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SEISMIC ASSESSMENT OF HOLLOW CORE CONCRETE BRIDGE PIERS

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ABSTRACT

Hollow core concrete bridge piers are traditionally believed to be vulnerable to seismic action. However, the seismic vulnerability of such piers has not been investigated fully. In this paper, an analytical model to assess seismic vulnerability of hollow core concrete bridge pier is developed. The model is validated with available experimental results. Code recommendations for hollow core bridge piers are evaluated. It is shown that confinement reinforcement requirements in the codes are sometimes highly conservative and sometimes non-conservative. However, the recently developed confinement reinforcement equations for solid bridge piers at Sherbrooke University can be applied for economic and safe design. It is demonstrated that hollow core bridge piers are not as vulnerable as it is traditionally believed. Such piers can attain expected ductility, if designed properly.

Introduction

Bridges often rely solely on the capacity of the piers to sustain large displacement without collapsing. Failure of bridge piers often causes collapse or failure of bridge span, as it is evident from several major earthquakes. Hence, bridge piers are usually designed as the first structural element to dissipate seismic energy well beyond their elastic limit.

Hollow core piers are often used in the construction of long-span balanced cantilever bridges, cable-stayed bridges, and bridges crossing deep valleys where tall piers are required. Compared to solid piers, hollow core piers have the advantage of having significant reduction in the volume of the material, large reduction of dead load, and high bending and torsional stiffness. Despite its wide use, research on the seismic behaviour of such piers is limited. Even the most modern codes of practice do not recognize specific problems associated with hollow piers, probably as the consequence of lack of knowledge (Calvi et al., 2005). However, these types of piers are commonly considered to be vulnerable to seismic action.

The aim of this paper is to present an analytical tool to accurately model the seismic behaviour of hollow core concrete bridge piers. The predictions on real piers are compared with experimental results. Code recommendations for hollow core bridge piers are evaluated. Finally, the vulnerability of hollow core bridge piers is investigated by re-designing piers of an existing bridge and evaluating their seismic vulnerability.

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Analytical Model

Constitutive Laws of Materials

Reinforced concrete is a highly nonlinear material. Realistic constitutive law of reinforced concrete is complex as the nonlinearities arising from concrete and the reinforcing bars should be appropriately combined to accurately describe the experimentally observed behaviour of reinforced concrete elements. The nonlinear material model of reinforced concrete consists of constitutive laws of concrete and reinforcing bars.

Stress-Strain Relationship of Concrete

An accurate model of confined concrete must be validated in terms of concrete strength, transverse reinforcement yield strength, column geometry, or load conditions. The model should be based on a rational approach to the confinement phenomenon rather than on multicriteria statistical analyses. Legeron and Paultre (2003) uniaxial confined concrete model has been chosen as the constitutive law of concrete for the analytical modelling of hollow core bridge pier. The model reflects the various conditions described above and is validated with large number of experimental results. The model is considered most suitable compared to other contemporary models (Sharma et al., 2005). In the model, the behaviour of confined concrete is related to the effective confinement index I'_e ($I'_e=f_{le}/f'_c$, where f_{le} is the confinement pressure, and f'_c is the compressive strength of concrete), which takes into account the amount of transverse confinement reinforcement, the spatial distribution of the transverse and longitudinal reinforcement, the concrete strength, and the transverse reinforcement yield strength.

Stress-Strain Relationship of Longitudinal Bars

An accurate model of a stress-strain relationship of steel bars must simulate the following characteristics: (i) elastic, yielding and strain hardening branches in the first excursion, (ii) compression behaviour including buckling of bars in compression, (iii) cyclic behaviour, and (iv) low cycle fatigue and premature rupture of bars in tension due to cyclic loading and previous buckling in compression.

If the buckling of the reinforcing bar is not included in the modelling, behaviour of the pier at large inelastic deformation may be overpredicted. Gomes and Appleton (1997) model has been chosen since it is simple and is proven to predict bucking of bars quite well. The model takes into account the effect of inelastic buckling of longitudinal reinforcing bars in a simplified way based on the plastic mechanism of buckled bar. When a bar is subjected to cyclic load, its maximum strength is less than the maximum strength observed in monotonic tensile tests. Ultimate limit strain of the bar has been considered according to the simplified method proposed by Legeron (1998), based on tangent modulus theory. The apparent tensile strain at fracture generally comprises between 0.03 and 0.06 and is related to the spacing of transverse bars.

