Washington Bridge, such as substitution of the tubular centerbeams, loss of precompression in the elastomeric springs, heavy traffic, effect of roadway grade, changing lake levels, and long expansion distance (*3*).

REVIEW OF PREVIOUS WORK

In the United States there has been little study of the fatigue life of modular expansion joints. Modular joints are complex because they have many members that move and interact with one another. Each modular system has unique (often patented) features developed by the manufacturer, and these features further complicate the load distribution and evaluation process. However, a relatively simple fatigue limit states design method has been proposed elsewhere (*4–6*). First, the loads on the bridge and the expansion joint are determined. The design limit states fatigue wheel loads, including impact proposed (*4–6*) are a vertical downward load of +91.0 kN and a minimum vertical rebound load of –27.3 kN and a horizontal load of +18.2 kN. The horizontal loads include the effects caused by traffic acceleration and braking. These design limit states fatigue loads are based on field measurements from several bridges in Europe (*6–8*).

By using these loads, the stresses are calculated at critical locations to determine the maximum computed stress range, $\Delta\sigma_{max}$. The centerbeams are treated as continuous beams, and the elastomeric springs and bearings are treated as rigid supports for determination of the moments and the stress level. For normal conditions, each centerbeam carries approximately 50 to 60 percent of the wheel load with a 1.8-m wheel spacing because the wheel distributes the load to more than one centerbeam. Note that the primary loading considered in the design method produces compressive stress in the same area as the fatigue cracking on the Third Lake Washington Bridge. Fatigue design practice historically has focused on the total stress range, and mean stress is ignored (*9*).

From laboratory fatigue tests, separate S-N curves were determined for each critical component or location. Pattis (unpublished data) conducted one fatigue test on the as-built tubular centerbeam

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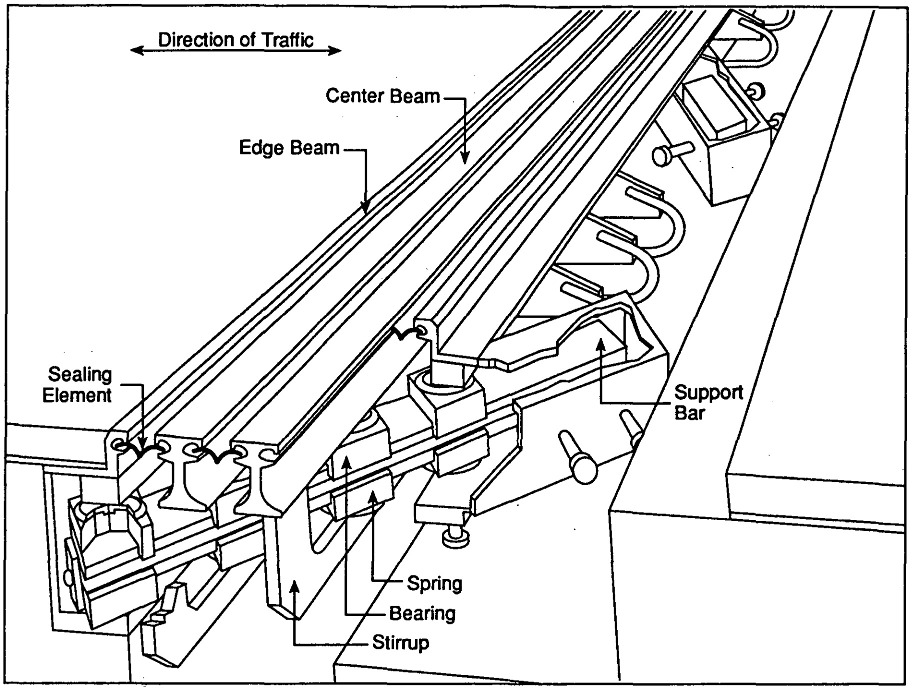

FIGURE 1 Partial plan single support bar swivel expansion joint.

to stirrup welded connection used in the Third Lake Washington Bridge. Figure 3 shows the S-N curve generated from this single test result. The S-N curve is constructed with a slope of –0.33 on the log-log S-N curve for a stress range of less than 5 million cycles and a slope of –0.20 for a stress range between 5 million and 100 million cycles. The intercept at 100 million cycles is the theoretical endurance limit, $\Delta\sigma_L$, proposed elsewhere (*4–6*), but all tests are performed at 2 million cycles or less. Using the proposed fatigue design loads (*4–6*), a fatigue life of 10 million cycles of total truck loading was predicted.

