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Seismic vulnerability of hollow core concrete bridge piers

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SEISMIC VULNERABILITY OF HOLLOW CORE CONCRETE BRIDGE PIER

VULNERABILITE SISMIQUE DES PILES DE PONTS CREUSES EN BETON

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ABSTRACT - Hollow core concrete bridge piers are traditionally believed to be vulnerable to seismic action. However, seismic vulnerability of such piers has not been investigated fully. In this paper, a modeling method to assess seismic vulnerability of hollow core concrete bridge pier is developed. The method 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 pier 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 believed traditionally. Such piers can attain expected ductility, if designed properly.

RÉSUMÉ - Les piles creuses de ponts sont généralement considérées vulnérables aux sollicitations sismiques. Pourtant, il y a peu de recherches qui ont porté spécifiquement sur ces piles. Dans cet article, un modèle analytique pour l'évaluation de la vulnérabilité sismique des piles creuses est présenté et validé avec des expériences. Les recommandations des codes pour les piles creuses sont évaluées. Elles s'avèrent quelquefois très conservatrices, et quelquefois peu sécuritaires. Des équations, récemment développées à l'Université de Sherbrooke pour des piles pleines, sont évaluées pour les piles creuses et donnent des résultats économiques et sûrs. Il est démontré que les piles creuses peuvent se comporter de manière suffisamment ductile si elles sont bien dimensionnées. Elles ne sont donc pas aussi vulnérables que cela est traditionnellement supposé.

1. 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, bride 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 and cable-stayed bridges. 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 behavior of such piers is limited. Even the most modern codes of practice do not recognize specific problems associated with hollow sections, 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 due to their uncertain shear strength and ductility capacity.

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

2. Analytical Model

2.1 Constitutive laws of materials

2.1.1. Stress-strain relationship of concrete

Legeron and Paultre (2003) uniaxial confined concrete model has been chosen as the constitutive law of concrete for the analytical modeling of hollow core bridge pier, as the model is suitable for the whole range of concrete strength. In the model, the behavior of confined concrete is related to the effective confinement index ($I'_e = f_{le} / f'_c$, 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. The model relates the increase of strength and ductility of concrete to the effective confinement index, I'_e, based on the following equations:

$$\frac{f'_{cc}}{f'_c} = 1 + 2.4(I'_e)^{0.7} \tag{1}$$

$$\frac{\varepsilon'_{cc}}{\varepsilon'_c} = 1 + 35(I'_e)^{1.2} \tag{2}$$

$$\frac{\varepsilon_{cc50}}{\varepsilon_{c50}} = 1 + 60 I_{e50} \tag{3}$$

Where I'_e is the effective confinement index at ε'_{cc}, and I_{e50} is the effective confinement index at ε_{cc50}. All other values are fully defined in Figure 1.

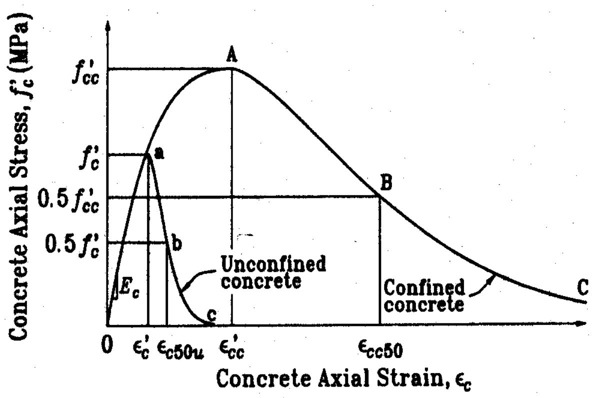


Figure 1. Stress-strain relationships of confined concrete (Legeron and Paultre, 2003)

2.1.2. 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 behavior including buckling of bars in compression, (iii) cyclic behavior, 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 modeling, behavior 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. The equivalent stress-strain relationships of buckling steel, considering the equilibrium of buckled bar between two consecutive ties, is given as:

$$\sigma_s = \frac{2\sqrt{2}M_p}{A_s s}\frac{1}{\sqrt{\varepsilon_s}} \tag{4}$$

where σ_s is the equivalent stress, ε_s is the equivalent strain, s is the spacing of the two consecutive ties, and M_p is the plastic moment of the bar. Neglecting the effect of axial load, the plastic moment of the bar can be expressed as:

$$M_P = 0.424\pi R^3 f_y \tag{5}$$

where R is the radius and f_y is the yield strength of the bar. Axial load does not have significant effect especially at larger strain level (Gomes and Appleton, 1997), and hence the effect of axial load has not been considered in the modeling. The stress-strain relationship for reinforcing bar follows uniaxial stress-strain relationship up to the crossing point with equation 4, and afterward it follows equation 4 (Figure 2).

