ANALYSIS OF DAMPING IN EARTHQUAKE RESPONSE OF CABLE-STAYED BRIDGES

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SUMMARY

For seismic design and analysis of major cable-stayed bridges it is typical to assume viscous structural damping at 5%. However, there appears to be little empirical basis for use of this value for cable-stayed bridges, other than it is the "traditional" number to use. Even when more sophisticated procedures (e.g., time history) are used in the seismic analyses, 5% damping is often still the target value. This paper reviews some of the background to evaluation and selection of damping for dynamic response of this type of structure. Results are presented from examination of strong motion records from the Suigo Bridge in Japan, a simple two-span cable-stayed bridge and one of only two such bridges where strong motion earthquake responses have been obtained. By comparing the actual seismic acceleration records to those predicted by finite element analysis it is observed that the Suigo Bridge exhibited low damping, typically between 0.5% and 2%. This occurred for a situation where the peak ground acceleration was 0.12 g and the peak structural response was 1.0 g. These limited results suggest that the common assumption of 5% damping may be too high (unconservative) for some applications. A comparative study of the seismic response of four cable-stayed bridges is used to examine the implications of low damping

INTRODUCTION

The damping most often assumed for seismic analysis of buildings and bridges is 5%. The choice of this 5% value is often made for traditional reasons, and experimental evidence for many buildings suggests that 5% is 'about right'. However, for cable-stayed bridges this 5% value may not be appropriate because the seismic energy dissipating mechanisms that exist in buildings often do not exist in bridges. For major cable-stayed bridges, where response at the maximum design earthquake level may be essentially elastic, there is little or no opportunity for energy dissipation through plastic deformation. One of the significant problems encountered in supporting this contention however is that information on the actual damping in cable-stayed bridges during strong motion response is very sparse.

This paper examines some of the uncertainties surrounding damping values used in seismic response analysis of cable-stayed bridges. A brief summary of a review of the literature on damping in cable-stayed bridges is presented. Strong motion records obtained on the Suigo Bridge, a cable-stayed bridge in Japan, are used for further study, along with structural models of the bridge. These strong motion records are one of the few sets of strong-motion data from a cable-supported bridge. Implications of various assumptions of damping levels are also examined.

DAMPING FOR SEISMIC AND WIND ANALYSIS

Seismic

Typically, 5% damping has been assumed for seismic design of most major cable-stayed bridges including the 465 m Alex Fraser and 340 m Skytrain Bridges in Vancouver, and the 890 m Tatara Bridge in Japan (all are main span dimensions). One notable exception is the 485 m Higashi-Kobe Bridge in Kobe. For this bridge

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seismic design was based upon a 2% spectrum, with additional checks being done for 1% (Narita and Yokoyama, 1991). These values were selected because the bridge has welded steel towers and the girder is carried entirely by the cables, with no bearings at the towers (Kitazawa, 1999).

Wind

The damping values used for wind design of cable-stayed bridges are distinctly different from the commonly assumed 5% for seismic design. In Japan, the Wind Resistant Design Manual for Highway Bridges, as summarized by Narita and Yokoyama (1991), suggests damping should be considered in the range of 0.25 - 0.5%. Scanlan and Jones (1990) have generally assumed 1% structural damping in all modes for aeroelastic analysis of cable-stayed bridges.

Comparison

The difference in choice of damping for seismic and wind analysis is usually attributed to several factors. First, the dominant frequency range of earthquakes is higher than that of wind. This observation is associated with a belief that increased damping is associated with higher modes of vibration (such as those excited by earthquakes). Second, the primary aeroelastic modes of response to wind vibration are bending and torsion, whereas in earthquake response longitudinal and transverse modes often play a significant role. In this situation, horizontal modes of vibration can lead to dissipation of structural energy into the ground, subsequently increasing the effective damping of the structure. Third, in seismic design some parts of the structure may be allowed to undergo limited plastic deformations that provide additional energy dissipation. Wind design, on the other hand, is based upon elastic response with no consideration of yielding of structural elements.

