Stay Cable Fatigue
Design Loading

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Cable stayed bridges have today emerged as a very popular bridge type for medium and long span bridges in the United States and coincidentally precast and prestressed concrete has assumed the status of the favored material in the design of these structures.

Since completion of the first major cable stayed bridge at Pasco-Kennewick (late 1970s) in the state of Washington, some other long span cable stayed bridges have been constructed, most notably the East Huntington Bridge in West Virginia (Fig. 1) and the recent Sunshine Skyway Bridge in Florida. Several other cable stayed bridges are now in design and construction across the United States.

It might be assumed that the cable stayed bridge form is something new. However, this is not the case. Cable stayed bridges were designed throughout the 19th century and even earlier. Crude cable stayed bridges built around the world still survive to this day. Such an example is the Chow Chow Bridge (see Figs. 2a and 2b) in the state of Washington. The bridge, built in the early 1950s, shows the intuitive imaginative skill of the western lumberman without the benefit of modern materials and engineering expertise.

In retrospect, one wonders, then, what factors have inhibited the development of cable stayed bridge design until more recent times? Probably the two most significant factors were analytical complexity and stay cable fatigue. The former has been largely conquered by the computer, the latter by modern materials technology.

Modern cable stayed bridges are designed as beam columns on an elastic foundation. The cables are closely spaced along the length of the deck to

Note: This article is based on material presented by the author at the First CIB Major Concrete Bridge Conference in New York City, April 2, 1986.
provide stiffness to the deck structure and to permit girders to be relatively free of the large bending moments that control the design of more conventional long span structures.

The material efficiency of steel is greatest in direct tension, and it is with direct stay tension that the major loads are carried in a cable stayed bridge. The superior stiffness of stays, along with the general absence of massive cable anchors, favors stay cable designs over competing suspension designs for increasingly longer spans.

The leading developers of modern stay systems are the major post-tensioning specialists that have been serving the prestressed concrete industry in recent years. Two major types of cable systems currently in favor are the parallel wire and the strand cable systems, with the former shown in Fig. 3.

The distinction between these two cable types and their prestressing counterparts is the stay anchorage. Manufacturers of these stay types have undertaken extensive research and testing programs in developing proprietary anchorage designs, and past project specifications for cable stays have required extensive qualification testing prior to acceptance for construction.

In recent years, requirements for stay designs and the general capabilities of various design types have come into better focus through the expanding experience afforded by actual construction. However, project specification requirements have not focused as quickly as cable stay development.

Some project specifications called for different fatigue loadings for alternate cable systems permitted on the same bridge. The cost of the stays can be significant — often as much as 10 percent of construction cost. Hence, practices affecting stay supply have influenced project cost in the past. Clearly, the basis for stay cable requirements needs some focus.

In response to this situation, the Post-Tensioning Institute appointed a special ad-hoc committee to assimilate the available information on stay cable design, and draft it into a format compatible with current AASHTO specifications. It is important to keep the AASHTO guideline in mind throughout this presentation. The PTI committee did not attempt to modify or modernize the AASHTO criteria in any way other than to encompass stay cable design within the framework of the AASHTO specifications.

History

Designs to date have been based on variations of AASHTO Sections 3.0 and 10.3 for loads and fatigue, respectively (references are to the 1983 edition of AASHTO, even through previous editions were used in design).

Maximum live loads for cables are determined according to Section 3.6.4, resulting in increased loading for twin cable plane systems (Fig. 4).

Influence lines are computed in the

Synopsis

A focal point in the design of prestressed concrete cable stayed bridges is the performance of the stay cables. The most critical difference between modern stay cables and the more traditional post-tensioning cables is the influence of fatigue on design.

This article will review the history and state of the art of stay cable design and practice and discuss the influence of highway loading spectra on stay cable fatigue requirements.

An Appendix section provides a detailed truck traffic simulation fatigue analysis of selected stay cables of the Pasco-Kennewick and East Huntington Bridges.
Fig. 1. Close-up of stay cables at anchorage tower of East Huntington Bridge.
usual manner for stay cable forces, and
appropriate lane and truck loads from
Sections 3.11 and 3.12 are applied.

In order to conform with the empirical
formula for impact factor in AASHTO,
the span length (L in AASHTO) is taken
as the length of the loaded influence
line, and impact computed in accor-
dance with AASHTO Eq. (3.1). This
static load analysis fits reasonably well
into the AASHTO format for more typi-
cal structures.

