



## Shear Capacity Prediction of Precast/Prestressed Hollowcore Slabs

Truderung, Karl<sup>1,4</sup>, El-Salakawy, Ehab<sup>2,5</sup> and Burak, Robert<sup>3,6</sup>

<sup>1</sup> Associate, Structural/Precast, Project Management, Tower Engineering Group, Winnipeg, MB, Canada

<sup>2</sup> Professor, University of Manitoba, Civil Engineering Department, Winnipeg, MB, Canada

<sup>3</sup> President, Canadian Precast/Prestressed Concrete Institute, Ottawa, ON, Canada

<sup>4</sup> [ktruderung@towereng.ca](mailto:ktruderung@towereng.ca)

<sup>5</sup> [Ehab.El-Salakawy@umanitoba.ca](mailto:Ehab.El-Salakawy@umanitoba.ca)

<sup>6</sup> [robert.burak@cpci.ca](mailto:robert.burak@cpci.ca)

**Abstract:** Precast prestressed concrete hollowcore slabs are an economical system for a floor or roof, where uniform load conditions coupled with large spans dictate the design. However, design provisions used for evaluation of shear capacity of precast prestressed members vary depending on the design code used. It is believed that the current North American concrete design codes may result in unduly conservative shear designs for precast prestressed hollowcore slabs. A testing program was undertaken on a total of twenty-four full-scale hollowcore slabs in the 203 mm (8-inch) to 305 mm (12-inch) depth range, from two hollowcore manufacturers, using two types of extrusion machines. The test apparatus for the hollowcore slab specimens used the standardized hollowcore shear test in the European code. The test variables included the length of bearing, the level of prestressing, and the slab depth. Results are presented in terms of predicted capacities and modes of failure. The test results are compared with the predicted shear capacities according to the European, Canadian and American codes. The mechanics of the shear design equations of each code are briefly reviewed, including the effect on shear capacity of horizontal shear stresses within the transfer zone, a design requirement of the European code that is not currently included in the Canadian or American code equations for evaluation of shear capacity.

### 1 Introduction

The Canadian, American, and European concrete design codes and standards all differ on their approach for the shear design of prestressed concrete members. The Canadian Standards, CSA/A23.3-14 (CSA 2014), provide one set of equations for the shear design of prestressed members, based on the response of the member to variable levels of axial strain. The inherent assumption with the Canadian shear resistance equations is that the member is already cracked, and the method is designed to predict the shear resistance after cracking.

The American code, ACI 318 (2014), has two equations used to evaluate the shear capacity of prestressed members, one equation for evaluation of the flexure-shear capacity and a second equation for the evaluation of the elastic web-shear capacity. Hawkins and Ghosh (2006) found that the equations for the web-shear and flexure-shear capacities of prestressed concrete members in the American Code ACI 318 (2005) yielded a conservative design if applied for prestressed hollowcore (PHC) slabs with depths up to 320 mm (12.5 inches), but the equation for web-shear capacity becomes un-conservative for PHC slabs exceeding 320 mm (12.5 inches) in depth. To address these concerns, ACI 318 (2008) introduced a requirement that PHC slabs with a depth exceeding 320 mm (12.5 inches) require minimum shear reinforcement, or that the maximum shear force not exceed half the web-shear capacity.

The European Code, EC2 (2005) is similar to the American Code, ACI 318 (2014), in that there are separate equations used for evaluating the web-shear and flexure-shear capacities of prestressed members. The over-prediction of web-shear capacities for some types of PHC slabs using the European code equations were confirmed by extensive shear tests performed in Europe (Pajari 2005), resulting in separate web-shear equations specific to PHC slabs and a quality assurance test to confirm the as-cast web-shear strength, as noted in EN 1168-2008 (Deutsche Norm 2008). These specific web-shear equations in EN 1168-2008<sup>3</sup> account for the effect on PHC slab shear resistance (positive or negative) resulting from internal horizontal shear stresses within the transfer zone, which are influenced by the geometry of the hollowcore webs, and the strand location within the slab depth.