Modelling Sectional Behaviour

The complete moment curvature response of the hollow core section is computed with the MNPHI computer program (Paultre, 2001) with a layer by layer analysis incorporating the constitutive laws of concrete and reinforcing bars, as described above, assuming that plane section before bending remains plane after bending.

Member Force Displacement Relationship

Having established the moment-curvature relationship of the cross-section, flexural force displacement at the top of the pier can be calculated based on the moment area method with the moment diagram of the pier. The pier is subjected to a linearly varying bending moment between the top of the cantilever and the base. The variation of curvature along the column height is determined from moment curvature analysis, as discussed in the previous paragraph. It is assumed that average curvature with the assumed plastic hinge length is constant. A computer algorithm has been developed to calculate the flexural force-displacement behaviour of pier taking into account bar slippage and shear deformation (Legeron, 1998).

In most cases, in practice, piers fail in flexure and calculation as described above is sufficient. However, piers constructed before the adoption of modern codes of practices may fail in Shear. Shear capacity of the bridge pier is calculated based on UCSD approach proposed by Priestley et al. (1994) for normal strength concrete, and USC approach proposed by Xiao et al. (1998) for high strength concrete piers.

Comparison with Experimental Results

Seismic performance of hollow core bridge piers has been investigated experimentally by several researchers (Mo and Nien, 2002; Pinto et al., 2002; and Calvi et al., 2005). Experimental results of Mo and Nien (2002), Calvi et al. (2005) and Pinto et al. (2002) have been compared with analytical results. Excellent agreement has been obtained between the analytical results and experimental investigations. Due to the space limitations, analytical predictions for piers HI-1-b of Mo and Nien (2002), pier A70 of Pinto et al. (2002), and pier S250 of Calvi et al. (2005) have been reported herein. Full details of all the comparisons can be found in Vivier (2006).

Mo and Nien (2002) investigated the seismic performance of hollow high strength concrete bridge piers tested under constant axial load and a cyclically reversed horizontal load. Pier HI-1-b failed in shear. It can be seen that analytical model can predict the shear behaviour of the pier when shear capacity is calculated based on the phenomenological shear model (USC model) proposed by Xiao et al (1998) (Fig. 1a).

Pinto et al. (2002) presented the results of cyclic tests on two large scale models of Wrath Bridge piers with rectangular hollow cross-section. Pier A70 is expected to fail in flexure as it is over designed for shear, which is also apparent from the analytical results (Fig. 1b).

Calvi (2005) reported cyclic response of 11 pier specimens. All the specimens were assumed to be 1:4 scaled with aspect ratios equal to 2 or 3. Pier S250 is made with normal strength concrete and is designed to fail in shear. It can be seen that analytical model can well predict the shear behaviour of the pier when shear capacity is calculated based on the phenomenological shear model (UCSD model) by Priestley et al. (1994) (Fig. 1c).