The load specifications for fatigue
have been similar to those of Table
10.3.2A for main load carrying mem-
bers. AASHTO truck loading has been
applied at 2 million cycles, full
AASHTO lane loading has been applied
at ½ million (or 700,000) cycles, and
cable stress ranges determined as a def-
inition of fatigue criteria. Variations of
these two criteria have generally been
established as acceptance criteria for
stay cable suppliers to meet in their
project by project acceptance testing.

Stay Behavior

Extensive testing carried out for proj-
ects in the United States, Japan, and Eu-
rope has resulted in a substantial
amount of data on stay cable systems.

Stay anchors have been designed to de-
velop fatigue capabilities approaching
those of the strand or wire in the free
length of the stay cables.

Major research on wire and strand
fatigue strengths has been carried out in

Fig. 2a. Overall view of Chow Chow Bridge (built by western lumbermen).

Fig. 2b. Anchorage tower of Chow Chow Bridge. For overall view, see Fig. 2a.
Fig. 3. Details of stay cable socket (parallel wire system).

Fig. 4. Lateral distribution of live load for East Huntington and Pasco-Kennewick Bridges.
Switzerland and at the University of Texas. Historical data on strand and parallel wire stay cable systems were assembled by the PTI committee, and evaluated along with the bare element data mentioned above.

In keeping with the philosophy of the committee to work within the guidelines of AASHTO, the data were evaluated against the provisions for allowable fatigue stress range in Table 10.3.1A. In Fig. 5 the cable allowables are compared with AASHTO Category B. This category was adopted as a reference for both strand and wire cable systems.

**Live Loads**

The AASHTO provisions have been developed through the years of American experience in bridge design. The vast majority of this experience is with short and medium span bridges. The focus of the present criteria is clearly on short span, multi-girder structures, and independent, determinant components in long span systems that can be treated as though short in span.

Most of the literature on the subject of highway bridge fatigue addresses the single truck as the prime load in evaluating fatigue demand on steel structures. Nevertheless, in some way lane load became specified in AASHTO as a fatigue load case.

Fig. 5 illustrates the relative capability of stay cable systems at ½ and 2 million load cycles. It is readily apparent that, for long span structures, the stress range for backstay components due to a full lane load will be significantly greater than that for a single truck load.

Often, the lane load stress range for a backstay cable will be approximately three times that for truck loading, while the capacity increase, as shown in Fig. 5, is only 50 to 70 percent. Therefore, in most cases, the full lane load at ½ million cycles controlled the fatigue design criteria for stay cables.

Live load influence lines for backstay cables reflect the broadest exposure along the typical bridge. Figs. 6 and 7 show the influence lines for the East Huntington and Pasco-Kennewick backstays. Also shown in Fig. 7 for comparison is the influence line for an interior cable. Using the aforementioned criteria for lane loading, the extreme behavior of
full cyclic loading on the backstays can be seen.

The influence line geometry requires a highly improbable pattern of lane loading — one that fills the negative region while void in the positive, followed by one that fills the positive region while void in the negative. Couple this planar representation with multilane effects, and it can be quickly seen why many bridge engineers believe the high cycle lane loading to be unreasonable for design.

In formulating the PTI recommendations for fatigue loading, the committee chose to incorporate a more practical loading level for stay fatigue. The standard AASHTO multilane truck loading was adopted as the governing load for cable stays, thus eliminating full lane load from consideration when evaluating cable stays for fatigue.

In this manner, the actual spanwise following of trucks is represented by a widthwise alignment, with all AASHTO design trucks at the maximum influence line ordinate acting at the same instant.

**Case Studies**

A variety of traffic surveys has been reported in the literature, classifying the statistics of truck traffic on interstate and urban highways. While the actual figures vary from site to site, the general character of the resulting gross weight distribution curves is fairly consistent for sites where loads are regulated to the AASHTO design level. Refs. 3 through 7 are but a few of the sources for such information.

Two case studies serve to illustrate the effect of the PTI recommendations for fatigue. Both the Pasco-Kennewick and East Huntington cable stayed bridges were designed using criteria similar to that described above. Figs. 8, 9, and 10 compare the original requirements for fatigue load (impact and local effects are omitted) with those of the new PTI recommendations (the HS-20 truck). Also shown is the result of a simple Monte Carlo traffic simulation using the data and technique described in Appendix A.