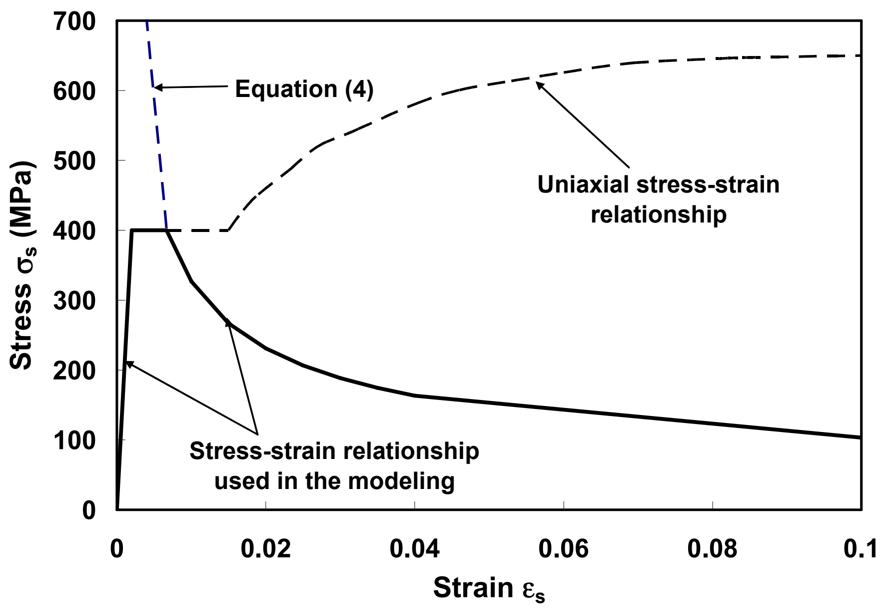


Figure 2. Stress-strain relationship of longitudinal bar

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:

$$\frac{\sigma_s}{f_y} = 1.5 - 0.08\frac{s}{\phi} \tag{6}$$

where ϕ is the bar diameter. It results in an apparent tensile strain at fracture generally varying from 0.03 to 0.06 and is related to the spacing of bars.

2.2. Modeling sectional behavior

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 law of concrete and reinforcing bars, as described above, assuming that plane section before bending remains plane after bending.

2.3. 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 can be determined from moment curvature diagram. It is assumed that average curvature within the assumed plastic hinge length is constant. A computer algorithm has been developed to calculate the flexural force-displacement behavior 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 avove 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.

3. 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) and pier A70 of Pinto et al. (2002) have been reported herein. Full details of all the comparisons can be found in Vivier (2006).