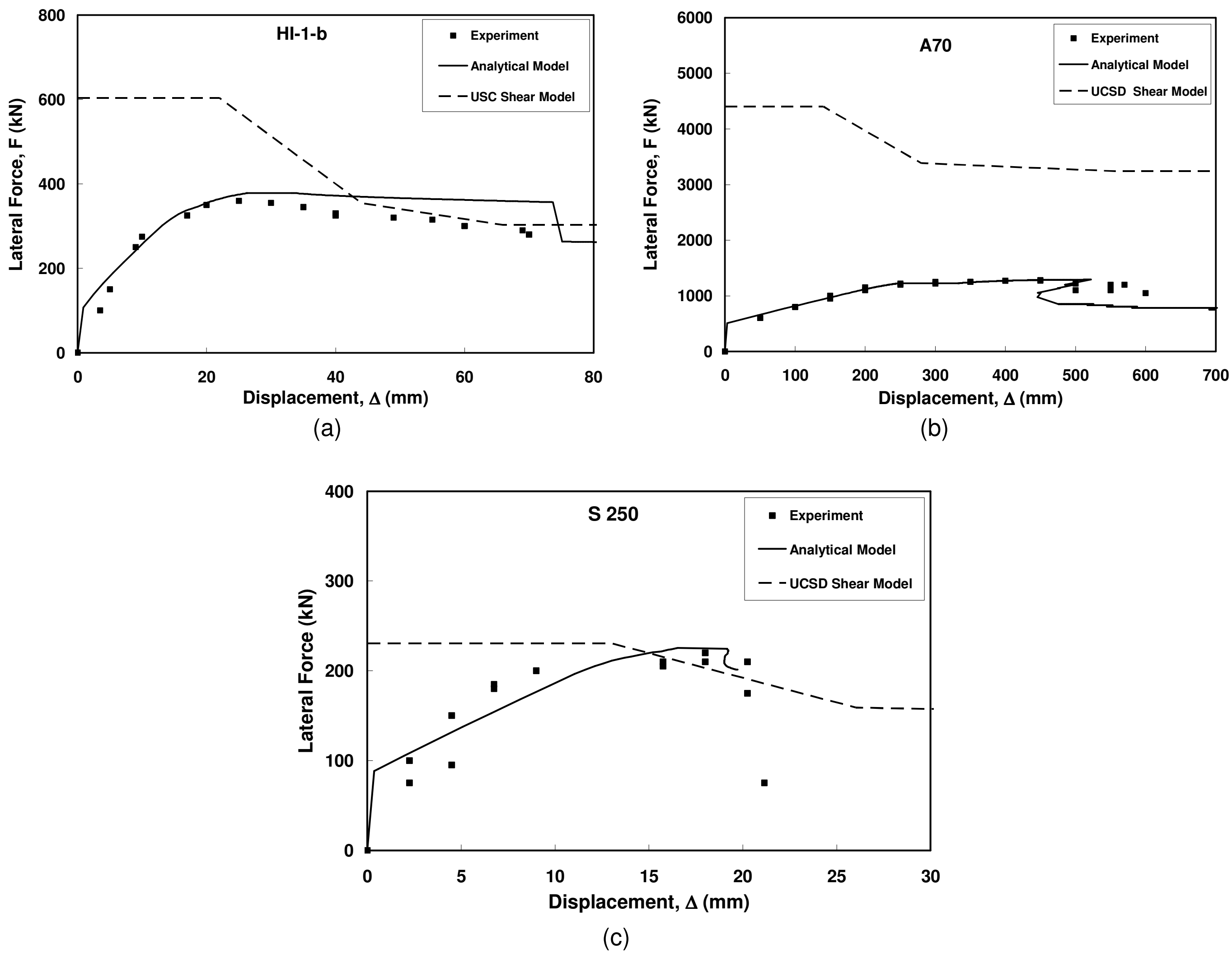

Figure 1. Experimental results compared with analytical predictions

It is evident from the comparison with the experimental result that the developed analytical method predicts the load-displacement behaviour of hollow core pier with reasonable accuracy. It has been observed that the proposed model can accurately predict the failure modes of the hollow core piers. The model has also been observed to accurately predict the behaviour as well as the performance limit states of solid piers (Sheikh et al., 2007).

Evaluation of Code Recommendations

Thickness of the Wall

Design codes recommend to confine hollow core piers as if they were solid (the hole is considered as if filled with concrete). This is counterintuitive and results in very high confinement demand. Parametric numerical study has been carried out to investigate the effect of wall thickness on the ductility capacity of hollow core bridge piers for different level of confinement. It can be observed that the ratio of concrete area to the overall cross-sectional area (A_c/A_g) has little influence on the ductility capacity of the bridge piers for all cases (Fig. 2a), except for very low longitudinal reinforcement (0.4%) with very low axial load ratio (n=0.087) (Fig. 2b). Even in such a case, the ductility capacity (μ_ϕ) of the piers remains nearly constant when the ratio of wall thickness (A_c/A_g) is more than 0.3, which is normally the case in most hollow core bridge piers.

