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Dynamic soil structure interaction of bridge piers supported on well foundation

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ABSTRACT

For both steel and RCC Bridges passing rivers or creeks, common practice in many countries is to provide concrete wells to support the bridge girders. For many bridges that are strategically important in terms of defense or trade, it is essential that they remain functional even after a strong earthquake hits the structure. The present state of the art for design of well foundation is still marred with a number of uncertainties where a simplistic pseudo static analysis of its response only prevails, though it is a well-known fact that loads from super structure, character of soil and its stiffness plays an important role in defining its dynamic characteristics. The present paper is thus an attempt to present a dynamic analysis model trying to cater to a number of such deficiencies as cited above and also provide a practical model (amenable to design office application) that can be used to estimate the pier, well and soil's dynamic interaction

1. Introduction

Well foundations otherwise called caissons are often deployed to support a number of important bridges around the world. Verrazano Narrows, San Francisco-Oakland bay bridge in USA, Rokko Island Bridge in Japan, Mahanadi River and Kolaghat Rail Bridge in India are some of the bridges that have been built on large diameter well foundations.

In all these cases the super-structure or the top deck is supported on massive piers, which in turn are supported on large diameter caissons transferring the load to foundation soil.

Because of large diameter (5–12 m) and depth (15–30 m or even more) of the caissons, it has long been assumed that well foundations are far too massive and stiff to be affected by any vibration either due to moving traffic or earthquake. Thus most of the codes of practice assumes the bridge pier supporting the superstructure to be fixed at top of well.

However, observations on performance of some of the bridges in recent earthquakes like Loma Prieta (USA) 1989, Kobe (Japan) 1999, Nepal 2015 (in Nepal and India), it is found that assuming the pier as fixed at its base is certainly not realistic. Despite being huge, well foundations are significantly affected by the propagating waves during an earthquake that affects the response of the bridge pier in turn.

Design and construction of such well foundations and piers are usually carried out as per recommendations of the codes of respective countries and some of the most commonly used codes are IRC 6,45 and 78 [1–3], AASHTO [4], CALTRANS [5], and Eurocode 8 Part2 & 4 [6].

The design procedure adapted by many of these codes for seismic analysis is quite simplified (though popular perception is that it is conservative), and overlooks a number of crucial issues like:

1. Ignoring self weight of pier while calculating its time-period.
2. Ignoring the shear deformation characteristics of the pier column, as in many cases, especially for flyovers having variable height depending on slenderness ratio of the pier, this can well dominate the pier's dynamic behavior.
3. Inertial and kinematical interaction of the pier with well and its surrounding soil.

Present paper proposes two mathematical models based on which many of these limitations in terms of earthquake analysis can be overcome to arrive at a realistic result. The analysis is analytical in nature and does not require any special purpose software to be used and can well be carried out in general purpose utility software like, MATHCAD, MATLAB or even a spread sheet if necessary.

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Nomenclature

A	Area of pier cross section;
C_x, C_θ	Damping of soil in translation and rocking;
[C]	Damping matrix;
C_{Ti}^s	Constant of time period in shear mode;
C_{Ti}^b	Constant of time period in bending mode;
D	Diameter of well foundation, Dead load reaction fro superstructure;
D_f	Depth of well foundation;
e	Embedded depth of foundation;
E	Young's modulus;
F	Horizontal force at tip of pier;
g	Acceleration due to gravity;
G	Shear modulus of soil or beam;
H	Height of pier;
I	Importance factor or moment of inertia of pier;
II	Inertial interaction;
KI	Kinematic interaction;
J_θ	Mass moment of inertia;
k	Stiffness;
K_e,K_b,K_s	Equivalent, bending and shear stiffness respectively;
[K]	Stiffness matrix;
m	mass;
m_i	modal mass in i^{th} mode;
M_f	Mass of foundation;
M_{si}	Moment in pier in mode i;
[M]	Mass matrix;
N	Number of blows in a SPT test;
p_x	Modal load vector;
[P]	Load vector;
r_s	Slenderness ratio;
R	Radius of gyration, Response reduction factor;
S_d,S_a	Spectral amplitude and acceleration;
T	Time period;
T_e,T_b,T_s	Effective, bending and shear mode time period respectively;
u_{si}	Amplitude of pier in mode i;
V_s	Shear wave velocity;
V_{si}	Shear in pier in mode i;
W_d	Load from top deck;
Y, Y_m	General expression for differential equation;
z	Vertical axis;
Z	Zone factor;
α_m	Parameter varying with mode number;
β	Code factor;
δ	Static deflection;
δ_t, δ_b, δ_s	Deflection of beam total, in bending and shear mode respectively;
φ_i	Eigen vector in i^{th} mode;
γ_s, γ_c	Density of soil and concrete;
η	Shear correction factor;
κ_i	Modal mass participation factor in i_{th} mode;
λ	A function;
μ_m	Parameter varying with mode number;
ρ_c	Mass density of concrete;
ω	Natural frequency;
ω_e, ω_b, ω_s	Effective, bending and shear mode natural frequencies;
ζ_x, ζ_θ	Modal damping ratio in translation and rocking.

