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Simple method to estimate residual displacement in concrete bridge columns under near-fault earthquake motions

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ABSTRACT

The most direct method to estimate lateral residual displacement under earthquake loading is nonlinear response history analysis, which is time consuming and complicated. A simple method is highly desired to estimate residual displacement. Different simple methods that were based on earthquake and structural characteristics were explored. An empirical method using data from shake table testing of six bridge columns was developed and the results were compared with those from a method developed by the Applied Technology Council and the Japanese code. Compared to other methods, the proposed method was found to provide a more reasonable estimate of residual displacements under moderate and strong earthquakes.

1. Introduction

Bridges are key components in the transportation network providing access for emergency response vehicles following strong earthquakes. Bridge columns are expected to undergo large inelastic deformations during severe earthquakes that can result in residual lateral displacements particularly under near-fault ground motions. Residual drift ratio is an important measure of post-earthquake functionality in bridges and can determine whether a bridge should be kept open to traffic or closed for repair or replacement. Following the 1995 Kobe earthquake, Kawashima et al. [1] reported that over 100 reinforced concrete bridge columns with a residual drift ratio of over 1.75% were demolished even though these bridges had not collapsed. A method to estimate residual displacements was developed in that study.

It has been observed that near-fault ground motions are frequently characterized by intense velocity pulses. These pulses expose structures in near-fault regions to high input energy that may result in high lateral displacement demands especially significant residual displacements in bridge columns (Saiidi and S. Ardakani [2]; Choi et al. [3]; Phan et al. [4]; Zatar and Mutsuyoshi [5]). The strong velocity pulse in the fault normal component of near-fault ground motions causes a whiplike behavior resulting in a large displacement that pushes the column to one side. This displacement is only partially recovered during the earthquake because the stiffness of the column upon load reversal is relatively large. As a result, large residual displacements are developed. This pulse-type velocity motion is particular to the forward direction where the fault rupture propagates toward the site at a velocity close to the ground shear wave velocity causing most of the energy arrive at the site within a short time. Near-fault ground motions come in large variations and this variety complicates evaluation or prediction of structural response. Analytical investigations by Saiidi and S. Ardakani [2], Lee and Billington [6], and Yazgan and Dazio [7] showed that accurate estimation of residual displacements is difficult to achieve. The objective of the study presented in this article was to develop a simple method to estimate residual displacements. Four approaches utilizing different aspects of the input motion and structural dynamic properties were investigated to determine the feasibility of using these properties for an approximate, simple estimation of residual displacements. Because these approaches did not yield satisfactory results, an empirical simple method was developed based on measured data of previously tested columns. This paper describes the first four approaches and the empirical methods and their ability to estimate residual displacements of columns subjected to near-fault ground motions. The results of the empirical method are compared with those from other available methods.

2. Current provisions to estimate residual displacement

There are currently no guidelines for the design of reinforced concrete bridge columns with respect to calculation of residual displacement in either the AASHTO Guide Specifications for LRFD Seismic Bridge Design [8] or the Caltrans Seismic Design Criteria [9].

The ATC 58 [10] document recommends a simple formulation to calculate the residual story drift for buildings. According to ATC 58,

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median residual story drift, Δ_r, should be calculated as:

$$\Delta_r = \begin{cases} 0 & \Delta < \Delta_y \\ 0.3(\Delta - \Delta_y) & \Delta_y < \Delta < 4\Delta_y \\ \Delta - 3\Delta_y & \Delta > 4\Delta_y \end{cases} \tag{1}$$

Where Δ is the median maximum story drift and Δ_y is the median yield story drift calculated by analysis. The yield drift can be calculated as the story drift associated with story shear forces that cause (a) the beams and/or columns reach their expected plastic moment capacity taking into account the effect of axial forces in the members, or (b) the beam-column joint panel reaches its expected yield strength.

After the 1995 Hyogo-ken Nanbu earthquake, part V: Seismic Design of the Design Specifications of Highway Bridges (Japan Road Association, JRA [11]) was revised to provide an estimate of residual displacement [12]. The method involves many parameters, some of which are not well defined for a broad range of columns, and hence are selected based on judgment. The method considers the ratio of post-yielding to cracked column stiffness to be a major factor. Based on the JRA specifications, in Type B bridges (important bridges, which should perform with limited damage for extreme ground motions with low probability of occurrence), residual displacement at a pier after an earthquake must be checked as [12,13]:

$$\delta_R < \delta_{Ra} \tag{2}$$

Where

$$\delta_R = c_R(\mu_R - 1)(1 - r)\delta_y \tag{3}$$

$$\mu_R = 1/2\{(k_{hc}W/P_a)^2 + 1\} \tag{4}$$

$$k_{hc} = c_z k_{hc0} \tag{5}$$

$$W = W_u + c_P W_P \tag{6}$$

$$P_a > k_{he} W \tag{7}$$

$$k_{he} = \frac{k_{hc}}{\sqrt{2\mu_a - 1}} \tag{8}$$

$$\mu_a = 1 + \frac{\delta_u - \delta_y}{\alpha\delta_y} \tag{9}$$

In which δ_R is residual displacement of the pier after an earthquake, δ_{Ra} is allowable residual displacement, r is bilinear factor defined as a ratio of the post-yielding stiffness and the cracked column stiffness, c_R factor depending on the bilinear factor (r), which is 0.6 for a RC pier, μ_R is response displacement ductility factor of the pier, δ_y is yield displacement of the pier, δ_u is the maximum displacement demand of the pier, k_{hc} is lateral force coefficient, k_{hc0} is a factor depending on the ground motion type, the soil, and the period (Table 1), c_z is modification coefficient, which is 0.7, 0.85, and 1.0 depending on the seismic zone, W is equivalent weight, W_u is the weight of a part of superstructure supported by the pier, W_p is the weight of pier, c_P is coefficient depending on the type of failure mode (0.5 for a pier in which either flexural failure or shear failure after flexural cracks are developed, and 1.0 is for a pier in which shear failure is developed), k_{he} is equivalent lateral force coefficient, μ_a is allowable displacement ductility factor, α is safety factor, which is 1.5 for Type B bridges and Type II ground motions. This type of ground motions comprise pulse-type accelerations with high peaks and short duration while the Type I ground motions are more repetitive accelerations with long duration. P_a is lateral load capacity of a pier when the force is applied at the gravity center of the superstructure. δ_{Ra} is 1/100 of the distance between the base of the pier and the gravity center of the superstructure (1% drift).

Table 1
Coefficient k_{hc0}.

Soil condition			
Type I Ground Motion			
Group I (Stiff)	0.7 for T ≤ 1.4	$0.876\,T^{2/3}$ for T > 1.4	
Group II (Moderate)	$1.51\,T^{1/3}$(k_{hc0} ≥ 0.7) for T < 0.18	0.85 for 0.18 ≤ T ≤ 1.6	$1.16\,T^{2/3}$ for T > 1.6
Group III (Soft)	$1.51\,T^{1/3}$(k_{hc0} ≥ 0.7) for T < 0.29	1.00 for 0.29 ≤ T ≤ 2.0	$1.59\,T^{2/3}$ for T > 2.0
Type II Ground Motion			
Group I (Stiff)	$4.46\,T^{2/3}$ for T ≤ 0.3	2.00 for 0.3 ≤ T ≤ 0.7	$1.24\,T^{4/3}$ for T > 0.7
Group II (Moderate)	$3.22\,T^{2/3}$ for T < 0.4	1.75 for 0.4 ≤ T ≤ 1.2	$2.23\,T^{4/3}$ for T > 1.2
Group III (Soft)	$2.38\,T^{2/3}$ for T < 0.5	1.50 for 0.5 ≤ T ≤ 1.5	$2.57\,T^{4/3}$ for T > 1.5

Table 1 presents the standard modification coefficient k_{hc0} for Type I and Type II ground motions. k_{hc0} accounts for the frequency-dependent amplification of ground motions by different local site conditions. The Type I ground motions had been used since 1990 Specifications, while the Type II ground motions were incorporated in the 1996 Specifications. The Type I ground motions had been derived from a statistical analysis of recorded ground motions based on the assumption that design force is smaller at stiff sites. This was because damage had been more developed at soft soil sites in the past. However, the Type II ground motions were determined by taking envelopes of response accelerations of recorded ground motions in Kobe during the Hyogo-ken Nanbu earthquake. The response spectral accelerations for this type of ground motions were larger at stiff sites than softer soil sites [12].

3. Summary of experimental studies

In previous studies by Phan et al. [4] and Choi et al. [3], six large-scale reinforced concrete bridge columns were tested on a shake table in the Large Scale Structures Laboratory at the University of Nevada, Reno (UNR). The results from those tests were used to develop an empirical method for estimating residual displacements. The columns were all flexure-dominated circular spiral columns tested as cantilever members. The specimens were labeled NF-1, NF-2, MN, ETN, SETN, and SVTN. The sections of the columns are presented in Fig. 1. The design of NF-1 was based on the 2004 Caltrans Seismic Design Criteria [14], but did not incorporate the Caltrans near-fault guidelines. Details of NF-1 is shown in Fig. 2. The design of NF-2 was based on the AASHTO 2002 Standard Specifications for Highway Bridges [15]. Testing of MN, which was similar to NF-1 but accounted for Caltrans near-fault guidelines, demonstrated that by using the provisions of Caltrans near-fault motion amplification, residual displacements are decreased by a moderate amount. The difference between MN and ETN was the initial cracked stiffness period of the columns. SETN was comparable to ETN; however, spectral acceleration for SETN was determined from a new design spectrum by Somerville [16]. Compared to Caltrans near-fault spectrum, the new spectrum presented considerably higher spectral accelerations (SA) at the period of 1.25 s or higher. SVTN was designed as a long- period column according to the new design spectrum. Table 2 presents information for all six columns. Details of the studies are presented in Phan et al. [4] and Choi et al. [3].

The NF-1, NF-2, MN, and ETN were tested using the fault normal component of the Rinaldi ground motion from the 1994 Northridge earthquake. The Rinaldi Receiving Station (RRS) record was applied in SETN and SVTN. The RRS record was a synthetic motion generated by matching in the frequency domain the Rinaldi motion to the new acceleration response spectrum developed by Somerville. The loading protocol for all the columns consisted of small amplitude motions followed by motions with gradually increasing amplitudes from one motion to the next until the columns failed or the shake table testing limits were reached. The most unique response seen in shake table testing of the specimens was the relatively large residual displacements even under moderate motions. Sample results presented in Choi et al. [3] demonstrate that successive input motions do not necessarily lead to