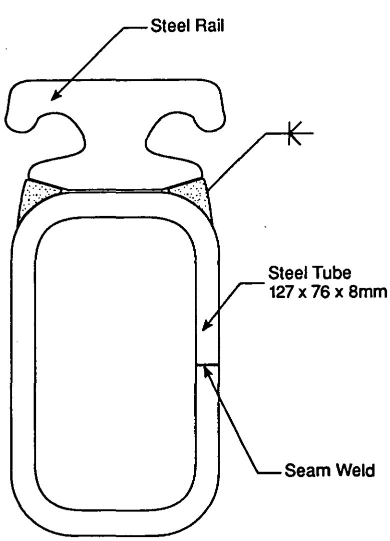

FIGURE 2 Tubular centerbeam with welded rail.

The maximum calculated stress range is compared with the theoretical endurance level from the S-N curves developed from experimental results. The comparison considers the full load spectrum and the accumulation of damage attributable to variable amplitude loading through the combination of Miner's rule and the load spectrum (*6*). The design comparison is made by

$$\frac{\Delta\sigma_{\max}}{2} < \Delta\sigma_L \qquad (1)$$

where $\Delta\sigma_{\max}$ is the calculated stress range on the basis of the defined range of wheel loads, and $\Delta\sigma_L$ is the limit states fatigue stress range at 100 million cycles, as projected from the fatigue tests. The maximum calculated stress range is divided by 2 because of the partial

FIGURE 3 Crack in centerbeam at toe of stirrup weld.

safety factors γ_{Mf}, and α, where α accounts for accumulated fatigue damage by Miner's rule.

There are reasons for questioning the validity of the Tschemmernegg procedure. Koster (*10*) states that the elastic deformation of the system affects the stress distribution and the fatigue potential and contends that deformability of the joint is desirable because it may spread the load and possibly reduce the critical fatigue stress. However, the elastic deformation and stress distribution may not occur because of the very short duration of the wheel loads.

Agarwal (*11*) performed a series of field measurements on a modular expansion joint on a bridge in Ontario, Canada. These field measurements suggested that the loads and load spectrum recommended by Tschemmernegg may not be universally applicable. Large horizontal forces noted by Tschemmernegg (*6*) were not detected and the load range and spectrum were different. However, the centerbeam instrumentation that Agarwal used may not have been adequately located or sensitive enough to detect horizontal loads on the joint.

ANALYSIS OF MODULAR JOINT

The modular joint at the west end of the eastbound lane of the Third Lake Washington Bridge was analyzed with the SAP90 finite element analysis computer program (*12*). As indicated in Figure 4, a global model was analyzed first and the results were used to evaluate local effects near fatigue cracking. The centerbeams, support bars, and stirrups were modeled with beam elements. The geometry, member properties, and stiffness were based on information obtained from the contract shop drawings. The elastomeric springs and bearings were modeled as compression and shear springs, where the spring stiffness was determined by typical models of elastomeric bearing stiffness (*13*). The shop drawings did not specify the elastomer stiffness, and a study was performed to determine the sensitivity of the computed response to variations in the elastomer stiffness. Modest variations in bending moments, deflections, and deformations were noted when the spring stiffness was doubled or when divided by two. The elastomeric spring stiffness did not have a dramatic effect on the bending moments, but other aspects of the behavior noted with elastomeric springs were different from those noted when rigid connections between the support bars and centerbeams were used.