The design implications of EN 1168-2008 are counter-intuitive to the way PHC slabs have been designed for decades in North America; the Canadian Standard and American Code both recognize the benefit to shear capacity of prestressing (axial compression on the cross section); the implication to the designer is that adding more prestressing strands will always increase the shear resistance. However, both the Canadian and American codes do not capture the effect on shear resistance of internal horizontal shear stresses within the transfer zone, varying web widths over the height of the cross-section (slab geometry), or the amount of prestressing (including multiple layers of strand). Evaluation of the EN 1168-2008 equations for certain types of geometric cross-sections will show that shear resistance cannot always be enhanced without limit by simply adding more prestressing strands; a peak shear capacity may be reached and adding more strand can actually reduce the web-shear capacity. This effect is especially pronounced in deeper sections, where due to width restrictions of the cross-section additional strands are added in a second layer above the bottom row of strands – this second layer may actually decrease the shear resistance of the PHC slab. The EN 1168-2008 captures these very important effects on slab shear resistance. This test program was designed to allow a comparison of the code predicted shear resistances for PHC slabs in the 203, 254 and 305 mm depth ranges by varying the levels of prestressing and length of bearing at the loaded end of support.

## 2 A Comparison of Canadian and International Code Provisions for Shear Design of Prestressed Hollowcore Slabs

### The Canadian Standards CSA/A23.3-14

For a hollowcore slab, the shear resistance is simply based on the concrete contribution alone.

$$[1] \quad V_c = \phi_c \lambda \beta \sqrt{f_c} b_w d_v$$

For design, the material resistance factor,  $\phi_c$  is taken as 0.70 for products manufactured in a certified plant. The term  $\lambda$  is a factor that accounts for low-density concrete,  $b_w$  represents the minimum effective web width and the term  $d_v$  is defined as the effective shear depth, taken as the greater of  $0.9d$ , or  $0.72h$ .

The value of  $\beta$  in Equation [1] is determined as follows:

$$[2] \quad \beta = \frac{0.40}{(1 + 1500\varepsilon_x)} \cdot \frac{1300}{(1000 + s_{ze})}$$

Where  $\varepsilon_x$  is the longitudinal strain at mid-depth of the member due to factored loads (positive when tensile) and  $s_{ze}$  represents an equivalent value of  $s_z$ , which allows for the influence of aggregate size.  $s_z$  is a crack spacing parameter, that is dependent on the crack control characteristics of the longitudinal reinforcement.

The variable  $\varepsilon_x$  is evaluated using the following expression:

$$[3] \quad \varepsilon_x = \frac{M_f / d_v + V_f - V_p + 0.5N_f - A_p f_{po}}{2(E_s A_s + E_p A_p)}$$

For hollowcore slabs, the only reinforcement is the prestressing strands – therefore,  $A_s = 0$ . In addition, the strands are horizontal - therefore  $V_p = 0$  and finally under typical conditions, there is no axial tensile restraint - therefore  $N_f = 0$ . Equation [3] becomes:

$$[4] \quad \varepsilon_x = \frac{M_f/d_v + V_f - A_p f_{po}}{2(E_p A_p)}$$

Where  $M_f$  equals the moment due to factored loads at the considered section,  $d_v$  is the effective shear depth,  $V_f$  is the factored shear force at the considered section and  $V_p$  is the component in the direction of the applied shear of the effective prestressing force.  $N_f$  is the factored axial load normal to the cross-section occurring simultaneously with  $V_f$ , including the effects of tension due to creep and shrinkage (positive for tension).

Finally,  $A_p$  is the total area of tendons on the flexural tension side of the member ( $A_p$  is constant along the member length) and  $f_{po}$  is the stress in the prestressing tendons when the strain in the surrounding concrete is zero (may be taken as  $0.70f_{pu}$  for bonded tendons outside the transfer length, where  $f_{pu}$  equals the specified tensile strength of the prestressing tendon.  $f_{po}$  varies from zero at the end of the slab, to a maximum value at the transfer point.