DAMPING EVALUATED FROM MEASURED RESPONSES FO CABLE-STAYED BRIDGES

Ambient vibration surveys

Ambient vibration surveys (avs), designed to measure small amplitude responses to ambient (typically traffic and wind) excitations, have been performed on a number of bridges. Although ambient vibration measurements have well-known limitations that can seriously affect an accurate determination of damping, they are nonetheless one of the few sources of actual damping data. Measurements on the Tampico Bridge in Mexico, and on the I-295 James River Bridge, concluded that nearly all modes exhibited damping less than 1%. Testing on the Quincy Bayview Bridge indicated damping values to be generally less than 2% (Atkins, 1998). Aside from well-known problems of measurement error, one unresolved limitation of avs results are the extent to which they are applicable for higher amplitudes of motion during a moderate to large earthquake.

Forced vibration testing

Forced vibration tests have been conducted on a number of cable-stayed bridges, including Alex Fraser, Shipshaw, Alamillo and Tjorn Bridges. All of these tests indicated a generally low level of damping, with the majority of results giving values less than 1.5%, and with some results substantially below 1% (Atkins, 1998). Tests on Japanese cable-stayed bridges, reported by Narita and Yokoyama (1991), found damping values between 0.3% and 2%. This illustrates both the low levels and large spread in damping values.

Strong-motion earthquake records

Few cable-stayed bridges have been instrumented with strong-motion recording systems. The most complete data comes from two instrumented bridges in Japan - the Suigo Bridge in Chiba Prefecture (near Tokyo) and the Higashi-Kobe Bridge. This study has used the records from the Suigo Bridge to make estimates of damping during earthquake response. The Suigo records and complete structural details were readily available when the study was started.

THE SUIGO BRIDGE AND STRONG MOTION RECORDS

The 290 m Suigo Bridge is illustrated in Figure 1. The bridge has a two span continuous cable-stayed part and approach sections. The superstructure is a steel box girder connected both longitudinally and transversely to the single steel tower. Three cables connect to the centre of the girder in a single plane on each side of the tower.

Bearings at each end of the cable-stayed part allow free translation of the deck in the longitudinal direction but provide restraint in the transverse direction. This study considers response of only the cable-stayed part. Locations of strong motion instrumentation in the longitudinal and transverse directions are marked by the six circled locations A1, etc., on Figure 1. The strong motion records used for this study were from the East Chiba-ken earthquake of December 17, 1987. The free-field (A6) peak ground acceleration (pga) was 1.14 m/s2 (transverse) and 1m/s2 (longitudinal). At the top of the tower (A1) the pga was recorded as 10m/s2 (transverse) and 4.46 m/s2 (longitudinal), both rather large structural responses. These represent acceleration amplifications of 8.8 (transverse) and 4.5 (longitudinal). At the centre of the longer span (A5) the pga was 3.63 m/s2 (transverse) and 2.47 m/s2 (longitudinal).

ANALYSIS OF SEISMIC RESPONSE OF SUIGO BRIDGE

The approach used to estimate the damping was to compare the actual strong motion records to structural responses computed using a calibrated finite element model of the bridge, with records from A6 (Figure 2) used as inputs (Atkins, 1998). The finite element model was constructed using the SAP2000 program, based on structural information provided by Kawashima et al., (1991). Damping was assumed to be viscous and constant in all modes. The finite element model was subjected to the longitudinal and transverse components of the ground motions recorded at A6 (free-field). Responses were computed at the top of the tower and at the centre of the longer span (corresponding to instrument locations A1 and A5 on Figure 1, respectively) for damping values of 0.5, 1, 2, and 5%. Foundation flexibility effects were included through the use of foundation springs, selected so that the frequency response characteristics of the model closely matched the Fourier spectral characteristics of the recorded earthquake responses. Summary details are provided in Table 1. No allowance was made for foundation damping for reasons that will become apparent later. The dynamic characteristics of this model were in good agreement with forced vibration tests conducted by Japanese engineers (Kawashima et al., 1991). Although twenty modes were included in the seismic analyses conducted in this study, most of the responses in each of the longitudinal and transverse directions resulted from participation of only a few modes. The modal participation factors associated with each of the main modes are indicated in Table 1. This is in contrast to the greater number of modes that must be used for many other larger and more complex cable-stayed bridges. The Suigo Bridge's relative simplicity in structural form, and the fact that responses in the longitudinal and transverse directions are largely uncoupled made it an ideal structure for this type of study.

