Michael H. Faber, Ton Vrouwenvelder,
Konrad Zilch (Hrsg.)

Aspects of Structural Reliability

In Honor of R. Rackwitz

Herbert Utz Verlag · München

Architektur und Bauwesen

Mitherausgegeben und gefördert vom
Förderverein Massivbau der TU München e. V.

Bibliografische Information der Deutschen Nationalbibliothek: Die Deutsche Nationalbibliothek verzeichnet diese Publikation in der Deutschen Nationalbibliografie; detaillierte bibliografische Daten sind im Internet über http://dnb.d-nb.de abrufbar.

Dieses Werk ist urheberrechtlich geschützt.
Die dadurch begründeten Rechte, insbesondere die der Übersetzung, des Nachdrucks, der Entnahme von Abbildungen, der Wiedergabe auf fotomechanischem oder ähnlichem Wege und der Speicherung in Datenverarbeitungsanlagen bleiben – auch bei nur auszugsweiser Verwendung – vorbehalten.

Copyright © Herbert Utz Verlag GmbH · 2007

ISBN 978-3-8316-0752-5

Printed in Germany

Herbert Utz Verlag GmbH, München
089-277791-00 · www.utz.de

Contents

Aspects of Structural Reliability – Faber, Vrouwenvelder, Zilch (Hrsg.)
©2007 Herbert Utz Verlag München, ISBN 978-3-8316-0752-5

Design bases vs. expected performance for long span suspension bridges

F. Casciati & L. Faravelli
Department of Structural Mechanics, University of Pavia, via Ferrata 1, 27100 Pavia, Italy

ABSTRACT: In a sort of dialog between a designer and an expert in structural reliability, the theme is to provide the bases of design for a long-span suspension bridge. Nevertheless, the discussion applies to many large civil engineering structural systems. Five main aspects are emphasized: action definition, sub-structuring, interactions, robustness and monitoring.

1 INTRODUCTION

Speculating by dialogs was an attitude at the age of the ancient Greek philosophers, as well as it was nearly the rule at the embryo of modern Mechanics (Galileo, 1630). Today, that style of arguing in scientific communications is lost. But the reader can regard this paper as the product of dialogs between two characters. The first is engaged in the formulation of modern bases for the design of a long-span suspension bridge. The second character serves as the knowledge basis of the most recent developments in structural reliability theory and probabilistic risk assessment.

The pioneering paper by Freudenthal (1956) in this area is dated fifty years ago, but only twenty years later the solution of the inherent computational problems was successful approached (Rackwitz & Fiessler, 1978; Olivi, 1980). In the Eighties a bundle of books were published on the topic (Ditlevsen, 1981; Ang & Tang, 1984; Augusti et al., 1984; Madsen et al., 1986; Melchers, 1987; Casciati & Faravelli, 1991; Ditlevsen & Madsen, 1996; Casciati & Roberts, 1996) and the Nineties generated the concept of "performance based design" (Fujitani et al., 2005) and the associated probabilistic model code format (JCSS, 2001).

Within this framework one is expecting that only minor obstacles are is met when the bases for the design of a long span suspension bridge are demanded (Gimsing, 1997; Calzona, 2005). The design bases of bridges of recent realization (Wong, 2003; Kitagawa 2003) are considered as a starting point. But the way is not straightforward. Five aspects will be discussed in detail:

1) the definition of time variant actions;
2) the ability to sub-structure the complex structural problem;
3) the interactions of the structural system with the foundation and the environment;
4) the way to manage accidental actions and related robustness issues;
5) the interconnection between structural reliability and structural monitoring.

This paper does not provide definite answers. It just underlines the problems and shows feasible solutions, if any. But the impression is that several points still require a better formulation of the problem, a more rigorous treatment and a final systematization of the pertinent theoretical background.

2 THE STRUCTURAL LIFETIME

The Brooklyn suspension bridge celebrated its centenary and it is likely it will celebrate several additional decades. No matter, therefore, that in designing the Stonecutters (HKHD, 2001) cable-stayed bridge in Hong Kong, the responsible authority was demanding an economic lifetime L of 120 years. Assume now that the authority X be involved in the design of a new suspension bridge and its span be one time and half longer than the presently longest suspension bridge in the world, the Akashi Kaikyo bridge (Kitagawa, 2003). It is likely that the authority X will ask for an economical lifetime of L = 200 years!