Evaluation of the 2014 Canadian code (CSA/A23.3-14) expressions for shear resistance of PHC slabs with low levels of prestressing results in much smaller values, in comparison with the previous 1994 Canadian code expressions (CSA/A23.3-94). In addition, the code equations imply increased shear capacity with increasing levels of prestressing, independent of the strand position within the slab depth, or the slab geometry.

#### The American Code (ACI 318-14)

In the American code, the shear resistance,  $V_c$ , is taken as the lesser of  $V_{ci}$  (flexure-shear) or  $V_{cw}$  (web-shear). According to the code commentary, flexure-shear cracking is initiated by flexural cracking. When flexural cracking occurs, the shear stresses in the concrete above the crack are increased. The flexure-shear crack develops when the combined shear and tensile stress exceeds the tensile strength of the concrete. The equation used to evaluate the nominal capacity for flexure-shear,  $V_{ci}$ , is specified as follows:

$$[5] \quad V_{ci} = 0.6\lambda\sqrt{f'_c} b_w d_p + V_d + \frac{V_i M_{cre}}{M_{max}}$$

The term  $\lambda$  is a factor that accounts for low-density concrete, while the term  $b_w$  is the minimum web width. In the above equation,  $d_p$ , the distance from the extreme compression fibre to the centroid of the prestressing steel, need not be taken as less than  $0.80h$ .  $V_d$  is the shear force at a section due to unfactored dead load, and  $V_i$  is the factored shear force at a section due to externally applied loads occurring simultaneously with  $M_{max}$ .

Finally, the term  $M_{cre}$  represents the moment causing flexural cracking at a section due to externally applied loads, calculated as:

$$[6] \quad M_{cre} = (I/y_t)(6\lambda\sqrt{f'_c} + f_{pe} - f_d) \quad ]$$

According to the code commentary, web-shear cracking begins from an interior point in a member when the principal tensile stresses exceed the tensile strength of the concrete. The equation used to evaluate the nominal capacity for web-shear,  $V_{cw}$ , is specified as follows:

$$[7] \quad V_{cw} = (3.5\lambda\sqrt{f'_c} + 0.3f_{pc})b_w d_p + V_p$$

Where  $f_{pc}$  is the compressive stress in the concrete (after allowance for all prestress losses) at the centroid of the cross section resisting externally applied loads, or at the junction of the web and flange, when the centroid lies within the flange. Finally,  $V_p$  is the vertical component of the effective prestressing force at a section, which for a hollow-core slab with horizontal strands is equal to zero.

Similar to the Canadian code, the American code equations imply increased shear capacity with increasing levels of prestressing for PHC, independent of the strand position within the slab depth, or the slab geometry.

### The European Code (EC2 2005):

For PHC slabs without shear reinforcement, the shear resistance of the regions cracked by bending shall be calculated using Equation [7].

$$[8] \quad V_{Rd,c} = \left[ C_{Rd,c} k (100 \rho_l f_{ck})^{1/3} + k_1 \sigma_{cp} \right] b_w d \geq (v_{min} + k_1 \sigma_{cp}) b_w d$$

The recommended value for  $C_{Rd,c}$  is taken as  $0.18/\gamma_c$  ( $\gamma_c$  = partial factor for concrete) and the recommended value for  $k_1$  is taken as 0.15. In the above expression,  $f_{ck}$  is the characteristic compressive cylinder strength of concrete at 28 days, and

$$[9] \quad k = 1 + \sqrt{\frac{200}{d}} \leq 2.0$$

$$[10] \quad \rho_l = \frac{A_{sl}}{b_w d} \leq 0.02$$

Where  $A_{sl}$  is the area of the tensile reinforcement, and  $b_w$  is the smallest width of the cross-section in the tensile area. In Equation 2.14,  $v_{min}$  is calculated from the following expression:

$$[11] \quad v_{min} = 0.035 k^{3/2} f_{ck}^{1/2}$$

Finally,  $\sigma_{cp}$ , the compressive stress in the concrete from axial load or prestressing is taken as:

$$[12] \quad \sigma_{cp} = N_{Ed} / A_c < 0.2 f_{cd}$$

Where  $f_{cd}$  is the design value of concrete compressive strength.  $N_{Ed}$  is the axial force in the cross-section due to the loading or prestressing ( $N_{Ed}$  is positive for compression) and  $A_c$  is the area of the concrete cross-section. However, for prestressed single-span hollowcore members without shear reinforcement, the shear resistance of the regions un-cracked by bending, should be calculated with the following expression:

$$[13] \quad V_{Rd,c} = \frac{I b_w(y)}{S_c(y)} \left( \sqrt{(f_{ctd})^2 + \sigma_{cp}(y) f_{ctd}} - \tau_{cp}(y) \right)$$

where  $\sigma_{cp}(y)$  (positive if compressive) is evaluated from:

$$[14] \quad \sigma_{cp}(y) = \sum_{i=1}^n \left\{ \left[ \frac{1}{A_i} + \frac{(Y_c - y)(Y_c - Y_{p_i})}{I} \right] P_i(l_x) \right\} - \frac{M_{Ed}}{I} (Y_c - y)$$

$$[15] \quad \tau_{cp}(y) = \frac{1}{b_w(y)} \sum_{i=1}^n \left\{ \left[ \frac{A_c(y)}{A_i} - \frac{S_c(y)(Y_c - Y_{p_i})}{I} + C_{p_i}(y) \right] \frac{dP_i(l_x)}{d_x} \right\}$$

Where  $I$  is the second moment of area of the cross-section;  $b_w(y)$  is the web-width at height  $y$ ;  $Y_c$  is the height of the centroidal axis;  $S_c(y)$  is the first moment of the area above height  $y$  and about the centroidal axis;  $y$  is the height of the critical point on the line of failure;  $l_x$  is the distance of the considered point on the line of failure from the starting point of the transmission length (=  $x$ );  $\sigma_{cp}(y)$  is the concrete

compressive stress at height  $y$  and distance  $\ell_x$ ;  $n$  is the number of tendon layers;  $A_i$  is the fictive cross-section surface;  $P_t(\ell_x)$  is the prestressing force in the considered tendon layer at distance  $\ell_x$ ;

$M_{Ed}$  is the bending moment due to the vertical load;  $\tau_{cp}(y)$  is the concrete shear stress due to the transmission of prestress at height  $y$  and distance  $\ell_x$ ;  $A_c(y)$  is the area above height  $y$ ;  $Cp_t(y)$

is a factor taking into account the position of the considered tendon layer;  $Cp_t = -1$ , when  $y \leq y_{P_t}$ ;  $Cp_t = 0$ , when  $y > y_{P_t}$ ;  $y_{P_t}$  is the height of the position of the considered tendon layer.

The main difference between the European and American code expressions for web-shear is that the European expression accounts for the internal shear stresses within the strand transfer zone, which can either increase or decrease the shear resistance depending on the slab geometry and height of strands within the slab depth. The implication is that increasing levels of prestressing do not necessarily result in increased shear resistance. The definitions for the terms in the above equations are provided below:

### 3 Details of the Experimental Program

A total of twenty-four PHC slabs were tested in shear until failure (Celal 2011, Truderung 2011). The PHC slabs were provided by two suppliers, eighteen of them from one supplier and six slabs from the other. The slabs were labelled according to their producer, the level of prestressing and the length of bearing used for the test. For example, slabs produced from the first or second manufacturer were labelled as "P1" or "P2", respectively. Slabs designated with a number (200-01A for example) were also produced by the first manufacturer (P1). The level of prestressing in the test slabs was varied to reflect low (minimum), medium, or high (maximum) prestressing levels (jacking force/slab area), used by the PHC suppliers respectively. The last letter of each slab name represents the length of bearing (measured from the end of the slab to the face of the bearing pad); "A" denotes 63 mm and "B" denotes 38 mm.