Table 1 Dynamic characteristics of the Suigo Bridge obtained by finite element analysis and by measurement

Measurement Location	Finite Element Model Frequency	Modal Participation Factor	Measured Bridge Frequency (Hz)
Longitudinal: A1	1.50	99	1.55
A5	1.50		1.55
Transverse: A1	0.75	25	0.73
A5	1.03	68	1.35

Locations: A1=top of tower; A5=centre of longer girder span

RESULTS

Figure 3 shows, as an example, time history responses computed at the top of the tower (A1) for various damping ratios. The model responses for the various damping values were compared to the actual acceleration records in terms of their maximum response, duration of strong motion, and envelope of the acceleration record. The best matches of the model responses to the actual bridge responses (as determined by visually matching the time histories) at locations A1 and A5 are shown in Figure 4. In each case separate matches were done for the 0-20 sec, and 20-40 sec segments of each record, and in some cases different 'best match' damping ratios were identified for these two time segments. These are summarized in Table 2. Although some refinement could possibly be made in the best match estimates by computing responses for more closely spaced damping values, it seemed that this was really not warranted. This is illustrated in the later section on comparisons of response spectrum values.

Table 2 Best match estimates of the damping ratio for the Suigo Bridge

Measurement Location	Finite Element Model* [0-20 sec]	Finite Element Model* [20-40 sec]	Japanese Analysis**
A1 - longitudinal	0.5 %	2 %	2 %
A5 - longitudinal	2 %	5 %	5 %
A1 - transverse	1 %	1 %	0 %-1 %
A5 - transverse	1 %	2 %	5 %

* this study; ** Kawashima et al., (1991)

DUSCUSSION OF RESULTS

The time histories in Figure 3 for the responses at the top of the tower for the various damping ratios clearly show the strong influence of damping on the amplitude of structural response. The stronger ground motion occurred in the 0-20 sec segment, as shown in Figure 2. For computed responses within this interval the ratio between maximum response amplitude for 0.5% and 5% damping was approximately 2 for the longitudinal direction and 1.5 for the transverse direction (responses are shown here only for A1 location).

Figure 4 and Table 2 show that for the 0-20 sec segment, which contains the strongest input ground motion, best match damping values were identified between 0.5% and 2%, values well-below those commonly assumed in design. Because of the nature of this analysis, these damping estimates inherently include any damping that might be associated with the soil and foundations. One of the causes of such low damping may well be the continuous and integral nature of this all-steel bridge that offers very little in the form of energy dissipating mechanisms.

In three of four cases (Figures 4a, b, d) higher damping values were identified in the 20-40 second segment after the strongest ground motion was over. However, in two of these (Figures 4a,d) the best match damping was found to increase to only 2%, and in one case (Figure 4c) the best match damping was 1% over the entire record. In only one case (Figure 4b; location A5 longitudinal) was damping identified as high as 5%. These higher damping values in the later part of the records are believed to be an artifact of the analysis resulting from the presence of feedback of structural response in the recorded motion at A6, rather than an actual increase in system damping late in the response. This can be discerned in the A6 motions in Figure 2 after 20 seconds. Using this as input to the finite element model would require a higher damping value be used to suppress response of the system to match the observed amplitudes of actual response.

Of interest is an earlier study of the damping for the Suigo Bridge performed by Kawashima et al., (1991). Table 2 includes the results from that study. Although that study had a similar approach to the present one, there are some significant differences. The Japanese results indicate a best match damping of 5% for the girder in both the longitudinal and transverse directions, and between 0% and 2% for motions measured at the top of the towers. One of the principal reasons for the differences is believed to be the use of A3 motions as input. Since A3 is on the base of the tower the recorded motions at this location already contain some of the structural responses in the form of feedback (as evident in Fourier spectra of A3, not shown here). Damping estimates obtained using A3 motions as inputs are therefore expected to be higher than those found in this study using A6 free field input. It would seem that using A6 is a more reasonable choice for the ground motion.

