Engineering Structures 31 (2009) 2317–2326

Contents lists available at ScienceDirect

Engineering Structures

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

Bridge support elastic reactions under vertical earthquake ground motion

Frederic Legeron [a,1], M. Neaz Sheikh [b,*]

[a] Department of Civil Engineering, University of Sherbrooke, Sherbrooke (Quebec), Canada J1K 2R1

[b] School of Civil, Mining and Environmental Engineering, University of Wollongong, NSW 2522, Australia

ARTICLE INFO

Article history:
Received 3 October 2008
Received in revised form
1 May 2009
Accepted 1 May 2009
Available online 30 May 2009

Keywords:
Bridge
Seismic design
Vertical earthquake ground motion
Support reaction
Simplified method

ABSTRACT

In North America, bridges are not typically designed for vertical earthquake ground motion. Design codes, AASHTO in the USA and S6-06 in Canada, do not explicitly consider the effect of vertical earthquake ground motion in the design of highway bridges. However, analytical and field evidences have often drawn engineers' attention to the damage potential of vertical earthquake ground motion to engineered bridges. This paper proposes a simplified method to calculate elastic support reactions under vertical earthquake ground motion. Support reactions are first calculated by exact dynamic method. By applying a few assumptions, a simplified method has been developed. The developed method can be readily used in the design of typical bridges with regular span distribution. This simplified method has been demonstrated with several examples.

1. Introduction

Vertical earthquake ground motion is not explicitly considered in American and Canadian design codes for bridges [1,2]. Canadian Highway Bridge Design Code [2] takes into account the vertical earthquake ground motion in a simplified way by increasing and decreasing the dead load action in load combinations, irrespective of earthquake magnitude, fault distance, and site soil condition. In the Caltrans seismic design criteria [3], vertical ground motion is only considered as an equivalent static vertical load when the site peak rock acceleration is 0.6g or greater. The recent edition of Eurocode [4] has proposed to take into account the effect of vertical earthquake ground motion when the bridge is located within 5 km of an active seismotectonic fault or in high seismic zones.

When the effects of vertical earthquake ground motion are explicitly considered in the design process, the vertical response spectrum is taken typically as the two-thirds of the horizontal response spectrum for the entire period range of engineering interest. This approach was originally proposed by Newmark et al. [5] and has since been widely used. However, analyses of strong motion data indicate that in the vicinity of moderate to strong motion earthquakes, vertical to horizontal (V/H) peak ground acceleration ratio often exceeds unity; hence, the 2/3 rule is not reasonable [6–10]. In addition, a recent research investigation has also shown that V/H spectral ratio can approach 1.8 at large magnitude earthquake events for short site source distances and short periods [10]. Some recent studies have focused on the shape of the vertical response spectrum and proposed design vertical acceleration response spectrum consisting of a flat portion at short periods (0.05–0.15 s) and a decaying spectral acceleration at longer periods [8,10,11]. Typically, the vertical ground motion has lower energy content than the horizontal ground motion over the frequency range of interest. However, all its energy is concentrated in a narrow frequency band and can be destructive to the engineered structures having vertical frequencies within that range (5–20 Hz).

A literature survey indicates that only a few studies have been conducted to quantify the effect of vertical ground motion on bridges [12–15]. Analytical studies, in line with field observations, found that certain failure modes are directly related to high vertical ground acceleration [15]. In addition to the possibility of compressive over-stressing or failure due to direct compression, vertical motion may induce failure in shear and flexure and can cause bearing and expansion joint failures. All the analytical studies carried out to date are based on finite element modelling of the bridges and emphasize the importance of incorporating the vertical ground motion in the design process [12–15]. However, these studies have not clearly defined the way to incorporate the effects of vertical ground motion into the design process. One of the studies emphasized the need to develop a simplified method for the inclusion of the vertical earthquake ground motion in the design process [15].

The purpose of the current study is to develop a simplified method to quantify the elastic support reactions of bridges under

* Corresponding author. Tel.: +61 2 4221 3009; fax: +61 2 4221 3238.
E-mail addresses: frederic.legeron@usherbrooke.ca (F. Legeron), msheikh@uow.edu.au (M. Neaz Sheikh).

1 Tel.: +1 819 821 7395; fax: +1 819 821 7974.

0141-0296/$ – see front matter © 2009 Elsevier Ltd. All rights reserved.
doi:10.1016/j.engstruct.2009.05.001

Nomenclature

The following notations are used in this paper:

A	Zonal acceleration ratio
a	Response spectral acceleration at the effective period of supports (for rigid bridge: maximum responses spectral acceleration)
C_{sm}	Elastic seismic coefficient
EI	Stiffness of superstructure
I	Importance factor
L	Main span length
L^n	Modal participation factor of mode n
M^n	Modal mass of mode n
R_i	Support reaction of support i
S_a	Response spectral acceleration
$S_{a,max}$	Maximum response spectral acceleration
S_d	Response spectral displacement
T_n	Modal period of the bridge
T_{eff}	Effective natural period of the support
w	Superstructure unit weight
ζ	Scaling factor
φ^n	Mode shape of mode n
ϕ	Phase angle
ψ	Ratio of length between end span and main span
Ω^n	Spatial natural frequency
Γ^n	Unit modal support reaction of mode n
μ	Superstructure unit mass

vertical earthquake ground motion. It is noted that nonlinear effect is not generally considered for the vertical earthquake ground motion [4]. The developed method requires few calculations and can be readily used by design offices for the design of simple highway bridges and preliminary design of more complex bridges. The method has been found to be in excellent agreement with the analytical results.