The initial global analyses were performed with a standard, vertical, 71.2-kN wheel load with a 1.8-m wheel spacing. No horizontal load or impact was applied during the initial analyses but was included in later calculations. The static load was distributed to several support beams in the ratio of 25, 50, and 25 percent as proposed by Tschemmernegg (*6*) when the joint opening is in the midrange position.

Bending moment diagrams were computed for the various centerbeams, with vehicle wheel loads passing over four different truck travel paths across the joint. Only one-half the bridge required analysis because the joint is nearly symmetric around the center line. This variation produced the full variation in stress states expected with traffic loading. Figure 5*a* shows the moment diagram of an edge centerbeam (CB13) and its stirrups. The figure shows that the bending moment in the centerbeam was large near the stirrup weld, and it produced tensile bending stress in the centerbeam

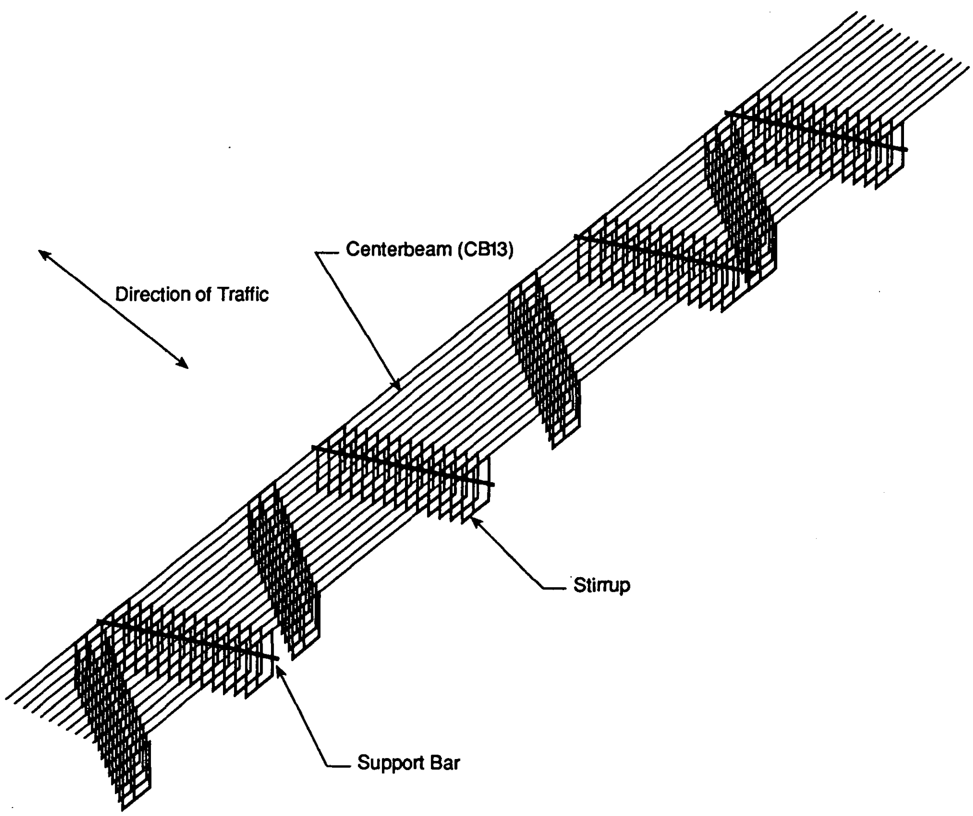

FIGURE 4 Finite element model of entire modular expansion joint LM line.