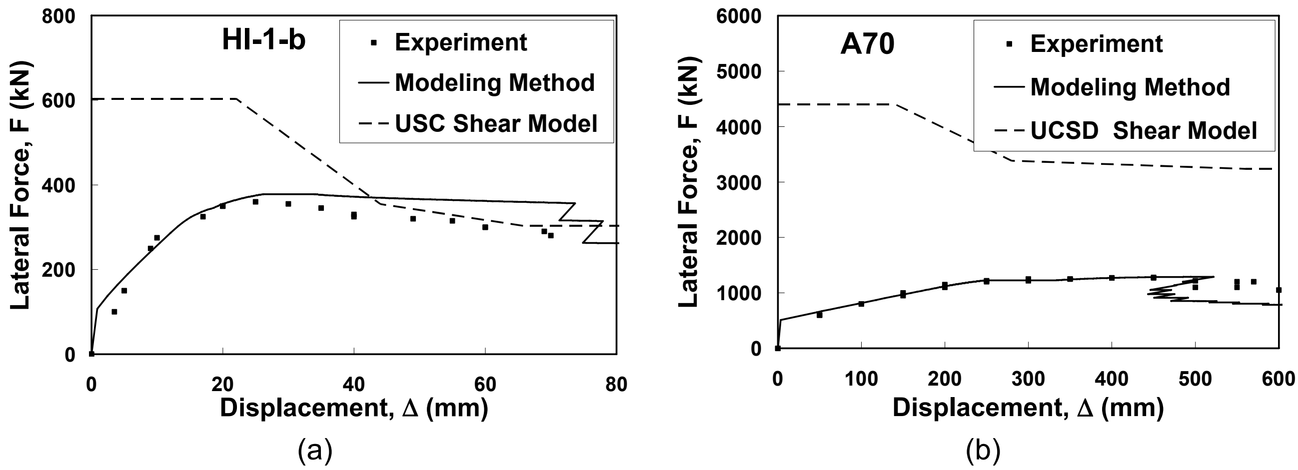


Figure 3. Experimental results compared with analytical predictions

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. The cross-sectional dimension of the pier HI-1-b is 0.5×0.5 m and the wall thickness is 120 mm. The pier is constructed of concrete having compressive strength of 50.5 MPa. 6 mm diameter bar (yield strength=480 MPa) at a spacing of 40 mm has been provided as confinement reinforcement. Pier HI-1-b failed in shear. It can be seen that the modeling method can predict the shear behavior of the pier when shear capacity is calculated based on the phenomenological shear model proposed by Xiao et al (1998) (Figure 3a).

Pinto et al. (2002) presented the results of cyclic tests on two large scale models of Wrath Bridge piers with rectangular hollow cross-section. The cross-sectional dimension of pier A70 is 2.74×2.74 m. The width of the flange and the web are 0.21 m and 0.17 m, respectively. The pier is constructed of concrete having compressive strength of 38.9 MPa. 6 mm diameter bar (yield strength=540.2 MPa) at a spacing of 125 mm at each face of the flange and the web has been provided as confinement reinforcement. Pier A70 is expected to fail in flexure as it is over designed for shear, which is also apparent from the analytical results (Figure 3b).

It is evident from the comparison with the experimental result that the developed modeling method predicts the load-displacement behavior of hollow core pier with reasonable accuracy. It has been observed that the proposed modeling technique can accurately predict the failure modes of the hollow core piers.

4. Evaluation of code recommendations

4.1. 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) (AASHTO, 2004). 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 curvature ductility (μ_ϕ) capacity of hollow core bridge piers for different level of confinement (I'_e). 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 (Figure 4a), except for very low longitudinal reinforcement ratio (ρ_g=0.4%) with very low axial load ratio (n=0.087) (Figure 4b). Even in such a case, the ductility of the piers remains nearly constant when the ratio of wall thickness is more than 0.3, which is normally the case in most hollow core bridge piers.

In all cases except 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 AASHTO (2004). Code recommended confinement reinforcement requirements have further been investigated in the following subsection.