2. Practice as in trend

Before we present the mathematical models for analysis of such system, the practice as in trend [1–3], and other codes used internationally [4–6] are briefly reviewed.

Present recommendations in [1–3] for design of bridges under seismic force are based on a study by Murthy and Jain [7] to bridge the gap between [1] and state of the art international practice as prevalent and made a significant improvement/modification to [1], compared to its previous version.

The study and subsequent recommendation are in line with procedure as furnished in [4,5] and is almost analogous in philosophy.

In the present study, though a number of recommendations were made on different aspect of bridge design under seismic loading like adapting different response reduction factor (R) for different parts of the bridge, development of plastic hinges, damping properties etc., the overview is restricted to the seismic response of the bridge pier part and its foundation only.

As per [1], time-period of pier in fundamental mode is computed from the expression

$$T = 2\sqrt{\frac{D}{1000F}} \tag{1}$$

In Eq. (1), D = Dead load reaction from superstructure in kN. F = Horizontal force in kN, to be applied at center of mass of the superstructure for one mm horizontal deflection of the bridge along considered direction of horizontal force.

In this context, it should be noted that code has not stated whether we use an Euler-Bernoulli type beam or a Timoshenko type beam for computation of F. The practice is usually to use beams ignoring the shear deformation.

The basis of Eq. (1) is obviously assuming the bridge pier as a member having single degree of freedom fixed at its base, with load from superstructure considered as a lumped mass at its head (see Fig. 1). The inertial mass of the pier itself is ignored.

Above expression is similar to what has been proposed in [4,5] that recommends an expression

$$T = 0.32\sqrt{\frac{D}{F}} \tag{2}$$

here D and F are same as defined in Eq. (1), except that values are in FPS unit.

The basis of these expressions is actually $T = 2\pi\sqrt{m/k}$ where m is the lumped mass considered at top of pier and k the stiffness of pier in fundamental mode.

Japanese code JRA Part V [8] also recommends an expression similar in nature and is given by.
$T = 2.01\sqrt{\delta}$. Here δ= static deflection at the tip of pier due to lateral load in meter. This can well be derived from the expression $T = 2\pi\sqrt{m/k} \rightarrow 2\pi\sqrt{\delta/g}$. Assuming g=9.81 m/s^2. One can easily arrive at the equation as proposed in the Japanese code.

This again shows that the idealized model used to compute time period in this case is also a single degree lumped mass inverted pendulum type, like one used in [1–6].

Fig. 1 shows typical bridge piers deployed in practice to support the superstructure of a bridge/flyover.

For the lumped mass model as shown in Fig. 1, ignoring the inertial effect of pier, one may argue is conservative, as the added mass of pier will only go on to elongate the time period, which will either reduce the spectral acceleration or it may even remain invariant-depending on pier geometry and superstructure load. However, as deflection, moment and shear are proportional to square of the time period will be *lower bound*. The pier mass should also be incorporated in the time period expression to arrive at a realistic result.

In a particular flyover, supporting piers (having same diameter/width) may have different height. They are usually tallest at the center