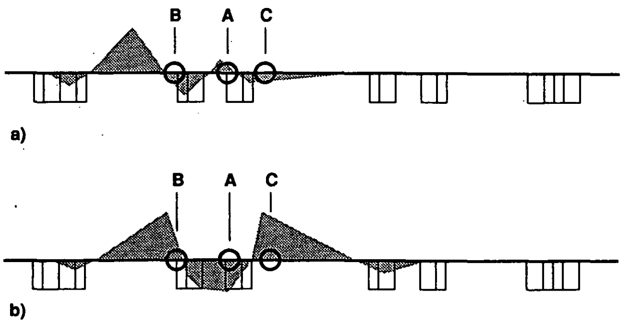

FIGURE 5 Moment diagram for edge centerbeam with (a) truck wheels in outside lane and (b) truck wheels in adjacent lane.

at the stirrup weld at critical Location A and compressive bending stress at the weld of critical Location B. Figure 5*b* shows the same centerbeam with the wheel loads simulating a truck in an adjacent intermediate lane position. The bending moment with this load position produced a sign reversal for the bending moment at all three critical locations. Comparison of Figure 5*a* and *b* shows that the largest stress range may be achieved because wheels are placed in different traffic lanes. AASHTO regards 2 million repetitions of the AASHTO truck loadings as an appropriate number of design load cycles for bridge fatigue. For expansion joints, the number of cycles depends on the number of axles crossing the joint and the number of cycles and the stress range may increase because trucks sequentially pass in adjacent lanes since the largest stress range is caused by this combination.

Similar behavior was noted for other centerbeams, but the bending moments and nominal stress ranges were smaller when the load was placed on interior centerbeams than on edge centerbeams because edge centerbeams had alternating long and short spans and interior centerbeams had more uniform span lengths. The more uniform spans reduced the range of the stress and the bending moments at the critical stirrup locations. Thus, the edge centerbeams experience earlier fatigue cracking than the interior centerbeams.

Global analyses also were performed with horizontal loads applied to the joint. Torsional deformation and weak axis bending of the centerbeams resulted when these horizontal loads were applied at the top of the centerbeam rail. Horizontal loads varying between 5 and 40 percent of the nominal 71.2-kN wheel load were applied and distributed to the centerbeams by the same proportions as those used for the gravity load. The system was surprisingly stiff against these horizontal loads because horizontal deflections of the centerbeam were no more than approximately 8.9 mm with the largest horizontal loads if slip of the elastomeric springs is avoided. The minor axis bending stress at the critical stirrup location was approximately 1.7 MPa when the lateral wheel load was 5 percent of the 71.2-kN gravity load and approximately 13.8 MPa with a 40 percent horizontal loading. Complete reversals were noted when the truck wheels were placed in alternate positions, as noted earlier in the gravity load analysis. The maximum stresses were increased to −49.0 and 59.3 MPa for the load paths of Figure 5*a* and *b*, respectively, when a 40 percent horizontal load was combined with the vertical load. Bending moments for both weak axis and strong axis bending were larger at locations other than the stirrup connection, but fatigue cracks were not likely to form at these other locations because they were not fatigue-sensitive details.

Local Finite Element Analyses

The global analyses showed the bending moments, forces, and deflections of the joints under a wide range of loadings. However, the analyses did not provide a complete picture of the state of stress in the critical stirrup location. The centerbeam and the stirrup were modeled with a detailed local model. The centerbeam was modeled with shell elements, and the stirrups were modeled with three-dimensional brick elements. The loads at the ends of the tube and the spring loads attributable to the elastomeric springs were obtained from the global computer analysis results. Note that the mesh used in this local analysis was appropriate for determining local stress and deformation but was not fine enough to determine stress concentrations or crack initiation conditions.

Local deformations had an impact on the stirrup connection location. The local analysis performed with gravity loads showed only considerable local bending deformation of the walls of the tube near the stirrup weld. The bending stresses caused by these plate bending moments were computed, and the stresses at the critical location were found to be approximately the same magnitude as the basic beam bending stress described earlier in the global analysis. These bending stresses varied from tension to compression through the thickness of the wall of the tube and caused increasing stress (tensile) on the inside of the tube and decreasing stress on the outside of the tube near the stirrup weld in the absence of precompression in the elastomeric springs. If the springs were precompressed, the local bending moments changed somewhat. This change in local bending moments could change the magnitude of the plate bending moment at some locations and ultimately might cause tensile bending stress at the outside of the tube at the critical location.