Immediate consequences of the selection of such a number are found both in the definition of the actions and in the modeling of the resistance.

2.1 *Defining the time variant actions*

The Stonecutters authority introduced the action "earthquake" through design spectra of assigned return periods, but different for the serviceability limit states and the ultimate limit states. The latter value was 2400 years, or simply the upper-fractile 5% of the distribution of the maximum in 120 years. It is the application of standard structural reliability features. When the authority X applies the same reasoning to its bridge with L = 200 years, a return period of 4000 years is found.

Moreover, in view of that later will be referred to as robustness, the authority X also needs to define action levels more devastating than those associated with the ultimate limit states in view of checking the "structural integrity" after catastrophic events. Extending the above reasoning, one fixes the 2% upper-fractile, thus achieving a return period of 10000 years (it was 6000 years for the Stonecutters case). Such a way, could be easily extended to the definition of other time variant actions as wind, snow, temperature and so on.

A structural engineers will not be disturbed by these digits. The collection of wind time histories started as a need for the aeronautic traffic and their records date back to the Fifties. The collection of accelerometric data comes after the introduction of the measurement instrument (Housner, 1947) and existing databases date back to the Sixties or Seventies. The structural engineer, therefore, does not rely on statistics, but rather on theoretical models statistically built on the available data.

But, consulting an expert in hydrology, whose data date back to the 19th century, the following two main objections arise:

1) on the basis of the available statistics, one can express confident estimates on return periods not longer than 300 years;
2) the basic assumption is that nature repeats its behavior without modifications; but it is well known that the period of nature cycles is just of some centuries.

Terminology must therefore be corrected in order to avoid misunderstandings. Since the realization of a large infrastructure cannot be preceded by an extremely long period of data collection, the available statistics are converted into probabilistic models on the basis of which the design values of the time variant actions are selected. These values can be derived as the ones with return periods of millenniums, but in this case they have to be regarded as conventional values and their specification in terms of return period should be completed by an attribute (f.i., conventional) clarifying their nature.

Unfortunately, a simple rule which sees the conventional return period values, to be used for the different limit states, as generated by the desired extension of the lifetime does not find unanimous consensus. A wind engineer, in fact, is anchored to the definition of the reference wind velocity with explicit return period of 50 years (JCSC, 2001; Carassale & Solari, 2005). This value can be increased, when necessary, to the lifetime period (HKHD, 1990). From this reference value, then, higher design values are achieved by introducing factors higher than 1, avoiding their interpretation in terms of return period. A consistent general definition of all time variant actions would certainly help, mainly because safety factors of uncertain source could be definitively replaced by statistics when available. On the other side, compromises between different scientific communities (as the ones of wind and earthquake engineering) do not result straightforward.

One should also distinguish between statistics collected in situ and information derived from a database. A long span bridge connecting two sea coasts, when realized, will see its deck at 65 m on the sea level, in order to allow the naval traffic below it. Moreover, the mid-span will be more than 1 km away from each of the two coasts. There is no way to measure the wind speed at the deck mid-span before the construction would have been completed. Numerical models are used to reconstruct the wind phenomenon at that point and along the deck in general, but their accuracy could be afflicted by some forgotten (or simply unexpected) local phenomena. In particular the existence of privileged wind directions could seriously affect the final design specifications. An adequate design, therefore, should ask for measurement during constructions (which often takes several years) and mainly during the first decade of the bridge utilization, in order to update the load models and, hence, the estimated reliability. A scenario including the bridge retrofitting should also be listed.

A further aspect must be emphasized: natural phenomena are rarely linear. It is difficult that a natural action can simply be scaled by its intensity to simulate less and less frequent events. In the seismic event case, given a site, the first data elaboration aims at establishing a link between peak ground acceleration (PGA) and return period: the model of seismo-genetic zones hold up to return periods of 1000 or 2000 years, and then by mere extrapolation one moves to longer periods. But the goal is the definition of time histories of the seismic action in terms of velocity, displacement and acceleration; they will serve as input to the dynamic transient analyses the design requires. For a suspension bridge, the motion intensity is not the only parameter of interest. One must watch relative difference of the motion at the towers, and between these and the anchorages, as well as differential amplifications due to the different mechanical properties of the foundation rocks and the motion (horizontal vs. vertical) incidence. The situation of Figure 1 cannot be a priori excluded. To impose simultaneous time histories at the four supporting areas is unacceptable.

