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Damping In Seismic Response of Cable-Stayed Bridges

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ABSTRACT

Seismic design and analysis of most major cable-stayed bridges typically assumes 5% damping, and response spectra values are set based upon this value. However, there appears to be little empirical basis for use of this value for cable-stayed bridges, other than the fact that it is the "traditional" number to use. Even if more sophisticated procedures (e.g., time history) are used in subsequent analyses, 5% damping is often still the target value. This paper presents results obtained from examination of strong motion records from the Suigo Bridge, 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 a finite element model it is concluded that the Suigo Bridge exhibited quite 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 indicate that the common assumption of 5% damping may, in certain cases, be too high (unconservative).

INTRODUCTION

Damping is one structural parameter having relatively large uncertainty, and it is one that cannot be assessed analytically. Traditionally 5% damping is assumed for seismic analysis of most buildings and bridges. For bridges however, and especially for cable-stayed bridges, this may not be entirely 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 mechanisms.

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 data sets available world-wide on full-scale earthquake response of a cable-supported bridge.

DAMPING USED FOR SEISMIC AND WIND ANALYSES

Typically, 5% damping has been assumed for *seismic* design of most major cable-stayed bridges. Most recent major 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), have all followed this approach. The notable exception however, is the 485 m Higashi-Kobe Bridge in Kobe, Japan where seismic design was based upon a 2% spectrum. In this case, the 2% damping was used because the bridge has welded steel towers and the girder is carried entirely by the cables, with no bearings being used at the towers (Kitazawa, 1999). Additional seismic analyses on this bridge were also conducted for 1% damping (Narita and Yokoyama, 1991).

The damping values assumed in *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), states that the damping for wind design 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.

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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 an additional energy dissipation mechanism. Wind design, on the other hand, is based upon elastic response with no consideration of yielding of structural elements.

DAMPING EVALUATED FROM MEASURED RESPONSES OF CABLE-STAYED BRIDGES

Damping from ambient vibration surveys

Information on actual damping in cable-stayed bridges is rather sparse. 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.

Damping from 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.

Damping from 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, oriented in the longitudinal and transverse directions are marked by the six circled locations A_1, 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 (A_6) peak ground acceleration (pga) was 1.14 m/s^2 (transverse) and 1m/s^2 (longitudinal). At the top of the tower (A_1) the pga was recorded as 10m/s^2 (transverse) and 4.46 m/s^2 (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 (A_5) the pga was 3.63 m/s^2 (transverse) and 2.47 m/s^2 (longitudinal).

MODELLING AND METHODS OF RESPONSE ANALYSIS

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 A_6 (Figure 2) used as inputs. 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 A_6 (free-field). Time history responses were computed at the top of the tower and at the centre of the longer span (corresponding to instrument locations A_1 and A_5 on Figure 1, respectively) for damping values of 0.5, 1, 2, and 5%. Complete details of the time history responses are given in Atkins (1998). 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 agreement with forced vibration tests conducted by Japanese engineers (Kawashima, 1991). For seismic analysis, 20 modes were used, providing more than 90% mass participation in the horizontal directions and more than 80% vertically. However, the principal response in each of the longitudinal and transverse directions resulted from participation of a single mode in each direction, as indicated in Table 1. This is in contrast to the much larger number of modes that must be used for many other larger 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 Principal Frequencies of Model and Recorded Seismic Response of the Suigo Bridge

Measurement Location	**Model Frequency (Hz)**	**Suigo Bridge Frequency (Hz)**
A_1 - longitudinal	1.50	1.55
A_5 - longitudinal	1.50	1.55
A_1 - transverse	0.75	0.73
A_5 - transverse	1.03	1.35

A_1=top of tower; A_5=centre of longer girder span

RESULTS

Figure 3 shows, as an example, time history responses computed at the top of the tower (A_1) 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 the decay of the acceleration record. The best matches of the model responses to the actual bridge response (as determined by visually matching the time histories) at locations A_1 and A_5 are shown in Figure 4. Separate matches with different damping values were done for the 0-20 sec, and 20-40 sec segments of each record. These are summarized in Table 2.

Table 2 Best Match Damping Estimates for the Suigo Bridge

Measurement Location	*Finite Element Model* [0-20 sec]*	*Finite Element Model* [20-40 sec]*	*Japanese Analysis***
A_1 - longitudinal	0.5 %	2 %	2 %
A_5 - longitudinal	2 %	5 %	5 %
A_1 - transverse	1 %	1 %	0 %-1 %
A_5 - transverse	1 %	2 %	5 %

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

DISCUSSION

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 (the girder responses at A_5 are not shown here due to limitations on space). 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 A_5 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 A_6, rather than an actual increase in system damping late in the response. This can be discerned in the A_6 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 also summarizes the results from that study. Although that study had a similar approach to the present one, the results show some differences. These results indicate 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 in the use of A_3 motions as input. Since A_3 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 A_3, not shown here). Damping estimates obtained using A_3 motions as inputs are therefore expected to be higher than those found in this study using A_6 free field input. It would seem that using A_6 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 such low damping values on seismic response of the bridge, and comparisons with responses at 5% damping, needs investigation.