A schematic of the test slab cross-sections is shown below in Figure 1. Slabs designated -01A or -01B only had 4 strands representing the minimum prestressing level used by Producer 1, one strand in the outer webs and a strand in every second interior web. Slab designated -18A, -18B, -20A, -20B represented the maximum prestressing level used by Producer 1. Table 1 shows the test-matrix for the PHC slabs tested in this paper.

Concrete strengths at the time of test ranged from approximately 60 to 90 MPa. Concrete strength were verified through a combination of standard cylinder compression tests and compression tests of cores taken from the slab specimens.

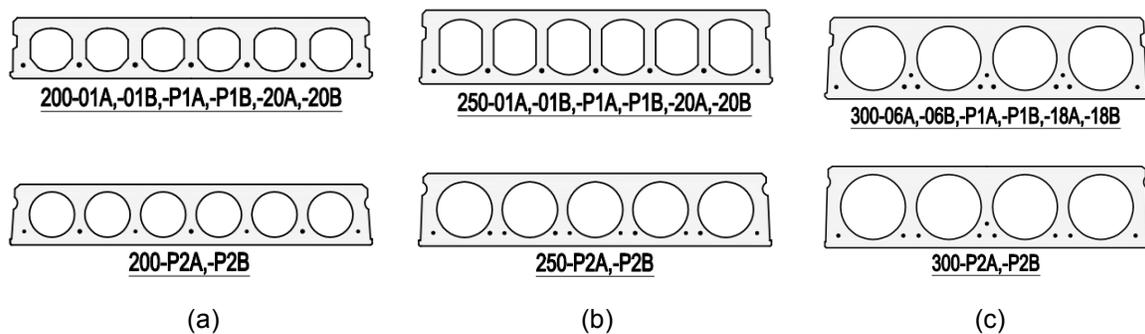


Figure 1 - Test Slab Identification (a) 204-mm slabs, (b) 254-mm slabs, (c) 305-mm slabs

Table 1: Slab Identification

Series	Slab ID	Nominal Slab Depth (mm)	Bearing Length (mm)	Prestress Force/Slab Area at Jacking (MPa)
Series-200	200-01A	203	63	2.18
	200-01B		38	2.18

	200-P1A		63	6.88
	200-P1B		38	6.88
	200-P2A		63	6.87
	200-P2B		38	6.87
	200-20A		63	8.93
	200-20B		38	8.93
Series-250	250-01A	254	63	1.97
	250-01B		38	1.97
	250-P1A		63	6.22
	250-P1B		38	6.22
	250-P2A		63	6.55
	250-P2B		38	6.55
	250-20A		63	8.08
	250-20B		38	8.08
Series-300	300-06A	305	63	3.61
	300-06B		38	3.61
	300-P1A		63	5.13
	300-P1B		38	5.13
	300-P2A		63	5.41
	300-P2B		38	5.41
	300-18A		63	10.21
	300-18B		38	10.21

#### 4 Test Set-Up and Procedure

The test set-up used in this research program followed the guidelines of the standard hollowcore shear test found in Annex J, of European Product Standard EN-1168 (Deutsche Norm 2008). A schematic elevation of the typical test set-up is outlined in Figure 2. The test slabs were made up of full-width elements with a nominal slab length of 4000 mm for both the 203 and 254 mm deep slabs, and a nominal slab length of 4575 mm for the 305 mm slabs.

