Baidar Bakht · Aftab Mufti

# Bridges

## Analysis, Design, Structural Health Monitoring, and Rehabilitation

*Second Edition*

EXTRAS ONLINE

Springer

Baidar Bakht • Aftab Mufti

# Bridges

## Analysis, Design, Structural Health Monitoring, and Rehabilitation

Second Edition

Springer

Baidar Bakht
Department of Civil Engineering
University of Manitoba
Winnipeg, MB, Canada

Aftab Mufti
Department of Civil Engineering
University of Manitoba
Winnipeg, MB, Canada

Additional material to this book can be downloaded from http://extras.springer.com
All photos are provided courtesy of JMBT Structures Research Inc.

ISBN 978-3-319-17842-4 ISBN 978-3-319-17843-1 (eBook)
DOI 10.1007/978-3-319-17843-1

Library of Congress Control Number: 2015951973

Springer Cham Heidelberg New York Dordrecht London


Printed on acid-free paper

Springer International Publishing AG Switzerland is part of Springer Science+Business Media (www.springer.com)

*This book is dedicated to the memory*
*of Leslie G. Jaeger CM, DSc, FRSE,*
*P.Eng. (1926–2013),*
*An outstanding teacher, a cherished research*
*colleague, a dear friend, and a mentor to*
*authors*

# Foreword

The authors, sometimes accompanied by Professor Leslie Jaeger (1926–2013), have given courses on bridge engineering in various countries of the world since early 1990s; these courses dealt with the aspects of bridge engineering in which the authors not only had a hands-on experience, but had also published technical papers. The course notes, initially comprising copies of published papers, progressed into a seminal document. The latest and most up-to-date version of this document is now in your hands in the form of a self-contained book.

It is the hope of the authors that both practicing bridge engineers and graduate students of structural engineering will find this book practical and useful. A number of computer programs, which can be downloaded from http://extras.springer.com, are expected to provide solutions to everyday problems facing bridge engineers. Some topics, such as arching in deck slabs, fibre reinforced polymers, and structural health monitoring, might not be covered by the design codes of many countries. Hopefully, the chapters on these topics that provide copious references and case histories will pave the way for their adoption in these countries.

Our outstanding graduate students and research associates have been acknowledged through the references cited in the books. We would also like to express our gratitude to the following persons, some of whom are not acknowledged through the references: Gamil Tadros for his friendship and collaboration on many engineering projects; Evangeline Murison for her help in many SHM projects; Mike Wilson of Atlantic Industries Limited, Canada, for his permission to use the details of the Whitehorse Creek soil-steel bridge. Also, we like to thank our various secretaries at Dalhousie University and University of Manitoba, who have assisted us in producing the book.

It is acknowledged that the CHBDC (2006) design provisions, described in Chaps. 8 and 9 were formulated by the Technical Subcommittee, which comprised B. Bakht, N. Banthia, B. Benmokrane, G. Desgagné, R. Eden, M.-A. Erki,

V. Karbhari, J. Kroman, D. Lai, A. Machida, A.A. Mufti, K. Neale, G. Tadros, B. Täljsten, with A.A. Mufti being the chair. The current chair of the CHBDC Technical Subcommittee for Fibre Reinforced Structures is S. Sheikh.

Winnipeg, MB, Canada
February 21, 2015

Baidar Bakht C.M.
Aftab Mufti C.M.

# Contents

# Chapter 1
# Loads and Codes

## 1.1 Introduction

Although not generally appreciated by lay people, it is not possible to design and construct a structure that will remain safe against failure under all conditions and at all times.

The several reasons for a structure being prone to failures include: (a) the strength of the various components of the structure cannot be assessed with full certainty; (b) the loads that a structure will be called upon to sustain also cannot be predicted with certainty; and (c) the condition of a structure may deteriorate with time due to the effects of the environment, causing it to lose strength. Because of these factors, there exists a probability that the strength of a structure will at some time be exceeded by the loads that it has to sustain, resulting in the failure of the structure. As noted in Sect. 1.3.2, the term failure is being used here not only to signify the collapse of the whole structure, but also to include the situation of the structure not being able to fulfil one or more of its intended functions.