The very limited results presented here seem to support the notion of low damping in cable-stayed bridges that is expressed in many of the literature studies. Additional analyses of these records, and those of the Higashi-Kobe Bridge, could be used to obtain additional information on the damping characteristics of all-steel cable-stayed bridges. The implications of low damping values on seismic response of cable-stayed bridges is examined in the following section.

COMPARISIONS OF STRUCTURAL RESPONSES AT LOW DAMPING

Computed seismic responses of four cable-stayed bridges of various designs were examined for low damping ratios. One of these was the Suigo Bridge, and finite element models of the other three, referred to here as Bridges 2-4, are shown in Figure 5. Each bridge was subjected to 3-D response spectrum analyses for 0.5%, 1%, 2% and 5% damping, calculated from an ensemble average of 15 time history records containing significant components in the longer period range. Moments and shears at the base of the towers for the two horizontal directions (a total of four quantities) were computed for each spectral damping. The results of these comparisons are summarized in Table 3 where responses at 0.5, 1 and 2% damping are expressed as ratios of the response at 5%. The range of amplification ratios (shown in parentheses) for the four response quantities is generally small. Considering the ratios for all four bridges gives approximate average response amplifications of 1.3 for 2% damping, 1.5 for 1%, and 1.7 for 0.5%. These values provide a direct indication of the higher average responses that can occur at these lower damping values, and hence the potential consequences associated with a possibly unconservative choice of 5%. More detailed discussions on these analyses are provided by Atkins (1998).

Table 3 Mean ratios of responses (for moments and shears at the base of the towers) for low damping values; referenced to responses at 5% damping

Bridge		Damping	
Suigo	1.72	1.53	1.32
Bridge 2	1.90	1.64	1.34
Bridge 3	1.64	1.45	1.24
Bridge 4	1.47	1.35	1.21

(Values in parentheses show range of ratios)

CONCLUSIONS

Evidence suggests that use of 5% damping for seismic response analysis of cable-stayed bridges may not be an appropriate, nor conservative assumption. Vibration tests on many cable-stayed bridges over the past decade or so have alluded to the possibility of this situation, and most arguments suggest that the low damping observed in these tests is associated with the low amplitudes of motion. At higher response levels occurring during earthquakes the damping would be expected to be larger. Although the data is very limited, the strong motion records from the Suigo Bridge show that low damping can exist during earthquake motions even when peak structural response was 1.0 g. Damping values in the range of 1% - 2%, and as low as 0.5% have been inferred from this data. A response spectrum study of four cable-stayed bridges of representing various designs indicates illustrates the implication of low damping. Average response amplification ratios (compared to responses for 5% damping) for shear force and moment at the base of the towers of up to 1.7 were found when damping was as low as 0.5%, with individual response quantity amplifications being as large as 2.1. For 2% damping the average response amplification was 1.3, and for 1% damping it was 1.5. These results also indicate that, for a particular bridge, there is a need to conduct response sensitivity analyses to examine the impact of various damping assumptions on the computed seismic response.

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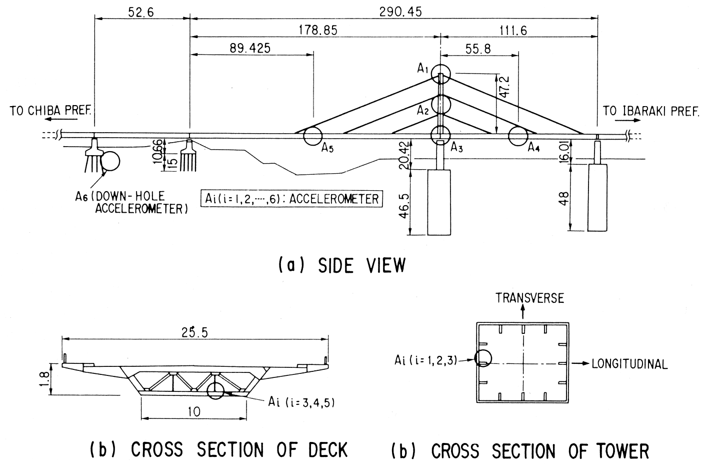