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. The strong motion data from the Suigo Bridge examined in this study indicates that damping has been very low during earthquake motions even when peak structural response was 1.0 g. Damping values of 2% and lower have been inferred from this data. The implication of this is that actual bridge responses will be greater than would be predicted from 5% design damping.

ACKNOWLEDGEMENT

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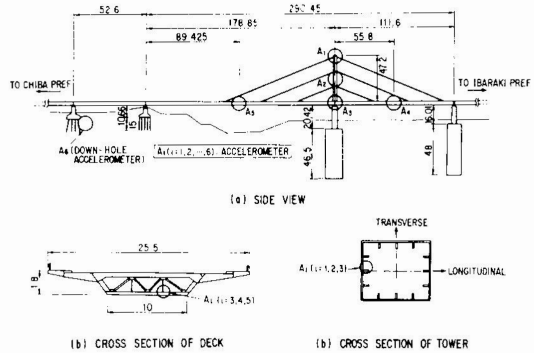
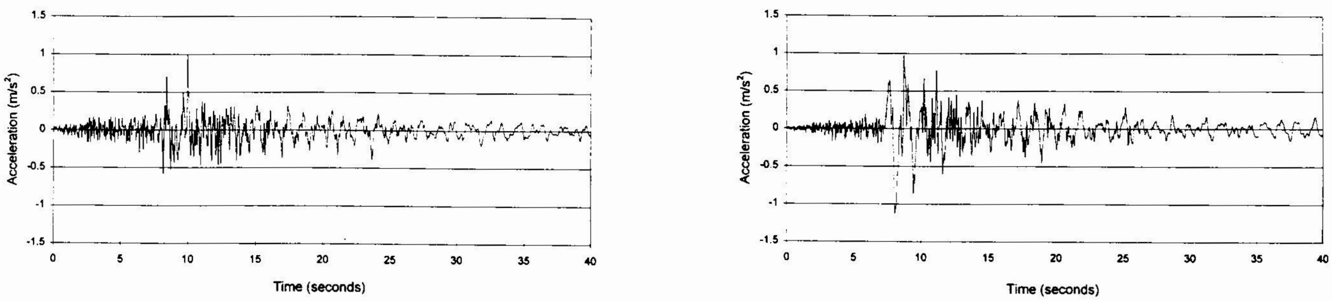


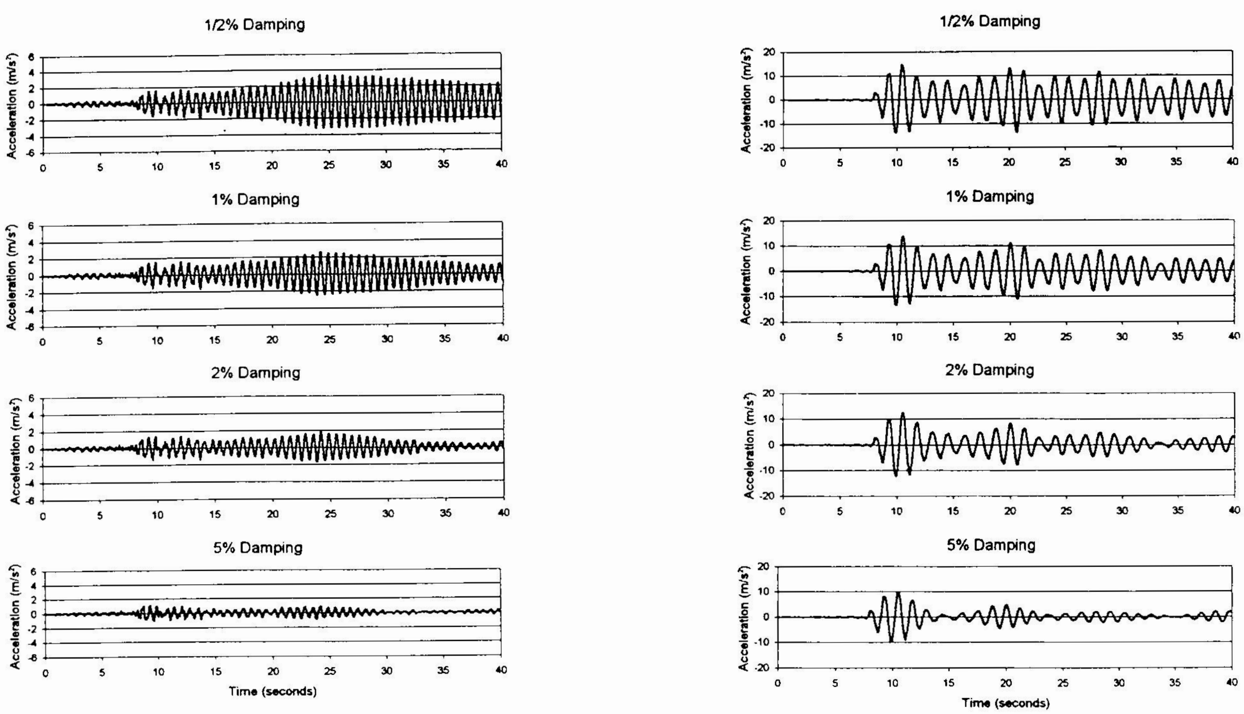
Figure 1 The Suigo Bridge showing instrument locations A_1 to A_6 (Kawashima et al; 1991)