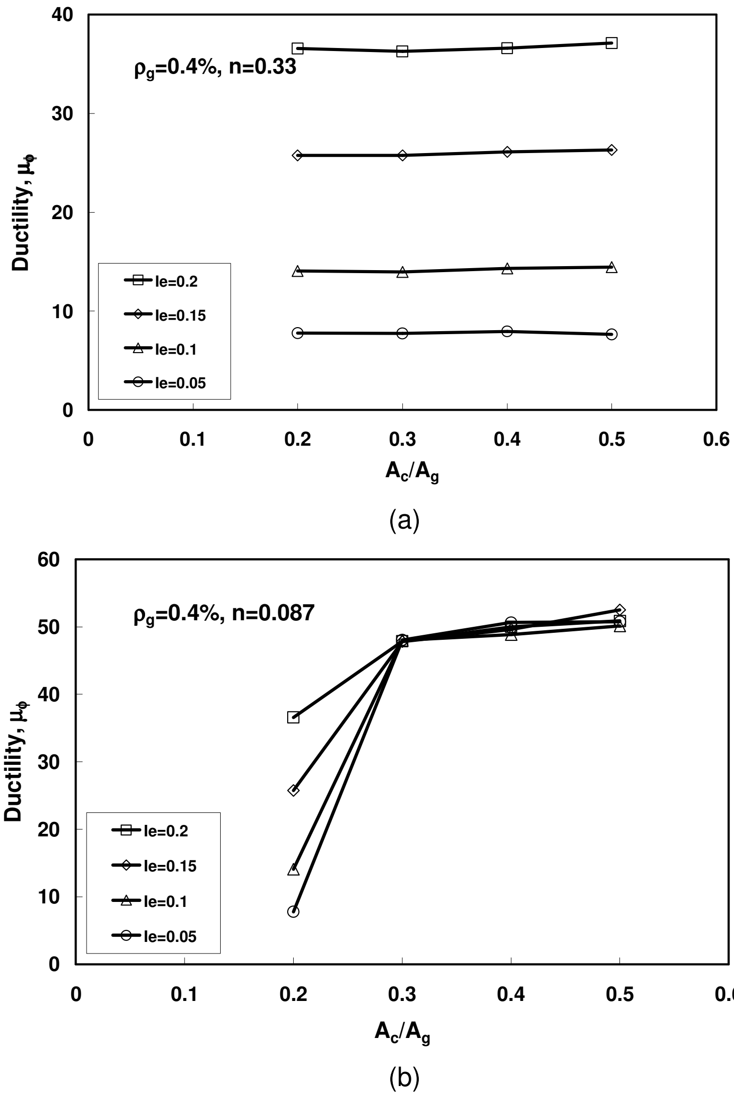

Figure 2. Influence of wall thickness.

In all cases except in the cases of low longitudinal reinforcement and low axial load level with $A_c/A_g = 0.2$, the neutral axis stays in the concrete and does not pass through the hollow core. Hence, the concrete at the inside face of the tube wall is in tension. As a result, the hollow core does not have significant influence on the ductility capacity of the bridge pier and hence does not need to be confined. This finding is in contrast with the guidelines of the codes. Code recommended confinement reinforcement requirements have further been investigated in the following subsection.

Confinement Reinforcement

Confinement reinforcement requirements specified in American code (AASHTO, 2004) and Canadian code (CAN/CSA-S6-06, 2006) provide uniform confinement regardless of ductility demand. When concrete strength is increased, the amount of confinement reinforcement has to be increased to reach a constant level of ductility for columns subjected to same level of axial load. This high amount of lateral reinforcement results in congestion of reinforcement cages and creates concreting problems, specifically in hollow core concrete bridge piers where it acts as a limiting factor for thickness of the wall. Recent research investigation at Sherbooke University on confinement reinforcement for bridge piers has resulted in new confinement equations (Légeron et al., 2006):

$$\rho_s = 0.48\frac{f_c'}{f_y}n \quad \text{(circular columns: moderate ductility)} \tag{1}$$

$$\rho_s = 0.54\frac{f_c'}{f_y}n \quad \text{(circular columns: ductile)} \tag{2}$$

$$A_{sh} = 0.23cs\frac{f_c'}{f_y}n \quad \text{(rectangular columns: moderate ductility)} \tag{3}$$

$$A_{sh} = 0.30cs\frac{f_c'}{f_y}n \quad \text{(rectangular columns: ductile)} \tag{4}$$

where ρ_s is the ratio of spiral reinforcement at plastic hinge region; A_{sh} is the total cross-sectional area of the transverse reinforcement at plastic hinge region; f'_c is the specified compressive strength of concrete; f_y is the yield strength of reinforcing bar; n is the axial load ratio; s is the vertical spacing of the transverse reinforcement; and c is dimension of the tied pier.

