Engineering Structures 33 (2011) 3351–3363

Contents lists available at SciVerse ScienceDirect

Engineering Structures

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

Real-time dynamic substructuring testing of viscous seismic protective devices for bridge structures

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ARTICLE INFO

Article history:
Received 10 November 2009
Received in revised form
21 August 2010
Accepted 27 June 2011
Available online 10 August 2011

Keywords:
Real-time dynamic substructuring testing
Seismic isolation
Seismic protective systems
Viscous dampers
Bridge engineering
Seismic effects
Earthquakes
Seismic design
Finite elements
Nonlinear analysis

ABSTRACT

This paper presents a real-time dynamic substructuring (RTDS) test program carried out on bridge structures equipped with two innovative viscous seismic protective devices: a seismic damping unit and a shock transmission unit. In the RTDS tests, the seismic protective units were physically tested in the laboratory using a high performance dynamic actuator imposing, in real time, the displacement time histories obtained from numerical simulations being run in parallel. The integration scheme used in the test program was the Rosenbrock-W variant, and the integration was performed using The MathWorks' Simulink and XPC target computer environment. The numerical counterpart included the bridge columns and the additional energy dissipation properties. The nonlinear response of these components was accounted for in the numerical models. The tests were run under various ground motions, and the influence of modeling assumptions such as damping and initial stiffness was investigated. Finally, the test results are compared to the predictions from nonlinear dynamic time history analyses performed using commercially available computer programs. The results indicate that simple numerical modeling techniques can lead to accurate prediction of the displacement response of bridge structures equipped with the seismic protective systems studied.

1. Introduction

Bridges are key elements of transportation systems and are crucial for economic prosperity. They must therefore be designed to withstand natural hazards such as ground motion effects from earthquakes. In Canada, seismic activity in highly populated areas exists along the Pacific west coast in western Canada and along the St Lawrence and Ottawa River valleys in eastern Canada. Seismic design provisions have been progressively implemented in the CSA-S6 Canadian Highway Bridge Design Code starting in 1966 [1], but only minimum earthquake horizontal design loads were prescribed until 1988 [2]. Additional seismic load requirements for bearings and qualitative ductile detailing provisions for reinforced concrete columns were introduced in 1988, but it was only in 2000 that explicit seismic detailing requirements and capacity design principles were introduced in CSA-S6. Between the 1966 and the latest 2006 [3] editions of CSA-S6, the prescribed seismic design forces have also steadily increased.

A recent study revealed that the average age of bridges and overpasses in Canada had exceeded 57% of their service life of 43.3 years in 2007 [4]. That number increases to 72% in the Province of Quebec, the highest in the nation, indicating that the vast majority of the existing bridges in the eastern Canada seismic active region may be at risk and require seismic retrofit. Seismic hazards also significantly impact the construction of new bridge structures as a result of the increasing severity and complexity in seismic detailing and the higher seismic design loads prescribed in recent code editions. In this context, there is an increased need for innovative techniques to achieve time- and cost-effective seismic retrofit and construction of bridge structures.

Although seismic base isolation has been known since the beginning of the 20th century, it was introduced in North America only in the 1980s [5]. In Canada, seismic isolation for bridge retrofit has been applied in British Colombia since the early 1990s [6]. In Quebec, the first seismically isolated bridge structure was built in Alma in 2002 [5]. The same year, seismic dampers and shock transmission units (or lock-up devices) were implemented for the first time in a bridge retrofit project in Quebec City [7]. Although substantial work has been dedicated to investigate the behavior of seismically isolated bridges over the last 20 years, experimental and numerical studies are still needed to validate the effectiveness of bridge seismic isolation considering different loading and seismic environments as well as the increasing variety of seismic protective systems.

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doi:10.1016/j.engstruct.2011.06.021

This paper reports on a study of the dynamic response of a bridge structure located in Montreal, Quebec, and equipped with innovative seismic damping and shock transmitting systems when subjected to seismic demand typical of eastern North America and to vehicle braking forces. An extensive experimental program consisting of real-time dynamic substructuring (RTDS) of seismically isolated bridges was conducted. RTDS testing is based on a substructuring technique where the investigated system is split into (i) a physical substructure consisting of a critical part or component tested experimentally under dynamic forcing, and (ii) a numerical substructure modeling the reaction of the remaining part of the system. To realistically emulate the behavior of the whole system during dynamic excitation, the control strategy and numerical algorithms are conceived so that the physical and numerical substructures interact in real time. A significant advantage of RTDS testing is that the physical substructure can be tested at full scale while including dynamic and hysteretic effects through real-time interaction between the physical and numerical substructures. This hybrid technique was first proposed by Nakashima and Takaoka [8] as an important improvement of the pseudo-dynamic testing method introduced by Takanashi et al. [9]. Hybrid simulation has been successfully applied recently to assess the dynamic response of bridges [10,11], and structures equipped with seismic protective devices [12,13]. In the present work, special attention is devoted to investigating the effects of high frequency content ground motions typical of eastern North America. We also studied the dynamic response of the shock transmission unit to braking forces, a loading condition that has not been addressed in the literature. Finally, the sensitivity of the RTDS testing results to Rayleigh damping assumptions is investigated. Nonlinear time history analyses of the seismically protected bridges are also carried out and the purely numerical predictions are validated against the RTDS testing results.