This simulation is itself conservative, for it assumes the same traffic pattern.
Fig. 7. Influence lines for Cables No. 4 and 9 (Pasco-Kennewick Bridge).
over influence line maxima in both directions. The original design criteria are shown to be far more conservative than the PTI recommendations. The traffic simulation proves to be very compatible with the PTI recommendations for both the four-lane Pasco-Kennewick Bridge and for the two-lane East Huntington Bridge.

Comparison of the traffic simulation results in Figs. 9 and 10 shows that both the interior cable (#9) and the backstay cable (#4) should undergo about the same live load stress range. This is a notably different result than that obtained when lane loading is considered for fatigue. Comparisons of this sort will be sensitive to the assumptions in the traffic simulation, but these results do point out that fatigue is not only limited to the backstays.

The backstays on a cable stayed bridge respond to movement of the tower rather than the deck, and thereby pick up load from all portions of the bridge. The degree to which opposing backstays in a twin cable plane bridge combine in resisting general traffic loading depends mostly on the response of the tower.

In the case of East Huntington, with its A-frame tower and single towerhead, the backstays work in tandem for general traffic loads. For this simplified case, the degree of conservatism of the single cable plane analysis can be assessed by performing a planar simulation of two independent lanes of truck traffic, resisted by two backstays. The results from such an investigation is superimposed over the East Huntington simulation in Fig. 8.

The results show the sizeable margin in fatigue load for this bridge when using the PTI recommendations. Since the tower effect will not influence the interior cables, the latter may see higher fatigue stress on East Huntington than the backstays.

Table 1 is taken from the PTI recommendations, and shows the general design criteria for stay cable sections. PTI requirements are divided into three categories, namely, final design values,
Fig. 9. Fatigue simulation study of Pasco-Kennewick Bridge, Cable No. 4 ($\lambda = 5.0$ trucks per minute).

Fig. 10. Fatigue simulation study of Pasco-Kennewick Bridge, Cable No. 9 ($\lambda = 5.0$ trucks per minute).

Table 2 summarizes the fatigue criteria of PTI and compares the PTI criteria to those used for original design. In the design stage for these two
Table 1. Summary of fatigue stress range values $F_{st}$ (ksi)\(^{(a)}\)

<table>
<thead>
<tr>
<th>Type of stay</th>
<th>No. of cycles (b)</th>
<th>Allowable design fatigue stress range</th>
<th>Stay test fatigue stress range (c)*</th>
<th>Component fatigue test stress range (d)*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Redundant load path</td>
<td>Nonredundant load path</td>
<td></td>
</tr>
<tr>
<td>Strand (e) or uncoupled bars (f)</td>
<td>$2 \times 10^8$</td>
<td>16</td>
<td>16</td>
<td>23</td>
</tr>
<tr>
<td>AASHTO Category B</td>
<td>$5 \times 10^6$</td>
<td>27.5</td>
<td>18</td>
<td>37.5</td>
</tr>
<tr>
<td>Wire (g)</td>
<td>$2 \times 10^8$</td>
<td>21</td>
<td>21</td>
<td>28</td>
</tr>
<tr>
<td>AASHTO Category B</td>
<td>$5 \times 10^6$</td>
<td>32.5</td>
<td>23</td>
<td>52.5</td>
</tr>
<tr>
<td>Bars (f) with (epoxy filled) couplers AASHTO Category D</td>
<td>$5 \times 10^6$</td>
<td>15</td>
<td>10</td>
<td>23.5</td>
</tr>
</tbody>
</table>

(a) Any flexural stress range in excess of 3 ksi shall be added to the axial fatigue stress range due to live load plus impact.
(b) See Table 10.3.2A.
(c) To ensure fatigue quality of stays, it is recommended that the stay specimens be tested at $2 \times 10^8$ cycles.
(d) Individual strand, bar, wire; or glued, coupled bar, respectively.
(e) See Section 3.2.2.
(f) See Section 3.2.3.
(g) See Section 3.2.1.
* Upper bound stress level shall be 0.45 $f_t$.

bridges, fatigue was not used as a criterion to limit the cable stresses. Favorable geometries were chosen to limit backstay stress range as much as possible. Both were constructed using parallel wire cables, permitting somewhat better fatigue strength relative to static strength, so these examples are not universally applicable. However, these two bridges do cover a wide range of application, and these results are applicable to many similar bridges.