(a) Longitudinal

(b) Transverse

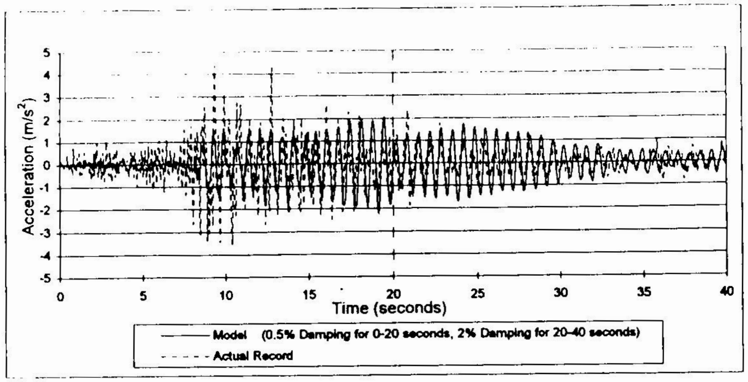
Figure 2 Ground accelerations recorded by down-hole accelerometer at location A_6



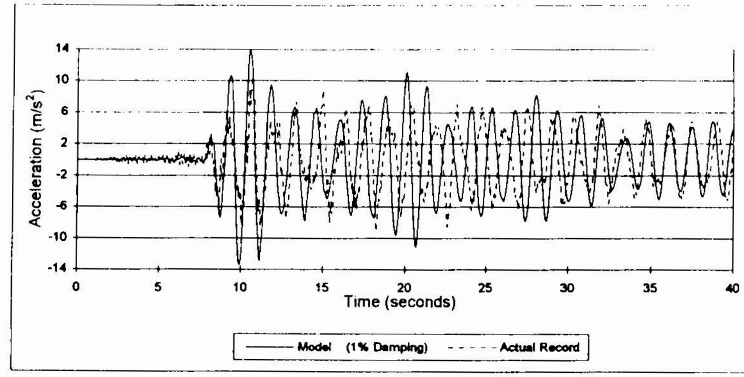
(a) Longitudinal A_1

(b) Transverse A_1

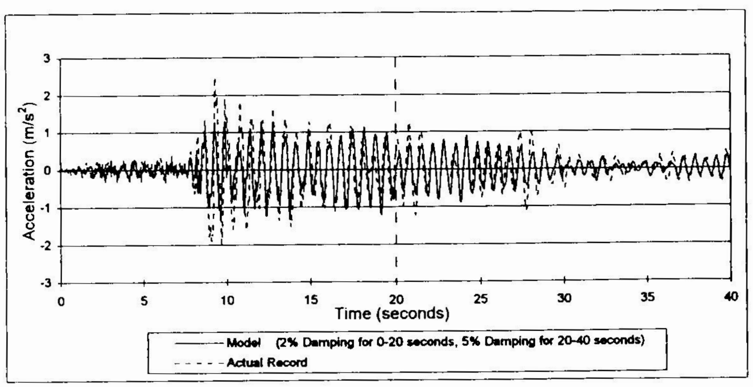
Figure 3 Response accelerations computed at the top of the tower (A_1) for 0.5, 1, 2 and 5% damping



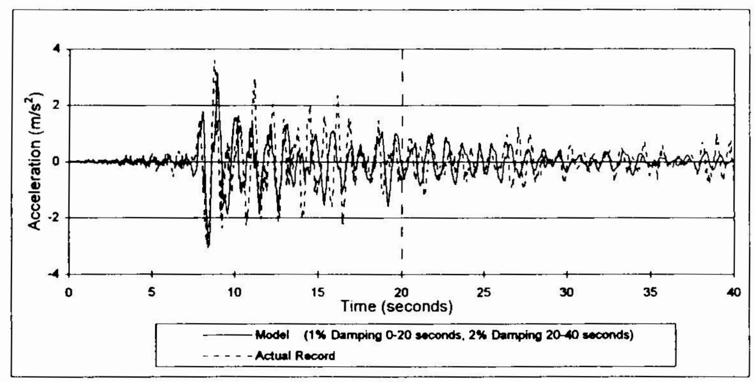
(a) Top of tower (A_1): Longitudinal



(c) Top of tower (A_1): Transverse



(b) Centre of longer span (A_5): Longitudinal



(d) Centre of longer span (A_5): Transverse

Figure 4 Comparisons of recorded (actual) acceleration responses and best matched damped responses of finite element model