2. Seismic protective systems studied

Two innovative seismic protective systems are investigated in this work: (i) a seismic damping unit (SDU), (ii) and a shock transmission unit (STU). Both devices are manufactured by LCL-Bridge Products Technology [14]. They are made of a double-acting piston driving a fluid through a parallel set of tubular orifices, thus producing fluid shear to resist dynamic movement [15]. The SDU offers little to no resistance to slow movement, such as thermal expansion, creep, and shrinkage, while it reduces dynamic movements due to braking or seismic loads through energy dissipation. The STU also allows slow movement, while offering an increasingly higher resistance to faster movement due to braking or seismic loads. In contrast to most lock-up systems, the STU tested in this work is designed so that the reacting force does not exceed an upper limit and, thereby, control the force demand imposed to the columns or abutments. Both types of device were used recently to retrofit a bridge over Highway 440 in Quebec [7].

There is a fair amount of literature about viscous-based devices and the numerical modeling of their dynamic response [7,15–22]. The reacting force F of a viscous damper can be expressed with Eq. (1).

$$F = C_p V^{\alpha}, \tag{1}$$

where C_p denotes the damping coefficient, V is the velocity of the piston, and α is a characteristic parameter. Typically, the parameter α varies between 0.1 and 2.0, with $\alpha > 1.0$ for a shock transmission device and $\alpha \leq 1.0$ for a viscous damper. In the latter case, the damper is linear when $\alpha = 1.0$, and nonlinear otherwise. Fig. 1 shows plots of reacting force F against piston velocity for different values of α. For the STU device investigated in this work, a parameter $\alpha < 1.0$ has to be adopted in view of its force-limiting

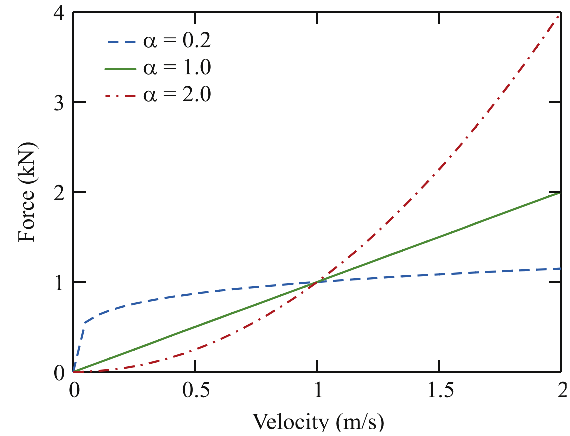

Fig. 1. Force in viscous device as a function of velocity considering $C = 1.0$ kN s/m.

capabilities. This will be discussed further. The two devices have the same exterior appearance and only differ by the design of the internal fluid flow system design. Fig. 2(c) shows an elevation view of an SDU or STU.

3. Bridge studied

A fictitious bridge structure located in Montreal, Quebec, is considered to investigate the seismic performance of the bridge equipped with the seismic protective devices. The bridge is straight and has two spans of 36.7 m each, as illustrated in Fig. 2(a). The bridge deck consists of four T-shaped reinforced concrete beams and has a total mass of 2560 t. The bridge deck is supported at mid-span by two reinforced concrete hammerhead wall columns placed side by side, as illustrated in Fig. 2(b). The bridge longitudinal response was examined in this study. Along this direction, the bridge deck is fixed to the columns and free to translate at the two abutments. The abutments are assumed to be infinitely stiff, and two seismic protective devices are introduced between the deck and one of the abutments.

Two different column designs were considered, depending on the type of seismic protective device used. For the SDUs, the columns were designed to yield under 0.5 times the elastic longitudinal seismic load of 5100 kN prescribed in the CAN/CSA S6-06 Canadian Highway Bridge Design Code [3], and the SDUs were selected to limit the longitudinal bridge deck displacements such that the columns remain elastic under the code seismic demand level. This design would result in no repair costs, no downtime period, and reduced construction costs, as no seismic ductility detailing would be required for the columns. It is likely that similar or even better performance could have been achieved with a lower yield strength for the columns and SDUs with greater damping capacity, but at the expense of higher force demand on the abutments. The selected combination was deemed to represent a good compromise between these design constraints. Using the RESPONSE 2000 computer program [23], the columns were found to yield under a total longitudinal horizontal load $F_y = 3200$ kN. An effective moment of inertia of 42% of the gross moment of inertia of the columns was considered to account for concrete cracking [23,24].

A different hypothetical scenario was considered for the STUs: the devices were assumed to be installed in an existing bridge built in the 1970s. No minimum seismic force requirement was prescribed in the 1974 [25] edition of S6, and the columns' longitudinal yield strength was set equal to 0.02 of the bridge deal load (F_y = 500 kN), as specified in the previous (1966) code edition. An effective cracked moment of inertia equal to 36% of the gross moment of inertia was assumed for the columns,