Discussion

It can be seen that design for fatigue need not govern in either example for strand or wire systems. The margin between typical fatigue stresses and testing requirements is substantial enough to accommodate significant increases in direct tension loading.

Current research\(^7\) is focusing on traffic weigh-in-motion results that reveal significantly heavier, large truck traffic. At the same time, materials research\(^8\) is revealing that the security of a fatigue endurance limit may not be available in cases where there are infrequent but large stress excursions beyond that limit. Sample analyses for assumed truck weight distributions with weights up to 150 kips (667 kN) indicate that the 95 percentile load does not increase significantly, but the extreme loading does.

The PTI recommendations call for
Table 2. Stay cable fatigue design and test values.

<table>
<thead>
<tr>
<th>Item</th>
<th>No. of cycles</th>
<th>Design stress range (ksi)</th>
<th>Test stress range (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pasco-Kennewick</td>
<td>0.5 x 10^6</td>
<td>18</td>
<td>—</td>
</tr>
<tr>
<td>Cable No. 4</td>
<td>2.0 x 10^6</td>
<td>—</td>
<td>24</td>
</tr>
<tr>
<td>East Huntington</td>
<td>0.5 x 10^6</td>
<td>30.2</td>
<td>33.6</td>
</tr>
<tr>
<td>Cable No. 1</td>
<td>2.0 x 10^6</td>
<td>7.0</td>
<td>24</td>
</tr>
<tr>
<td>PTI recommendations</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wire</td>
<td>0.5 x 10^6</td>
<td>32.5</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>2.0 x 10^6 +</td>
<td>21</td>
<td>28</td>
</tr>
<tr>
<td>Strand</td>
<td>0.5 x 10^6</td>
<td>27.5</td>
<td>23</td>
</tr>
<tr>
<td></td>
<td>2.0 x 10^6 +</td>
<td>16</td>
<td>—</td>
</tr>
</tbody>
</table>

*Local effects are not included.

Note: 1 ksi = 6.895 MPa.

testing all stay systems to the levels of their capability. Inspection of Figs. 8, 9 and 10 shows that the current recommendations appear to conservatively cover the runout condition, as well as provide ample margin for increase in truck weights beyond the current AASHTO requirements.

There are other considerations in stay design that must be attended to — some which can be more significant than the direct axial stresses discussed here. The design of local detail around the cable anchorage, and the manner in which local flexural stresses are accommodated in the tower and deck transitions are the more likely areas where cable distress will first occur.

The Neoprene dampers first used for the Pasco-Kennewick cables are one way of limiting stress concentrations at cable anchorages. These dampers are circumferential rings of Neoprene, placed in the cable anchor regions to soften the transition from the free, flexible cable section to the rigid cable anchor. While reducing the flexural stress imposed on the anchor region due to live load displacements, they also serve as vibration dampers for wind induced vibration, and as cushions to prevent fretting of the cable sheath and wire.

CONCLUSIONS

These examples show that for properly detailed stay cables on typical highway bridges employing the present quality of stay cable systems, direct axial fatigue considerations need not affect the economy of concrete cable stayed bridges.

The PTI recommendations are an excellent state of the art summary for cable stay systems. While structured to conform to the AASHTO design specification, they are generally applicable to cable stayed structures of all types, provided appropriate load models are considered. The focus on AASHTO truck loading for fatigue stress can be applied as changes to the AASHTO truck loading develop.

Considering the information in Ref. 9, and other modern assessments of highway loading, revised loading spectra may indeed be appropriate for maximum design loading as well as fatigue. The methods presented here for illustration are suitable for establishing fatigue criteria on long span cable stayed bridges for any future loading spectra.

The National Cooperative Highway Research Program is currently sponsoring research into fatigue in cable stayed bridges. Their report, for NCHRP Proj-
ect 12-30, is due out in approximately 2 years, and should add additional insight on this issue.

The Appendix section gives a detailed truck traffic simulation fatigue study of selected stay cables of the Pasco-Kennewick and East Huntington Bridges.