CORRELATION OF COMPUTED STRESS TO FATIGUE CRITERIA

Efforts were made to correlate the computed stress ranges with existing fatigue criteria. The stresses were computed at the critical stirrup location (Location A on CB-13) because of the large tensile and compressive stresses. More than 60 percent of the visible cracks observed during an inspection of the joints in January 1993 were in similar locations.

Normal AASHTO fatigue design is based on 2 million repetitions of the HS-20 truck loading. The stress range is the difference between the maximum stress caused by the load and its impact and the unloaded condition. The welded stirrup to centerbeam detail is somewhat analogous to Detail 17 in the AASHTO Specifications (*14*) in which attachments are welded to a longitudinally loaded member with short fillet welds. Detail 17 indicates fatigue Category D or E. Two million truck passes will cause far more than 2 million cycles of wheel loading. Category D of the AASHTO Specifications (*14*) requires a maximum stress of 48.3 MPa if more than 2 million cycles is used, and no more than 31.1 MPa is permitted for Category E. However, the existing joint of the Third Lake Washington Bridge has not experienced 2 million cycles of HS-20 wheel loading in the short time it has been in service. This suggests that either the detail is closer to the more critical Category E condition, or the wheel load is larger than 71.2 kN, or the dynamic amplification of the stress range is large.

Different stresses occur at the critical stirrup location when the truck axle passes over a different line of travel, and complete stress reversals are possible when the cyclic stress is caused by these alternate truck path loadings. A stress range of 45.5 MPa, neglecting local bending effects, should be expected with a 71.2-kN wheel load without horizontal load or impact caused by these alternate lane loads. If 30 percent impact is added to this stress range, the range becomes 59.3 MPa. Thus, dynamic amplification clearly raises the stress range to a level well above the fatigue limit for AASHTO Categories D and E.

The analyses were performed with the modular joint in its midrange position. If the joint is opened to its maximum width, the resulting stress ranges are 30 percent larger than those noted earlier. Local bending effects likely play a role in the fatigue cracking, but Category D would predict only several hundred thousand cycles of alternate lane loading when local bending effect is neglected. These ranges do not include any horizontal load. In addition, the stress range used in this evaluation requires passage of two trucks. The trucks do not pass simultaneously, but they pass over the joint in different lanes. It is reasonable to expect up to one cycle of this higher stress range with each truck passage. Additional smaller-amplitude cycles can be expected with each wheel passing over the joint, and this accumulated damage would further reduce the number of cycles of severe loading that the joint could sustain.

Using the proposed Tschemmernegg fatigue design loads and the S-N curve shown in Figure 6, a fatigue life of 10 million cycles of total truck wheel loading was predicted for the as-built stirrup-to-centerbeam connection. As indicated in Figure 7, this estimate is different from the AASHTO (*14*) and AASHTO LRFD (*15*) life estimates because it includes the total number of truck passings and an estimate of accumulated damage. The accumulated damage estimate is based on a design wheel load spectrum proposed for expansion joints in Europe. Fatigue cracks were noted approximately 18 months after the bridge was opened to traffic, and 10 million cycles would require approximately 18,000 axles for one lane of traffic per day. A traffic count performed in 1990 found that the westbound lanes of the bridge experienced approximately 6,720 axles of bus and truck traffic during the busiest 12-hr period of a normal work day. When the traffic was distributed over three lanes and the lighter weekend traffic was considered, the accumulated traffic was less than 20 percent of the fatigue life estimate proposed by Pattis (unpublished data). Further, the cracks obtained in the laboratory fatigue test were quite different than those observed on the Third Lake Washington Bridge. The initial and predominant cracking in the laboratory test was longitudinal cracking along the edge of the stirrup to center beam weld. This cracking is different from the transverse-through-depth cracking seen on the Third Lake Washington Bridge and illustrated in Figure 3. No longitudinal cracking has been noted on the Third Lake Washington Bridge. Transverse cracking was eventually noted on the test specimen, but it occurred only after the longitudinal crack had grown large and it did not progress through the depth of the centerbeam (Pattis, unpublished data). This observation suggests that the Tschemmernegg procedure may not be applicable for all joints.