The probability of failure of a structure can be reduced by increasing its design strength, which invariably leads to a higher first cost. The role of the structural engineer is to strike a socially acceptable balance between the risk of failure and the cost of the structure. For example, a bridge can indeed be built to have the same probability of failure as the pyramids of Giza, shown in Fig. 1.1. The cost of such a bridge, however, is likely to be so high that society may not be prepared to pay for it. By contrast, society may not be prepared to accept in a bridge the same high frequency of failure as in an automobile.

It is sometimes argued that a good engineer can strike a balance intuitively between the cost and safety of a structure, and that design codes tend to restrict the creative ability of the designer. The ideal criteria for structural design, it is argued, are those which merely require that a structure remain safe while fulfilling its intended functions. Examples of the world's most spectacular bridges, which



B. Bakht, A. Mufti, *Bridges*, DOI 10.1007/978-3-319-17843-1_1

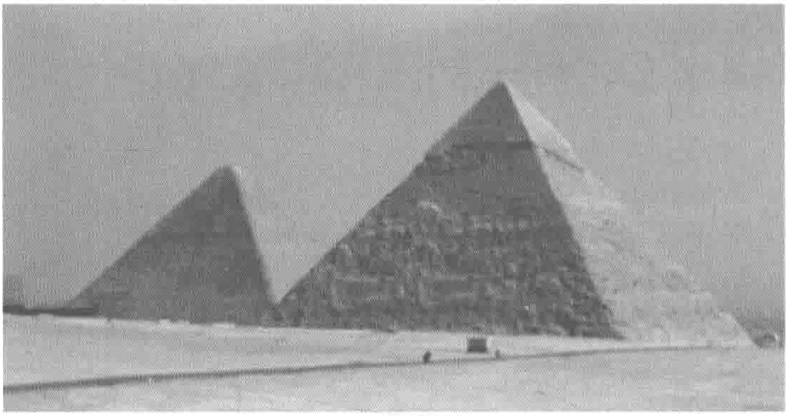

**Fig. 1.1** Pyramids of Giza in Egypt, examples of structures with low probability of failure

have very long spans and for which there existed no design codes until recently, are given in defence of the argument for having no design code at all.

It can be demonstrated readily that, due to the lack of a set of comprehensive design criteria, different structures designed by different designers are likely to have different probabilities of failure. This situation is particularly undesirable for bridges on the same roadway system. Since all such bridges are likely to be subjected to nearly the same maximum vehicle and environmental loads, the bridge with the highest probability of failure will govern the capacity of the road; in this case, it can be readily appreciated that the resources put into making the rest of the bridges extra-safe are not being expended wisely.

Since the designs of short and medium span highway bridges are governed mainly by vehicle weights, the design live loads constitute a very important part of the design criteria. It is surprising that little attempt is usually made to ensure a realistic correspondence between the actual vehicle weights in a jurisdiction and the design live loads for its bridges.

This chapter presents a method using which any number of vehicles can be compared with each other with respect to the maximum load effects they induce on bridges; this method can also be used to formulate one or more design vehicles corresponding to a given population of vehicles. The chapter also provides the basics of the probabilistic methods, which are used to quantify safety in modern design codes.

## 1.2 Vehicle Loads

Designs of most short and medium span highway bridges are governed predominantly by longitudinal moments and shears. The live load components of these responses are caused by heavy commercial vehicles and are governed by the spacing and weights of their axles. The task of quantifying the commercial vehicles

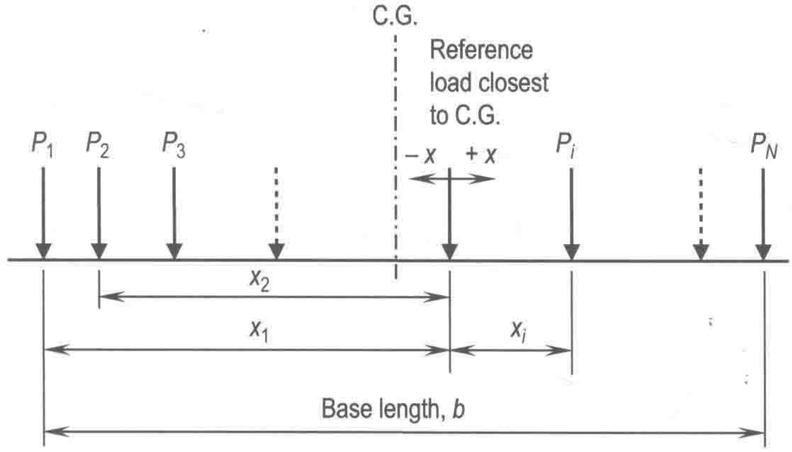


**Fig. 1.2** Notation for a series of point loads and their spacing

with respect to the load effects which they induce in bridges is made difficult by the very large number of axle weight and spacing combinations that are encountered in practice.

