Engineering Structures 97 (2015) 29–46

Contents lists available at ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

Temporal evolution of seismic fragility curves for concrete box-girder bridges in California

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ARTICLE INFO

Article history:
Received 8 January 2015
Revised 31 March 2015
Accepted 31 March 2015
Available online 17 April 2015

Keywords:
Fragility
Box-girder bridges
Seismic design
Evolution of bridge design philosophy
Bridge classes
HAZUS

ABSTRACT

Fragility analyses is a powerful tool for the seismic risk assessment of highway bridges. Multispan continuous concrete box-girder bridges account for the bulk of the California bridge inventory. Although several studies have looked at the seismic response and fragility of individual box-girder bridges, very little research has explored the fragility of the box-girder bridge class holistically. Exacerbating this situation is the lack of considerations of design details unique to the era in which the bridges were designed and constructed, in their fragility development. This paper addresses this gap by investigating the influence of evolution of seismic design principles and details on the seismic performance of multispan continuous concrete box-girder bridge class, as well as the failure probability through the development of fragility curves across three design eras demarcated by the 1971 San Fernando and 1989 Loma Prieta earthquakes. Detailed nonlinear analytical models capturing the bridge characteristics within a design era are developed and nonlinear time history analysis is employed to develop analytical fragility curves. Significant improvement in performance is seen in modern bridges designed according to capacity design principles.

1. Introduction

Bridges form a critical link in a highway network and are vulnerable to earthquakes, often with severe consequences in terms of impaired emergency response, economic loss and its effect on the regional economy. With the advancement of Performance-based Earthquake Engineering (PBEE) framework [1,2], metrics such as damage probability functions or fragility curves for describing the performance and vulnerability of highway bridges under seismic input are crucial and of paramount importance. Fragility curves are conditional probability statements that give the likelihood that a structure will meet or exceed a specified level of damage for a given ground motion intensity measure (IM).

Multispan continuous concrete box-girder (MSCC-BG) bridges account for the bulk of the overall (state and local) California inventory. Based on the analysis of the National Bridge Inventory (NBI) [3] and in-house databases of state bridges assembled by the California Department of Transportation (Caltrans) engineers, it is seen that MSCC-BG bridges account for about 37% of the state bridge inventory and is the predominant bridge type. Most of the fragility curves developed for California bridges are structure specific [4,5] which do not capture the uncertainty associated with the geometric parameters that describe a bridge class and other uncertainties associated with them. On the other hand, researchers [6–9] developed fragility curves for as-built (seismically and non-seismically designed bridges) and retrofit bridge classes in central and south eastern United States (CSUS). These are not applicable for vulnerability assessment in California due to discrepancies in the composition of bridge classes and design details. Further, there is a significant evolution in the seismic design philosophy for bridges in California over the last few decades which is absent in the case of CSUS bridges, thereby preventing the adoption of CSUS bridge class fragilities for their California counterparts. Added discrepancies in the definition of damage states to support regional risk assessment and decision-making needs, further add

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http://dx.doi.org/10.1016/j.engstruct.2015.03.069
0141-0296/

to the incompatibility between CSUS and California bridge class fragilities.

The only fragility curves that are remotely applicable to bridge classes in California were the ones developed by Mander and Basoz [10] and are employed in HAZUS [11]. The HAZUS fragility relationships were developed for bridge classes based on a limited number of parameters available in the NBI, damage states based on limited sets of field damage observations and simplified two dimensional analysis techniques. Another significant drawback in the field of bridge seismic risk assessments is the mismatch between the damage state definitions used in fragility analysis and overall bridge functionality post a seismic event. This hampers the decision making needs by agencies like Caltrans with regards to emergency response and management. A gap currently exists in the literature and fragility models used in practice to support risk assessment of bridge classes that align with decision making needs of the bridge maintenance community. Exacerbating this situation is the lack of systematic organization of bridge design, retrofit, and maintenance data (beyond the NBI parameters) required to make substantial improvements.