Although the Tschemmernegg method does not duplicate the fatigue cracking noted in the Third Lake Washington Bridge, the stress ranges predicted by the test may be fairly realistic. The reason for the approximate accuracy of the S-N curve is that the modular joint details are likely to always be close to AASHTO Category D or E because the weld detail is similar to that of AASHTO fatigue Details 9 and 17. The detail may be closer to Category D or even Category C if the modular joint is less susceptible to fatigue and closer to Category E if it is more susceptible. However, it is clear that the load history is most important for establishing a fatigue design criteria.

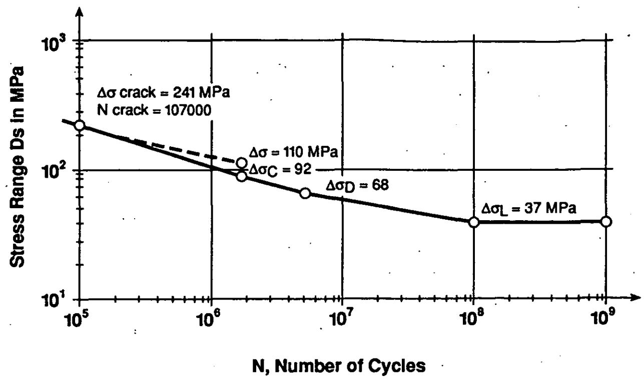

FIGURE 6 S-N curve proposed for tubular centerbeams (Pattis, unpublished data).

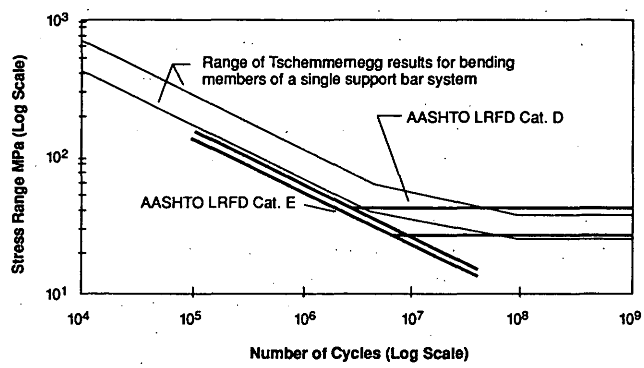

FIGURE 7 AASHTO LRFD S-N curves compared with a range of different Tschemmernegg S-N curves.

The static analysis shows larger stress ranges than those suggested by the Tschemmernegg method because of variations in the travel path of trucks across the modular joint and the alternating long and short spans of the edge centerbeams of this joint. Nearly complete stress reversals are possible because of these conditions. The Tschemmernegg method is based on application of an accumulated damage model that is based on load spectrum data from several expansion joints in Europe. There is no indication of how wheel loads vary from location to location in the United States. The fatigue behavior of modular joints may not be as simple as that proposed by Tschemmernegg.

DYNAMIC ANALYSIS OF MODULAR JOINT

The global finite element model was used to perform dynamic analyses on the modular joint. The mass of the components of the modular expansion joint were added to the model, and many modes of vibration were computed. No damping was used because of uncertainty about its magnitude. However, damping must be relatively large (20 percent of critical or more) before significant changes in the dynamic periods are noted. The dynamic modal computations required a large amount of computer time because of the broad distribution of mass and stiffness and the large number of degrees of freedom. In most modal analyses, only a few modes of vibration are required because the modes are well spaced and most of the mass is participating in very few modes. The modes of vibration for this modular expansion joint were closely spaced with hundreds needed to include the predominate portion of the mass in three-dimensional vibration. Nevertheless, the procedure produced some general observations worth noting.