Figure 1 The Suigo Bridge showing strong motion instrument locations A1 to A6 (Kawashima et al; 1991)

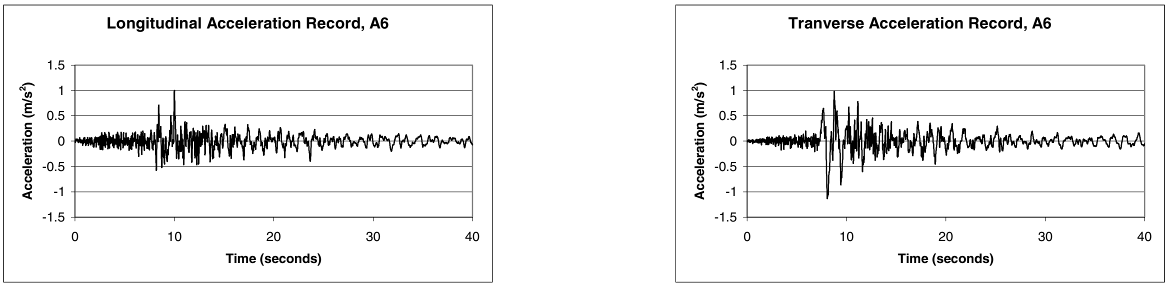
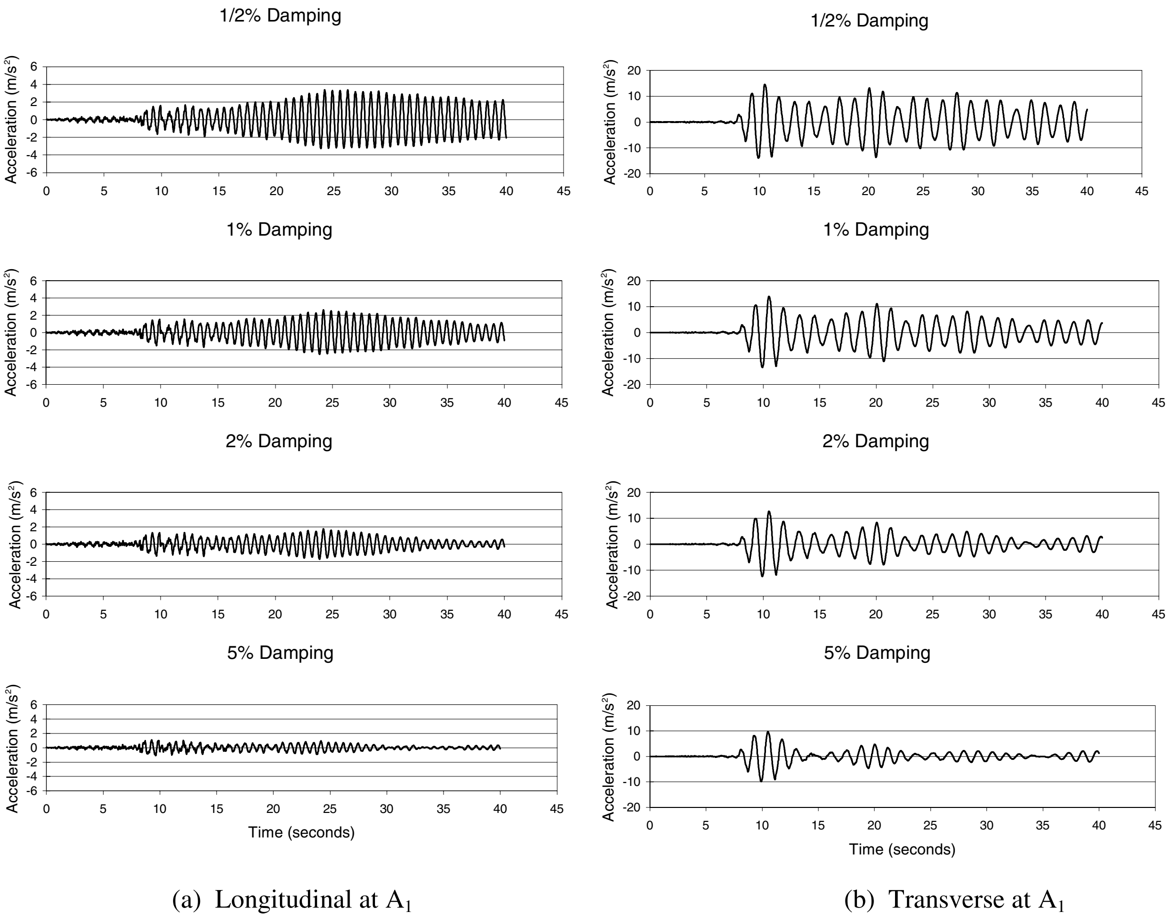
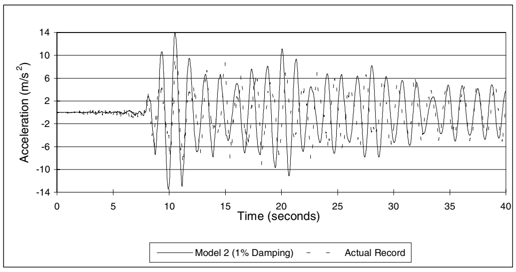