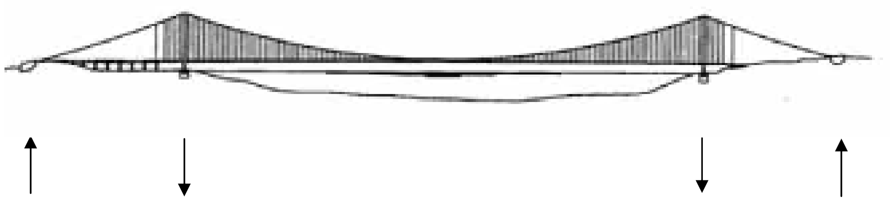

Figure 1 – Possible negative seismic excitation scenario.

The seismic motion must therefore be re-constructed starting from the actual knowledge on geology, morphology and seismology at the site. The characteristics and the magnitude of the event depend on the properties of the seismic source, on the seismic waves pattern between source and site and on the features of the more superficial soil layers These three aspects are the basis of the simulation of design time histories. Historical catalogs and instrumental records support the identification of source zones and associated magnitude of the strongest expected events in the area. For each of these areas the relevant return period is estimated. The signals of the three acceleration components are than propagated from the source to the four bridge support areas. It is worth noting that such an approach is respectful of the physical properties of the seismic events:

a. the frequency content depends on the magnitude and hence, for the same source, on the return period;
b. the motion intensity varies in the four support areas depending on the provenience azimuth;
c. the provenience direction establishes the delay at the four support areas;
d. geometry and nature of the most superficial soil layers alter the intensity of the signals and the relative delays.

2.2 *Defining the resistance features*

The Brooklyn bridge suspension cables are giving its supervising authority a sound expertise on their durability (Yanev, 2005). But they were produced with the technology of more than one century ago! This expertise cannot be exported to different bridges. The enemy from which one has to defend is the corrosion, and still reliable models of its progress are lacking. The Akashi designers adopted an advanced de-humidification process to extend the cable life and a special painting (coating) to protect the steel structural components. Estimates of the reliability one can achieve by adopting these technologies are presently lacking.

In such a protective frame, assuming deteriorating parts and devices are replaced periodically, an extended lifetime could only find obstacle in the resistance to fatigue. Usual loading spectra are helpless since the 10 million of cycles will be likely achieved within a 200 years lifetime. The strength of the material undergoing so many cycles must be limited to its asymptotic lower bound detected by fatigue tests.

3 SUB-STRUCTURING

A suspension bridge is a structural systems supported in four different areas: the foundations of the two towers and the two cable anchorages. An accurate numerical models would see (Figure 2) four foundation soil volumes, two pylons, two or more cables running from anchorage to anchorage and supported on the top of the pylons, several hangers supporting the deck transversal beams on which the longitudinal beams insist. The structural analysis software runs on standard personal computers, but every transient dynamic analysis could required days of elaboration. This macroscopic model comes together with mesoscopic and microscopic model, which allow the designer the study of the details and their specification (Figure 2).

As observed in (Faravelli & Bigi, 1990), a structural analysis under uncertainty cannot rely on usual simplification assumptions as those based on symmetry: the geometrical symmetry is easily realized, but the symmetry of the realization of a random field is certainly a strong questionable assumption. In (Faravelli, 1989; Breitung & Faravelli, 1996), it was theorized that a response model depending on N variables can be expressed in a reduced space of size n by the form:

$$g(\boldsymbol{x}, \boldsymbol{y}) = F(\boldsymbol{x}) + \varepsilon(\boldsymbol{x}, \boldsymbol{y}) \qquad (1)$$

where the whole model is defined in the space of size N of the variables $\boldsymbol{x}$ and $\boldsymbol{y}$, the model $F(\,)$ in the n size space of the variables $\boldsymbol{x}$ and $\varepsilon(\boldsymbol{x}, \boldsymbol{y})$ takes into account both the pure error and the effect of the space reduction. The approach, however, is successful when the variability in the error term can be quantized in a few units per cent.