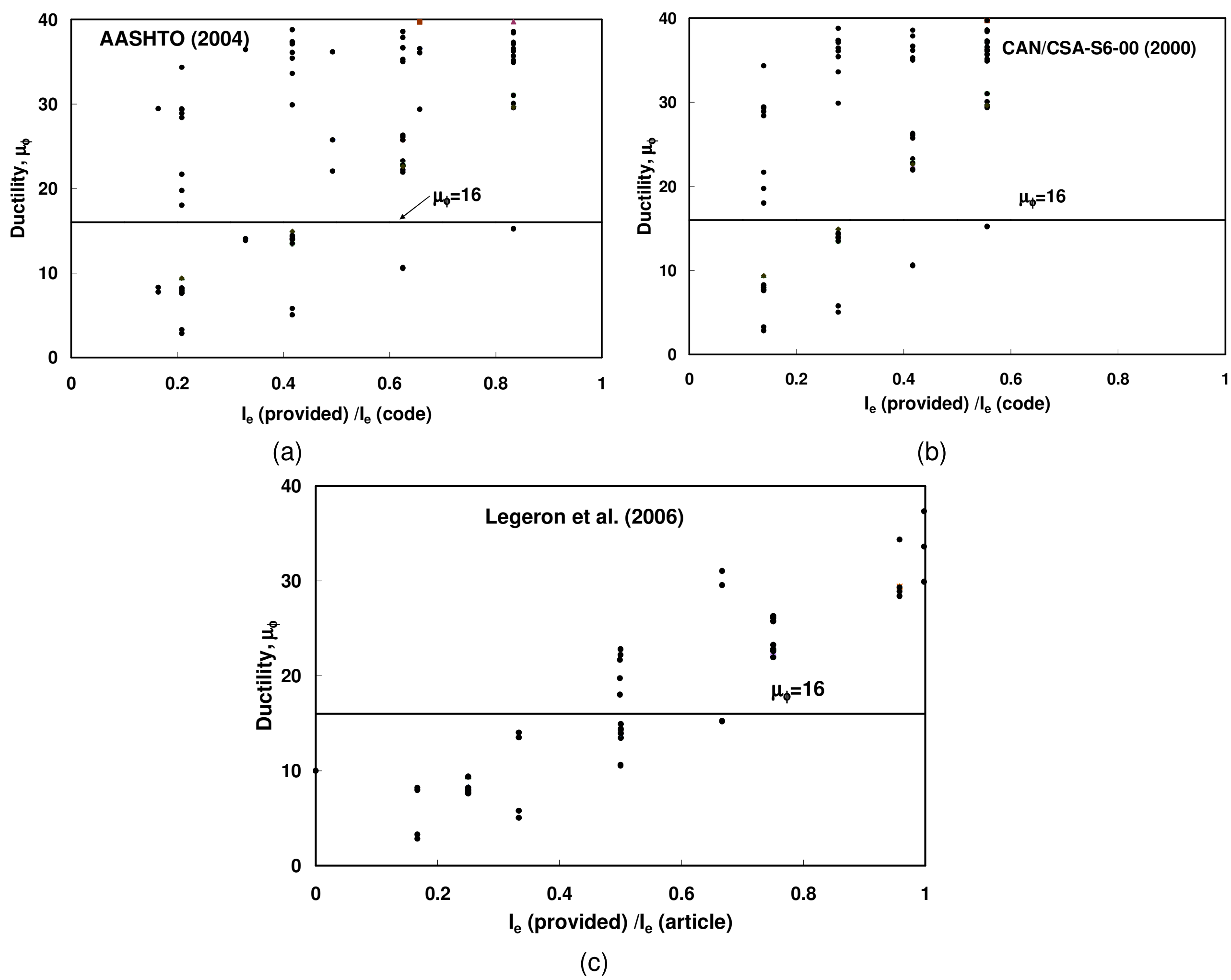

Figure 3. Comparison of confinement reinforcement requirements.

The proposed equations provide more economic and safer design and are considered as a significant improvement over the current design code provisions and expected to be included in the future Canadian highway bridge design code. The proposed equations take into account of the levels of ductility (moderate ductility level and fully ductile level) of the piers. Curvature ductility for moderate ductility level and fully ductile level has been considered as 10 and 16, respectively.