Figure 2 Ground accelerations recorded by down-hole accelerometer at location A_6

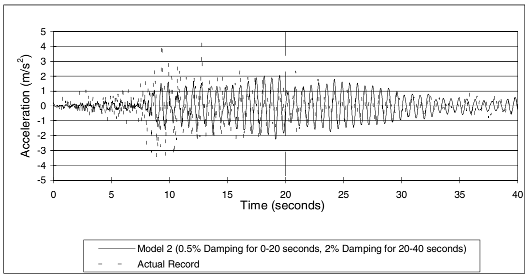


(a) Longitudinal at A_1 (b) Transverse at A_1

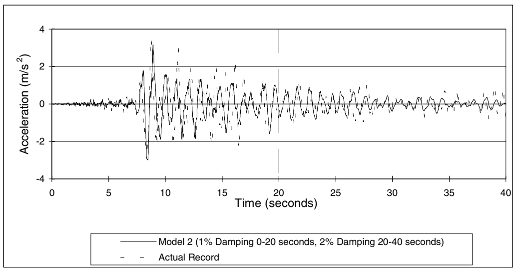
Figure 3 Response accelerations computed at the top of the tower (A_1) for 0.5, 1, 2 and 5% damping when subjected to A_6 motions as input

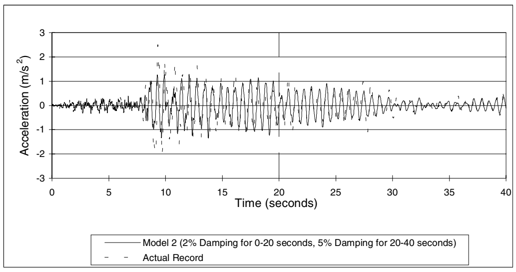


(a) Longitudinal direction at top of tower (A_1)



(c) Transverse direction at top of tower (A_1)





(b) Longitudinal direction at centre of longer span (A_5) (d) Transverse direction at centre of longer span (A_5)

Figure 4 Comparisons of accelerations from recorded (actual) responses and best matched damped responses from finite element model for Suigo Bridge

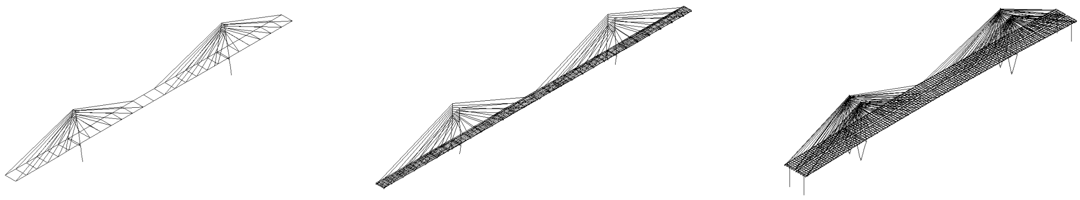


Figure 5 Three additional cable-stayed bridges used in response spectrum study (centre span length and fundamental period are shown for each)