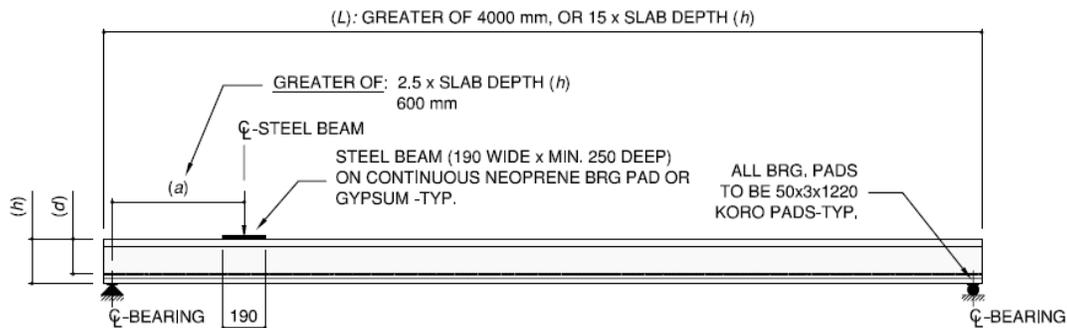


Figure 2 - Elevation of Typical Test Set-Up for Full Scale Shear Test

An important parameter for shear tests is the shear span, or shear span-to-depth ratio ( $a/d$ ). Slender shear spans with an  $a/d$  ratio between 2.5 and 6.0, typically fail in shear at the inclined cracking load. To ensure the observed test loads at failure are not falsely increased by the beneficial effects of arching action between the load and the support, an  $a/d$  ratio larger than 2.5 was used for the test set-up. The length of bearing (defined as the distance from the end of the slab to the face of the 50 mm wide bearing pad) at the loaded end was 63 mm for slabs in series “A” (to represent the standard detailed bearing length) and 38 mm for slabs in series “B” (to represent potential reduced bearing on-site).

The test slabs were initially loaded up to 70% of the predicted failure load for two successive cycles, while the load was increased up to failure during the third (final) cycle. A 5000-kN MTS testing machine was

used to apply the load under a load-controlled rate of 20 kN/min. A summary of the statistical comparison of the code predictions is presented in Table 2 below.

Table 2: Comparison of the code predictions

	V-Exp/V-CSA	V-Exp/V-ACI	V-Exp/V-Euro
Mean (203 mm slabs)	1.49	0.99	0.86
Std. Deviation (203 mm slabs)	0.30	0.21	0.19
Mean (254 mm slabs)	1.62	1.23	1.07
Std. Deviation (254 mm slabs)	0.35	0.22	0.19
Mean (305 mm slabs)	1.25	1.11	0.96
Std. Deviation (305 mm slabs)	0.26	0.33	0.25
Mean (All slabs)	1.39	1.14	1.03
Std. Deviation (All slabs)	0.32	0.26	0.21

## 5 Test Results and Analysis

Both flexure-shear and web-shear failures were observed. A comparison of test results for each code are presented below. Figure 3 shows the experimental versus code predicted shear capacities for each test slab, for all three tested depths