With an evolution of bridge design standards over time, box-girder bridges have a broad range of differences in their design attributes such as evolution of the column design philosophy, hinge seats, superstructure to substructure connectivity, and restrainer details to mention the least. Quantification of their vulnerability by not accounting for these features, as in the case of the existing HAZUS [11] fragilities, might not be very appropriate. The aim of the current study is twofold:

1. Capturing the evolution in design details for California bridges (with emphasis on box-girder bridge class) based on a thorough review of bridge-plans, reconnaissance data, and in-house databases.
2. Quantifying the importance of these design changes on the fragility of box-girder bridges to uncover the significance of accounting for design era in seismic fragility modeling of bridges.

Detailed nonlinear analytical models capturing the bridge characteristics within a design era are developed and nonlinear time history analysis is employed to develop analytical fragility curves. The study considers multiple component vulnerability and uncertainty in geometric and material parameters in addition to the imposed seismic demand.

2. Bridge design eras and typical design details

Seismic bridge design provisions in California have evolved significantly over the last few decades in response to the deficiencies exposed after significant seismic events [12–16]. In order to develop reliable metrics such as fragility curves to quantify the seismic performance of bridges classes, it is imperative to understand the vulnerability associated with them as these design standards evolve. Geometric attributes captured in the NBI [3], such as span length, deck width, number of spans, and minimum vertical underclearance which are often used to support bridge fragility modeling, do not furnish sufficient information regarding the design details that affect bridge seismic vulnerability. This section identifies unique bridge design attributes and their evolution over three significant design eras, separated by the historic 1971 San Fernando and the 1989 Loma Prieta earthquakes. Such identification is achieved by an in-depth review of a few hundred bridge plans pertinent to the design eras for the chosen bridge classes, use of Caltrans in-house databases such as Bridge Information Records and Inspection System (BIRIS), extensive input from design engineers and maintenance staff together with lessons learned from past earthquakes.

2.1. Pre 1971 design era

Early Californian bridge seismic design codes dealt with the philosophy that seismic forces were proportional to the dead weight of the structure. Bridges were designed for a lateral seismic force equal to 6% of the structural dead weight until 1965, at which point structural period and amplification factors were considered [17]. The concept of ductility was absent and the detailing of reinforcement to achieve ductility by current standards was very poor.

2.1.1. Typical design details

The column shear reinforcement consisted of #4 transverse stirrups spaced at 12 in. on center regardless of the column size or the size of the longitudinal reinforcing bars. Very short seat widths in the range of 6–8 in. were typical at the expansion joints. There was inadequate lap splice lengths of the column longitudinal bars near the footing and inadequate development of the column longitudinal bars into the footing without any standard hooks. Lap splicing of the column stirrups in the cover was also very common.

2.1.2. Vulnerabilities

The 1971 San Fernando earthquake revealed several vulnerabilities associated with bridges designed prior to that date. Column shear failure and pull-out of the longitudinal reinforcement was predominant due to the lack of ductility. Provision of short seat widths at the bents and the abutments increased the unseating potential. These were seen during the San Fernando, Loma Prieta and Northridge earthquakes [18–21].

2.2. 1971–1990 design era

The 1971 San Fernando earthquake emphasized the importance of detailing and ductility in the response of bridge structures with the introduction of capacity design principles in their design standards. The lateral load carrying capacity of the bridges was increased by a factor of 2 or 2.5 and the aspects of fault proximity, site conditions, dynamic structural response and ductile details [19]. These factors featured in the Caltrans design specifications in 1973. The Applied Technology Council (ATC) developed guidelines which were documented in the ATC-6 report [14]. These formed the basis for design of Caltrans bridges and primarily remained unchanged until the 1989 Loma Prieta earthquake. The standard practice was to design for plastic shear in the columns with the intention of failing the column in flexure while all the other components of the bridge remained elastic.

2.2.1. Typical design details

Some of the typical design details of this intermediate era are summarized below:

- The spacing of the transverse reinforcement in the columns was reduced with a typical spacing of 4–6 in. However, the confinement of the plastic hinge region was still absent.
- Increase in the negative moment reinforcement in footing and pile caps without any shear reinforcement.
- Splicing of column longitudinal bars was not permitted at locations of maximum moment.
- Seat widths were slightly increased from 6 to 8 in. in the Pre 1971 design era to about 12 in.
- Joint reinforcement between column and the bent cap and column and the footing was absent.