2. Calculations of support reactions under vertical earthquake ground motion

Generally, three methods are used in practice for the calculation of support reactions: (i) Rayleigh method: This method has been recommended by American and Canadian bridge design codes [1, 2] for irregular ordinary multispan bridges and regular essential or emergency-route bridges in seismic performance zone 2 or less. Rayleigh method is generally not recommended for vertical ground motion since the choice of deflection shape for a complex system is not straightforward [16] and does not provide results close to the exact calculation in most cases (refer examples in Section 5). (ii) Modal analysis: This method is most suitable for structures with irregular geometry, mass and stiffness. This method does not take into account the nonlinear effects. However, nonlinear effect is not considered significant for vertical earthquake ground motion [4], as mentioned earlier. Moreover, ductility of bridge piers under vertical earthquake ground motion is not well known and usually considered to be low. (iii) Time history analysis: This method is very complex and time consuming and is usually carried out only for critical structures in high seismic zones [1]. Moreover, the choice of representative ground motion may add complexity in applying this method. Hence, elastic modal analysis has been considered sufficient for the calculation of support reactions under vertical earthquake ground motion in this study.

It is also possible to calculate the support reactions directly by applying dynamics of continuum [17]. Considering an unloaded

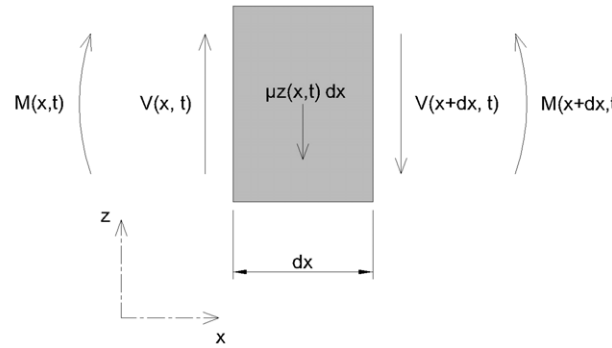

Fig. 1. Equilibrium of a slice of bridge superstructure.

slice of bridge superstructure (Fig. 1) and assuming superstructure unit mass $\mu(x) = \mu$ and stiffness $EI(x) = EI$, equilibrium of forces results in:

$$\frac{\partial^2 z(x,t)}{\partial t^2} + \frac{EI}{\mu}\frac{\partial^4 z(x,t)}{\partial x^4} = 0. \quad (1)$$

Solution of this equation can be found by classical separation of spatial and time variables:

$$\begin{aligned} Z(x,t) &= Z(t)\cdot\varphi(x) \\ &= (A\cos(\Omega x) + B\sin(\Omega x) + C\cosh(\Omega x) \\ &\quad + D\sinh(\Omega x)) \times \cos(\omega t + \phi) \end{aligned} \quad (2)$$

where A, B, C and D are integration constants related to boundary conditions; ϕ is a constant depending on initial conditions; Ω is the spatial natural frequency; and ω is the natural frequency. They are related by Eq. (3).

$$\Omega^2 = \omega\sqrt{\frac{\mu}{EI}}. \quad (3)$$

There are four unknowns in each span (A, B, C and D). For each span, two equations can be written for zero displacement at supports and another two equations can be written for continuity of moments (or zero moments at end support). It results in a system of $4r$ equations where r is the number of spans. Using matrix notation:

$$[M]\vec{X} = \vec{0} \quad (4)$$

where $[M]$ is a real matrix of $4r \times 4r$-dimension with terms $\pm\cos(L_i\phi\Omega)$, $\sin(L_i\phi\Omega)$, $\cosh(L_i\phi\Omega)$ and $\sinh(L_i\phi\Omega)$, and L_i is the length of span i.

$\vec{X}$ is a vector of $4r$-dimensions with components $\{A_1, B_1, C_1, D_1, \ldots, A_i, B_i, C_i, D_i, \ldots, A_r, B_r, C_r, D_r\}$.

The system of $4r$ equations has nontrivial solution only if

$$\mathrm{Det}(M) = 0 \quad (5)$$

which has $4r$ eigenvalues Ω^n corresponding to eigenmodes $\varphi^n(x)$. Eigenmodes are chosen so that $\int\varphi^n(x)\mathrm{d}x = 1$ to create an orthogonal base of eigenvectors.

This set of equations can be solved for some typical bridges. The bridge geometry is chosen as follows:

(i) single-span bridge with span length L;
(ii) two-span bridge with first span length ΨL and second span length L;
(iii) three-span bridge with end spans length ΨL and central span length L;
(iv) four-span bridge with end spans length ΨL and interior spans length L.

Ω^n can be calculated as a function of $1/L$ according to Eq. (6):

$$\Omega^n = \frac{\alpha^{n,\psi}}{L}. \quad (6)$$