With the help of the method described in Sect. 1.2.1, a set of discrete loads can be reduced to an equivalent uniformly distributed load (UDL) which gives very closely the same maximum moments and shears in one-dimensional beams as the discrete loads. The equivalent UDL, as explained later, is useful in comparing the effects of different vehicles in all bridges.

### *1.2.1 Equivalent Base Length*

It has been shown by Csagoly and Dorton (1978) that $N$ discrete loads, with a total weight of $W$, on a beam can be replaced by a UDL which is also of total weight $W$, and has a length $B_m$ so that the moment envelope along the beam due to the UDL is very nearly the same to the moment envelope due to the set of discrete loads. The length $B_m$, which is referred to as the *equivalent base length*, is given by the following equation:

$$B_m = \frac{4}{W}\sum_{i=1}^{N} |P_i x_i| - \frac{2(N-1)}{bNW^2}\left\{\sum_{i=1}^{N}(P_i x_i)\right\}^2 \tag{1.1}$$

where $N$ is the total number of discrete loads and other notation is as illustrated in Fig. 1.2. The load closest to the centre of gravity of the set of loads is taken as the reference load and distances of other loads $x_i$, are measured with reference to this load.

It can be seen that Eq. (1.1) is independent of the span length of the beam; it gives only approximate values of $B_m$ which, as shown later, are accurate enough for most practical purposes. Equation (1.1) is adapted from the following more accurate expression which incorporates the span length, $L$, of the beam and which is reported by Jung and Witecki (1971).

$$B_m = \frac{4}{W}\sum_{i=1}^{N}|P_i x_i| - \frac{2}{LW^2}\left\{\sum_{i=1}^{N}(P_i x_i)\right\}^2 \quad (1.2)$$

#### 1.2.1.1 Accuracy

The percentage of error incurred in the determination of beam moments through the simplified approach of equivalent base length defined by Eq. (1.1) is denoted by $\Delta$ and quantified by:

$$\Delta = \left\{\left(\frac{M_B}{M}\right) - 1\right\} \times 100 \quad (1.3)$$

where $M_B$ is the maximum beam moment at a reference point due to the uniformly distributed load of length $B_m$ obtained by Eq. (1.1), and $M$ is the corresponding maximum moment due to the given set of discrete loads.

Values of $\Delta$ are plotted in Fig. 1.3a against span length for moments in simply supported beams due to a truck with five axles. It can be seen in this illustrative example that the degree of error is within +1 % and −8 % for all reference points considered. Values of $\Delta$ are large only where the magnitude of moment is small and hence the magnitude of $\Delta$ is irrelevant.

Although Eq. (1.1) was developed for moments in simply supported beams, it is also valid for shears and for continuous beams. In Fig. 1.3b, values of $\Delta$ are plotted against the span length of a two-span continuous beam for maximum moments at different points also due to a truck with five axles. It will be noted that the values of $\Delta$ are somewhat larger than those of their counterparts in the simply supported beam, but are still small, being within +1 % and −10 %. In both beams, $\Delta$ reduces with the increase in span length.

The actual envelope of maximum moments in a simply supported beam with a span of 10.67 m span to a five-axle truck is compared in Fig. 1.4 with the envelope of maximum moments due to the UDL of length $B_m$ obtained by Eq. (1.1). The closeness of the two envelopes is striking. The figure also shows the variation of $\Delta$ along the span. It can be seen that the values of $\Delta$ are very small in the middle half of the bridge, where moments are usually considered in design, being within ± 5 % in this region.

Near the supports, where the magnitudes of moments are small, $\Delta$ becomes as high as about −13 % but is not of concern in design.

It has been shown by Csagoly and Dorton (1973) that the value of $\Delta$ increases as the number of discrete loads is reduced. However, even for a set of three