Confinement reinforcement requirements of American and Canadian design codes are compared with available ductility capacity of the piers. Theoretically, there should be some relationship between compliance to code requirement and available ductility. As demonstrated in Fig. 3(a,b), no real tendency has been observed in confinement reinforcement requirements specified in the codes. This means that some piers designed with the codes behave in a ductile manner that is well beyond what is necessary (confinement reinforcement could be cut by 2 or even 3 times), and some other piers, designed with the same procedure, do not achieve the required ductility. Hence, design codes do not provide consistent confinement reinforcement for hollow core piers. However, newly proposed confinement equations better represent the actual requirements (Fig. 3c). It can be observed that about 75% of the confinement reinforcement is utilized for curvature ductility demand of 16. Hence the newly proposed confinement reinforcement equations provide economic and safe results even for hollow core bridge piers. It should be noted that in few cases where neutral axis passes through the hollow core, the newly proposed confinement equation should be used with caution.

Example Wrath Bridge

The suitability of proposed equations has been investigated by redesigning piers of an existing bridge, the Wrath Bridge in Austria, and evaluating its behaviour. It is composed of two identical viaducts and is located on Motorway A23 (Fig. 4). Only one of the viaducts has been studied. A complete numerical analysis of the viaduct can be found in Legeron (2000). The piers are of rectangular cross-section having external dimensions of 6.8 x 2.5 m with a hollow core of 5.8 x 1.9 m. The viaduct is constituted of 5 spans of 67 m and two lateral spans of 62 m. The heights of the piers vary from 17.8 m to 40.0 m, and the aspect ratios vary from 2.06 to 5.9.

Design and Modelling

Seismic loads are determined according to the recommendations of Canadian bridge design code (CAN/CSA-S6-06, 2006), considering the bridge as ‘Other Bridge’ (Importance factor I=1.0). Confinement reinforcements are designed according to the recommendation of Legeron et al. (2006). Cross-sectional dimensions of the piers have been kept the same as the original bridge. The piers are redesigned for zonal acceleration ratio (A) of 0.4, soil profile type III (site coefficient S=1.5), and response modification factor (R) of 3.0. A complete calculation can be found in Vivier (2006).

Modelling of the bridge piers has been carried out according to the methodology developed in an earlier section. Effectiveness of the distribution of confinement reinforcement is taken into account through the calculation of effective confinement index (I'_e). P-Δ effects have also been taken into consideration. The pushover analysis is conducted in order to find out the failure mechanisms and to compute the vulnerability functions with an in-house computer program (RITA) developed at Sherbrooke University. Pier behaviour is assumed to be tri-linear with the three points defining the curve being cracking, yielding and rupture. The response of the bridge under the unit peak ground acceleration is scaled from 0 to rupture for this purpose. For each of the ground accelerations, the structure is considered as a single-degree-of-freedom system with generalized coordinates. The effective structural characteristics are calculated from each element secant characteristics: (i) the generalized coordinate is computed from the deformed shape of the deck, which is determined based on relative pier stiffness, (ii) the displacement for each pier is computed from the assumed deformed shape of the deck, (iii) the secant stiffness of each pier is represented by tri-linear curve, (iv) the equivalent viscous damping is evaluated from hysteretic damping, (v) secant characteristics of the pier is updated for convergence (v) the effective characteristics (stiffness and damping) of the piers are computed, (vi) period of the bridge is evaluated from the effective bridge properties, (vii) generalized acceleration is computed, (viii) peak ground acceleration is incremented up to failure of the pier. Vulnerability functions of the piers are represented as a function of Δ/Δ_u, where Δ_u is the ultimate displacement.