The longest period modes were associated with horizontal movement. The majority of the participating mass (98+ percent) for the two translational degrees and one in-plane rotational degree of freedom were included in modes with periods of between 0.16 and 0.035 sec (6.2 to 28 Hz). These translational degrees of freedom occurred because of deformation of the elastomeric springs. Even though the stiffness of these springs was not precisely known, a 100 percent increase in stiffness would decrease the period only by approximately 30 percent. A 50 percent decrease in elastomer stiffness would increase the period by approximately 40 percent. These variations in elastomer stiffnesses are possible, but they represent upper limits on the probable variation.

Two typical vertical modes of vibration in a centerbeam located near the edge of the joint (CB13) are shown in Figure 8. The vertical modes of vibration with significant participating mass had periods ranging from 0.05 to 0.005 sec (20 to 200 Hz). There were many similar closely spaced modes, each with a modest participating mass. However, the period of these vertical modes of vibration was about 0.015 sec (67 Hz) for most of the participating mass of the system.

In past inspections of the Third Lake Washington Bridge joints, inspectors noted that the elastomeric bearings were sometimes loose and not precompressed. A lack of precompression reduces the stiffness of these bearings because they cannot act in tension without the precompression. As a result, several analyses were performed to evaluate the effect of loose bearings. Individual loose bearing might double the period of a single critical mode but have minimal effect on most modes of vibration. An increased number of loose bearings can increase the period of a larger number of modes of vibration, but the relative magnitude of the period increase would often be smaller than that noted for a single mode.

Impact represents the dynamic amplification of the system attributable to the dynamic loading. The wheel load on any centerbeam is initially 0 until the wheel makes contact with the given beam, and it reaches its maximum value when the wheel is nearly centered over the given centerbeam. The load on a beam then decreases until the wheel separates from the beam. If the truck is traveling at a constant velocity, this translates into the linear time-dependent load function shown as an insert in Figure 9. For a tire contact length of 24 cm and a centerbeam width of 80 mm, vehicles at 33, 67, and 100 km/hr would have load durations of 0.035, 0.017, and 0.012 sec, respectively.

Figure 9 shows the maximum dynamic response divided by the static response as a function of the ratio of the duration of the ramp function loading to the period of the system. The dynamic amplification of 1.0 indicates that the structure feels the full static loading, and a factor greater than 1.0 implies impact or dynamic amplification. Figure 9 shows that the centerbeam feels the full static load and potential impact if the duration of the loading is longer than approximately 30 percent of the dynamic period of the structure. If the duration is less than 10 percent of the period, less than 30 percent of the static load is felt. The maximum duration is approximately 0.035 at a truck speed of 33 km/hr, and this duration is similar to the shortest dynamic periods associated with horizontal movement and deformation. The duration at 100 km/hr is 0.012 sec, and this is a small percentage of all but the shortest periods associated with horizontal movement. This suggests that significant amplification of horizontal forces should be expected at slower vehicle speeds. High-speed vehicles may cause the expansion joint to experience the static force or slight attenuation of horizontal loading. Therefore, this expansion joint may not experience the large horizontal loads suggested by the Tschemmernegg method (*6*). Comparison of load duration to vertical modes of vibration suggest that amplification will occur over a wide range of vehicle speeds. These observations are meaningful for this particular expansion joint because of the transverse flexibility of the system. Other modular joint systems, particularly multiple-support bar joints with a rigidly welded centerbeam to support bar connections, may be much stiffer transversely and feel a greater horizontal loading and possible dynamic amplification.

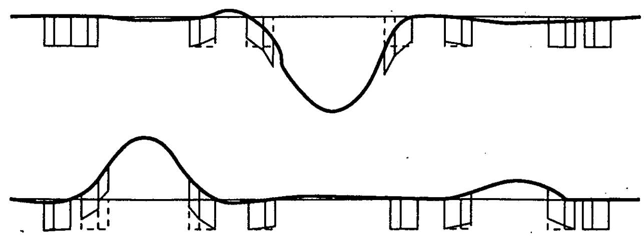

FIGURE 8 Typical vertical modes of vibration.