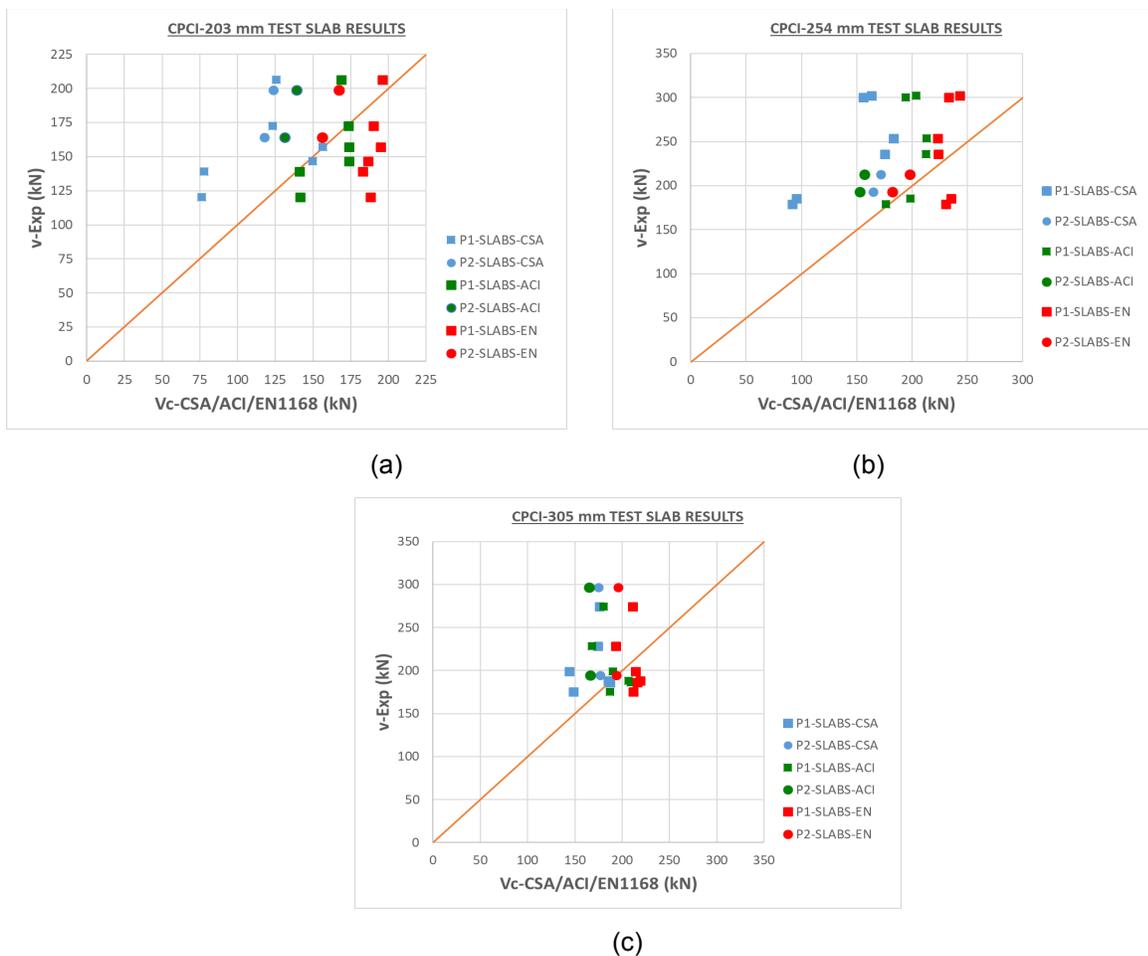
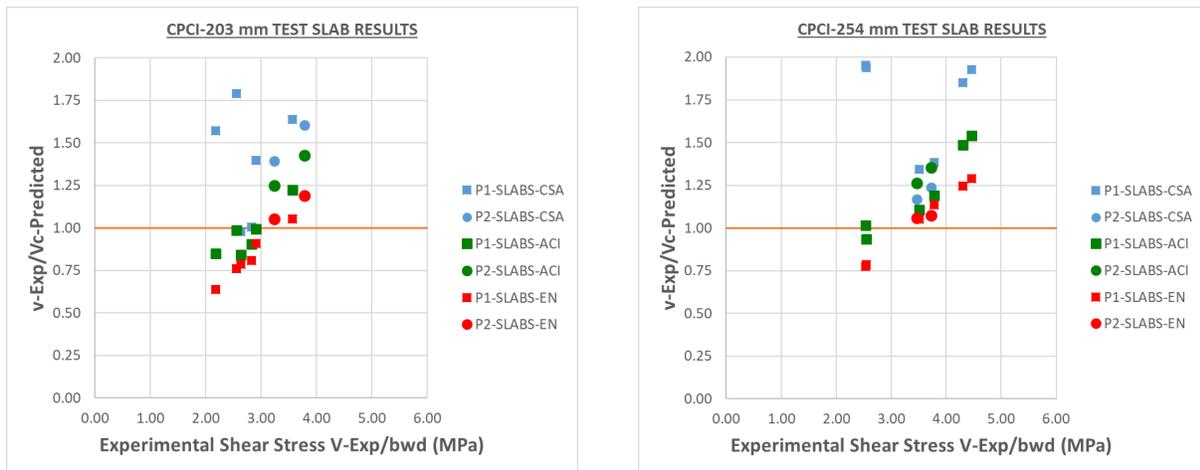