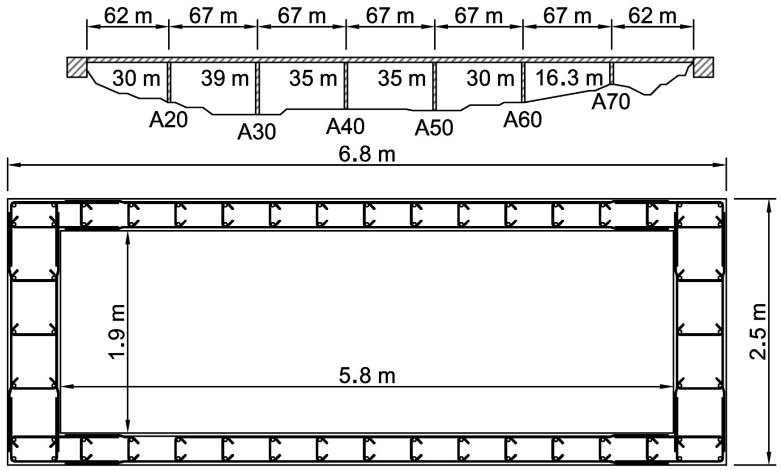

Figure 4. Wrath Bridge in Austria: elevation and pier cross section

Results

Curvature ductility (μ_ϕ) of the piers has been determined to be around 21 and the displacement ductility (μ_Δ) of the piers to be around 6.0. This confirms the suitability of the newly proposed equation for the design of hollow core bridge piers with predictable ductility capacity. Vulnerability functions of the bridge piers are presented in Fig. 5. It can be observed that although the bridge piers are designed for the acceleration coefficient of 4.0 (i.e. peak ground acceleration= 4.0 m/s/s), the Δ/Δ_u value ranges from 0.12 to 0.35. This may be due to the low R used for the design of the piers. It should be mentioned that Canadian code (CAN/CSA-S6-06, 2006) does not treat hollow core bridge piers separately from solid piers: it specifies similar R factor for both solid piers and hollow core piers. Some additional calculations show that response modification factors up to 5 could be used for hollow core piers. It is evident from Fig. 5 that hollow core bridge piers are not as vulnerable as it is believed traditionally. Moreover, if properly designed, it can achieve adequate ductility to sustain anticipated displacement demand imposed by design earthquake events.

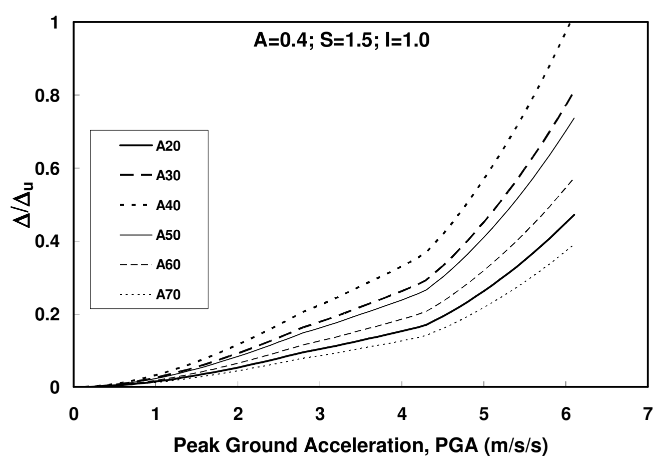

Figure 5. Vulnerability of Wrath bridge piers.

Conclusions

An analytical tool for seismic vulnerability assessment of hollow core bridge piers has been developed. The predictions using this analytical model have been compared with available experimental results. Both flexural and shear behaviour of the piers are evaluated. An excellent agreement between the results of analytical model and results of experimental investigations has been observed.

Ratio of concrete area to the overall cross sectional area (A_c/A_g) has little influence on the ductility capacity of bridge piers. In all the cases, the neutral axis stays in the concrete and never passed through the hollow core. Hence, hollow core does not need to be confined. This investigation is in contrast with the specification of the design codes, which prescribes to confine the hollow core.

Confinement reinforcement requirement in American (AASHTO, 2004) and Canadian (CAN/CSA-S6-06, 2006) codes has been investigated. It has been concluded that the codes are sometimes overly conservative. However, newly proposed confinement equation for bridge piers can well predict the ductility capacity of the hollow core bridge pier and may result in economic and safe design.

Wrath Bridge in Austria has been redesigned according to Canadian code (CAN/CSA-S6-06, 2006) but confinement reinforcement has been considered according to the newly proposed confinement equations for ductile level (Légeron et al., 2006). The bridge is predicted to withstand at least 150% of the design peak ground acceleration, which is satisfactory. It has been demonstrated that hollow core bridge pier is not as vulnerable as it is believed traditionally. If properly designed, hollow core bridge pier can achieve adequate ductility to sustain anticipated displacement demand imposed by design earthquake events. However, this conclusion is based on the result of a single bridge. Research on other bridges with hollow core piers is a part of an ongoing research at Sherbrooke University.

Acknowledgments

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