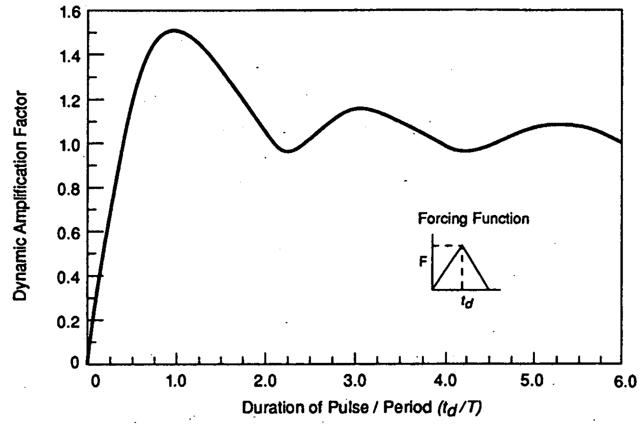

FIGURE 9 Dynamic amplification for ramp function loading.

These analyses neglect the effect of the vibration of the truck suspension systems and the additional impact caused by rough roadway surfaces. Additional amplification is possible when these factors are considered, but the maximum amplification always will occur when the duration of loading (t_d) is similar to the periods of the centerbeam and the truck suspension system.

SUMMARY

1. An analytical study of centerbeam cracking observed in the large modular expansion joints on the Third Lake Washington Bridge is described. The cracking is a result of fatigue caused by cyclic loads induced by truck wheel loads on the joint.

2. The fatigue problem is most serious in the edge centerbeams because of the larger stress range produced by the alternating long

and short spans. Residual stresses near the stirrup to centerbeam weld may cause the cyclic compressive stress to be in cyclic tension.

3. The tubular centerbeams contribute to the fatigue problem because of local deformation and through-thickness plate bending stress, but fatigue would have been a problem even if another section had been used for the centerbeams because of the centerbeam span length.

4. Wheel loads cause multiple stress cycles for a single truck passage. Therefore, the fatigue evaluation procedure of modular expansion joints must be different from that used for bridge girders. The analyses showed that a much larger stress range is possible between different trucks because the trucks do not travel over the same path across the joint. This variability may double the stress range over that computed for a single wheel load.

5. The elastomeric springs and bearings are an important element in the joint behavior. However, frequently they have been reported as being loose in the joints. The precompression or looseness of the bearings will affect local bending stress in the critical region surrounding the stirrup and may also lengthen periods of critical modes of centerbeam vibration. However, precompression is not thought to be a predominant contributor to the fatigue problems noted in the joints.

6. The Tschemmernegg design load spectrum and S-N curves are based on field measurements, analysis, and fatigue tests of modular joints in Europe. Fatigue design criteria for modular joints must consider the unique features and dynamic response of each joint system. The fatigue test must be appropriate for the loads the joint experiences, or it will lead to improper S-N curves and failure modes. The fatigue test on the as-built tubular centerbeam may not be indicative of the fatigue behavior of this joint because the fatigue test does not include the flexibility of the joint with respect to horizontal loads.

7. Welded repairs are not an effective long-term repair solution because most of those previously repaired by welding have recracked.

8. The AASHTO specifications should include fatigue design loads, allowable fatigue stress ranges, number of cycles to determine the theoretical endurance limit of fatigue critical expansion joint components, and expected design life for modular expansion joints.

9. The dynamic behavior of each type of modular joint system is strongly influenced by the dynamic response of that system. This single support bar expansion joint amplifies horizontal loads that are applied slowly, but it amplifies vertical loads through a wide range of vehicle speeds. The vertical modes of vibration with significant participating mass had periods ranging from 0.05 to 0.005 sec.

11. Single support bar expansion joints may not experience the large horizontal or lateral loads because of the transverse flexibility of the joint.

12. Dynamic analyses can be useful in determining the dynamic behavior and amplification or impact factor used in the design of modular expansion joint systems. However, the impact factor under field conditions may be significantly higher because of road roughness and the various dynamic characteristics of truck suspension systems.

13. The 1200-mm movement modular joints will require replacement before their expected design life of 25 to 30 years is achieved.

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