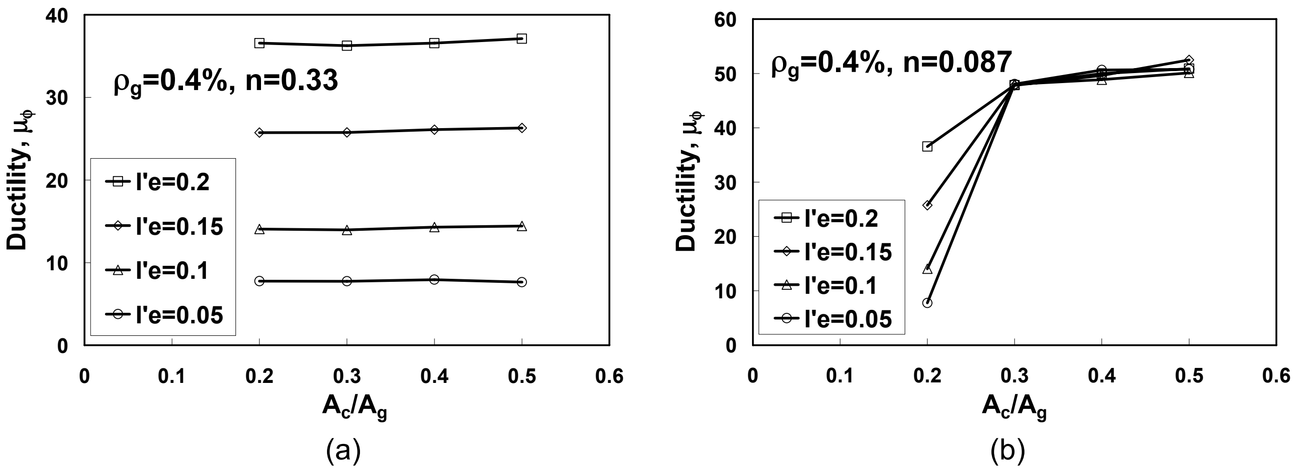


Figure 4. Influence of wall thickness

4.2. Confinement reinforcement

Confinement reinforcement requirements specified in AASHTO code provide uniform confinement regardless of ductility demand and level of axial force. 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 steel results in congestion of reinforcement cages and creates concreting problems. Recent research investigation at Sherbooke University on confinement reinforcement for bridge piers has resulted in new confinement equations (Legeron et al., 2006):

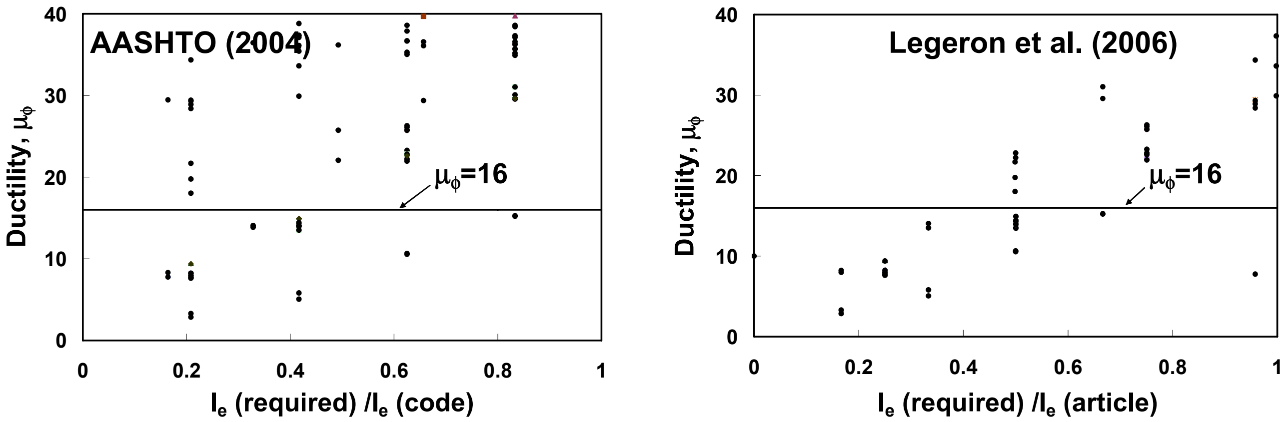
$$\rho_s = 0.48\frac{f'_c}{f_y}n \qquad \text{(circular columns: moderate ductility)} \qquad (7)$$

$$\rho_s = 0.54\frac{f'_c}{f_y}n \qquad \text{(circular columns: ductile)} \qquad (8)$$

$$A_{sh} = 0.23cs\frac{f'_c}{f_y}n \qquad \text{(rectangular columns: moderate ductility)} \qquad (9)$$

$$A_{sh} = 0.30cs\frac{f'_c}{f_y}n \qquad \text{(rectangular columns: ductile)} \qquad (10)$$

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



(a) (b)

Figure 5. Comparisons of confinement reinforcement requirements

Confinement reinforcement requirement in AASHTO (2004) is compared with available ductility capacity of the piers. Theoretically, there should be some relationship between reliance to code requirement and available ductility. As demonstrated by Figure 5a, no real tendency has been observed in confinement reinforcement requirement for American code. This means that some piers designed with AASHTO 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 have the required ductility. Hence, American code does not provide consistent confinement reinforcement for hollow core piers. However, newly proposed confinement equations better represent the actual requirements (Figure 5b). 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 bride piers.