REFERENCES


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NOTE: Discussion of this paper is invited. Please submit your comments to PCI Headquarters by February 1, 1988.
APPENDIX A — TRUCK TRAFFIC SIMULATION

Background
The fatigue of properly detailed stay cables occurs under fairly high cycle repeated loads. Design lane loads represent maximum pattern loads that occur seldom, if ever, in the life of most long span bridges. High cycles of lane loading require that highly improbable pattern loading develop repeatedly over the life of the structure. For this reason, virtually all of the literature and research on fatigue loading for bridges deal with truck loads, not lane loads.

Design
The PTI recommendations call for the use of the full AASHTO truck design loading as the fatigue loading. Using the full complement of multilane truck loads, along with the required shift in lane pattern to one side of the bridge (see AASHTO Section 3.6.4) for twin cable plane structures, results in more than one truck per lane at the maximum influence line ordinate for each stay cable. The maximum tension and maximum compression combine to give the stress range for design.

Traffic Simulations
Traffic studies (Ref. 4) indicate that at least 80 percent of the heavy truck traffic travels in the outside lane of a multilane expressway. A variety of studies over the years (Refs. 3 through 7) has been used to develop a picture of truck traffic patterns for both highway and urban traffic. Fig. A1 shows one such pattern, taken from Ref. 3. This probability density function is qualitatively similar from one locale to another. Fig. A2 shows the cumulative probability distribution for the truck weight data in Fig. A1.

The arrival time of traffic is generally characterized by a Poisson arrival process (Refs. 5 and 10). Ref. 5 supports this.

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Fig. A1. Gross weight distribution curve for traffic.

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model with actual traffic statistics for an interstate study in Ohio. Using this formulation for the arrival time of trucks, and converting this time at 45 miles per hour (72 km/hr) to a headway distance, one can assign headway probabilities in an analytical formulation, and use these arrival times in conjunction with the truck weight representation discussed above.

A Monte Carlo type simulation was set up using the above probability distributions for truck weight and arrival times. In a Monte Carlo simulation, the probabilistic nature of truck traffic is modeled using the random number generating capabilities of a computer. The effect is to create repeated computer simulated experiments, and corresponding histograms of the results. Here both the truck weight and the truck spacing is modeled with probability density functions as described, with each set of values chosen at random by the computer.

Using a random number generator, successive trains of trucks were "run" over the influence lines for several cables to determine the stress range in each cable. The number of truck trains was increased until the statistics of response (mean and standard deviation) stabilized. One truck lane was assigned to each cable plane. Thus, it is assumed that the truck train loads occur simultaneously in each direction—a conservative assumption, but one in keeping with the AASHTO loading. Note that local effects, such as anchor zone flexure and impact, are omitted in these illustrations.

The process is outlined next.

**Poisson Arrival Process**

The probability density function, \( f(t) \), is taken as \( f(t) = \lambda \exp(-\lambda t) \), where \( \lambda \) equals the average number of trucks per minute. Then the probability of arrival being within time \( t \) is \( F(t) = 1 - \exp(-\lambda t) \).

With a random number generator, the above probability is assigned using \( Rx = 1 - \exp(-\lambda t) \) \([f(Rx) = 1, 0 < Rx < 1]\). Arrival time is then solved for, and headway distance is computed assuming a speed of 45 miles per hour (72 km/hr) for trucks. Truck trains are ass-
sembled by repeating the above process until the length of the truck train exceeds the length of the bridge. Truck weights are also determined using the random number generator for the probability of non-exceedence of a given truck weight as depicted in the distribution curve (Fig. A1).

**Analysis**

The response of chosen cables to the truck trains is determined by assigning one truck lane to each cable plane. Truck loads are treated as point loads as truck trains are moved over the planar influence line for the cable. Maximum and minimum stresses are recorded for each truck train, and stress range statistics are accumulated across all of the truck train trials. The mean and standard deviation for stress range stabilized at approximately 200 cycles for East Huntington's backstay, and about 400 cycles for Pasco-Kennewick's backstay. The results are reported for 1000 cycles for both bridges.

Lambda ($\lambda$) is taken as 1.5 trucks per minute for East Huntington and 5.0 trucks per minute for Pasco-Kennewick. Both of these values are conservative considering the present use of these facilities, especially for Pasco-Kennewick. Since East Huntington is a two-lane structure, truck headways were limited to following distances. For Pasco-Kennewick, short headways represent side by side truck travel.