5. Example Wrath Bridge

The suitability of proposed equations has been investigated by redesigning piers of an existing bridge, the Wrath Bridge, and evaluating its behavior. It is composed of two identical viaducts and is located on Motorway A23 in Austria. 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 five spans of 67 m and two lateral spans of 62 m. The heights of the piers vary from 17.8m to 40.0 m, and the aspect ratios vary from 2.06 to 5.9.

5.1. Design and modeling

The piers are redesigned with a response modification factor R=3, according to the recommendation of American Code (AASHTO, 2004). 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 acceleration coefficient (A) of 0.4, soil profile type III (site coefficient of S=1.5), and is considered as part of an ordinary bridge. A complete calculation can be found in Vivier (2006).

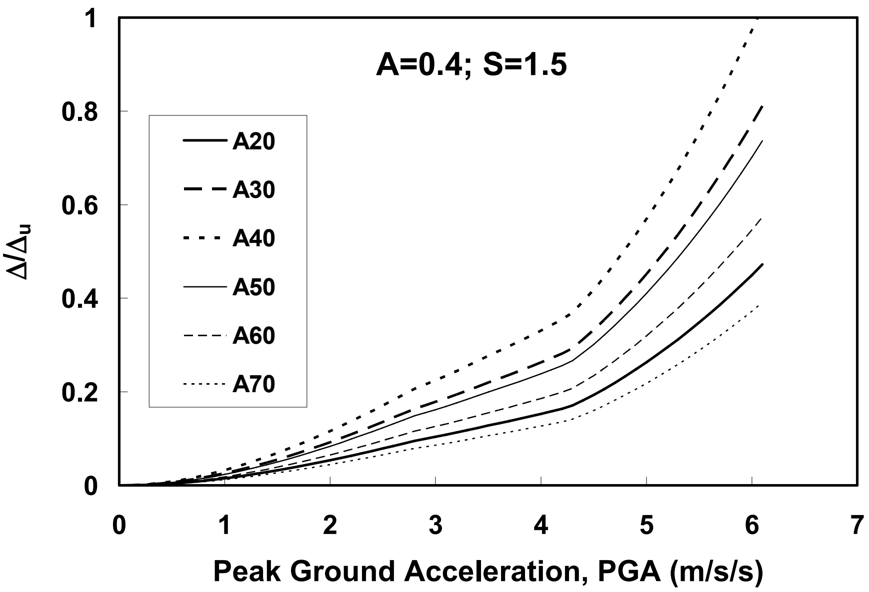


Figure 6. Vulnerability of Wrath Bridge piers

Modeling of the bridge piers has been carried out according to the methodology developed in Section 2. 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 behavior 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. Vulnerability functions of the piers are represented as a function of Δ / Δ_u, where Δ_u is the ultimate displacement where the pier fails.

5.2. Result

Curvature ductility of the piers has been observed to be around 21 and the displacement ductility 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 bride piers are presented in Figure 6. It can be observed that although the bride piers are designed for the acceleration coefficient (A) 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* factor suggested in the code. It should be mentioned that AASHTO (2004) does not treat hollow core bridge piers separately; rather, it specified the same requirement as for solid piers. Some additional calculations show that response modification factors up to 5 could be used for hollow core piers. It is evident from Figure 6 that hollow core bridge pier is 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.

6. Conclusions

An analytical tool for seismic vulnerability assessment of hollow core bridge piers has been developed. The predictions using the developed modeling method have been compared with available experimental results. Both flexural and shear behavior of the piers are evaluated. An excellent agreement between the results of analytical technique 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 AASHTO code, which prescribes to confine the hollow core.

Confinement reinforcement requirement in American code (AASHTO, 2004) has been investigated. It has been concluded that AASHTO (2004) is sometime overly conservative and sometimes non-conservative. However, newly proposed confinement equation for bridge piers can well predict the ductility capacity of the hollow core bridge pier and may render economic and safe design.

Wrath Bridge in Austria has been redesigned according to AASHTO (2004) but confinement reinforcement has been considered according to the newly proposed confinement equations for ductile level. The bridge is predicted to withstand at least 150% of the design peak ground acceleration. This demonstrates 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 ongoing research at Sherbrooke University.

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