Figure 3 – Experimental ( $V_{Exp}$ ) versus Code Predicted Shear Capacity ( $V_{c-Code}$ ), (a) 203 mm Slabs, (b) (254 mm Slabs), (c) 305 mm Slabs)

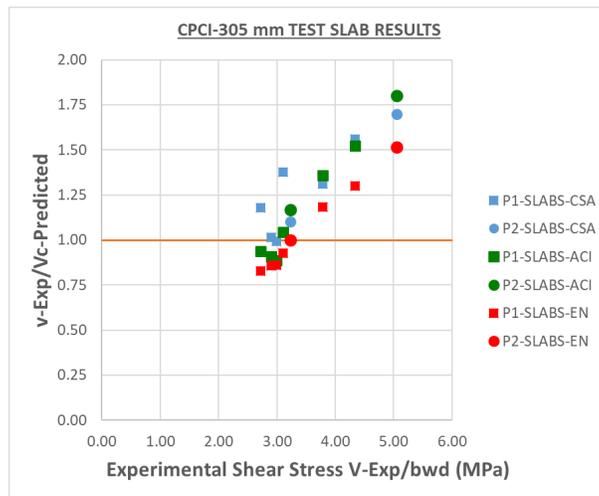
From Figure 3 and Table 2, it can be seen that the Canadian standards is the most conservative in its predictions of shear capacity, and the level of conservatism is not uniform. Given its simplicity, the ACI code is a very good predictor of the shear capacity for the tested slabs, with conservatism in most cases. The European code is similar to the ACI in the pattern of experimental to predicted shear capacities, except the results plotted closer to unity compared to the other codes. In addition, the European code had some unconservative results. In terms of statistical comparisons, the European code had the closest mean experimental-to-predicted results of 1.03, and the smallest standard deviation on the experimental-to-predicted results of 0.21 based on the results of the twenty-four test slabs. Statistically the coefficient of variation is similar for all three which indicates consistency in testing.

To account for variations in the web-widths and slab depths, the test results were normalized by calculating the experimental shear stress for each slab. This is presented in Figure 4. In general, the plotted test results observed when comparing the ACI and Eurocode codes indicate similar patterns, especially for the 305 mm test slab, where the Canadian code also follows a similar pattern. However, it is clear that there is a high level of conservatism in the Canadian code for shear tests of 203 mm and 254 mm slabs with low levels of prestressing. Additional trends in the code predicted comparisons for shear resistance can be observed by plotting the experimental and code predicted shear stresses, as shown in Figure 5.



(a)

(b)



(c)

Figure 4 – Experimental versus Code Predicted Shear Capacity as a Function of Experimental Shear Stress, (a) 203 mm slabs, (b), 254 mm slabs, (c) 305 mm slabs

It can be seen from Figure 5 that the Canadian code predicts increased shear resistance with increase levels of prestressing (from the same producer), and the ACI code also predicts the same but with less of an increase. However, the predicted shear capacities of the European code are almost the same for all levels of prestressing. The results consistently indicate that the peak shear resistances occurred with the slabs that had mid-levels of prestressing for all slab depths, not at maximum levels of prestressing as the Canadian and American codes imply. In addition, the effect on shear capacity of reduced bearing length (slabs with the designation “B”) is not very significant, compared to slabs with standard bearing (with the designation “A”).

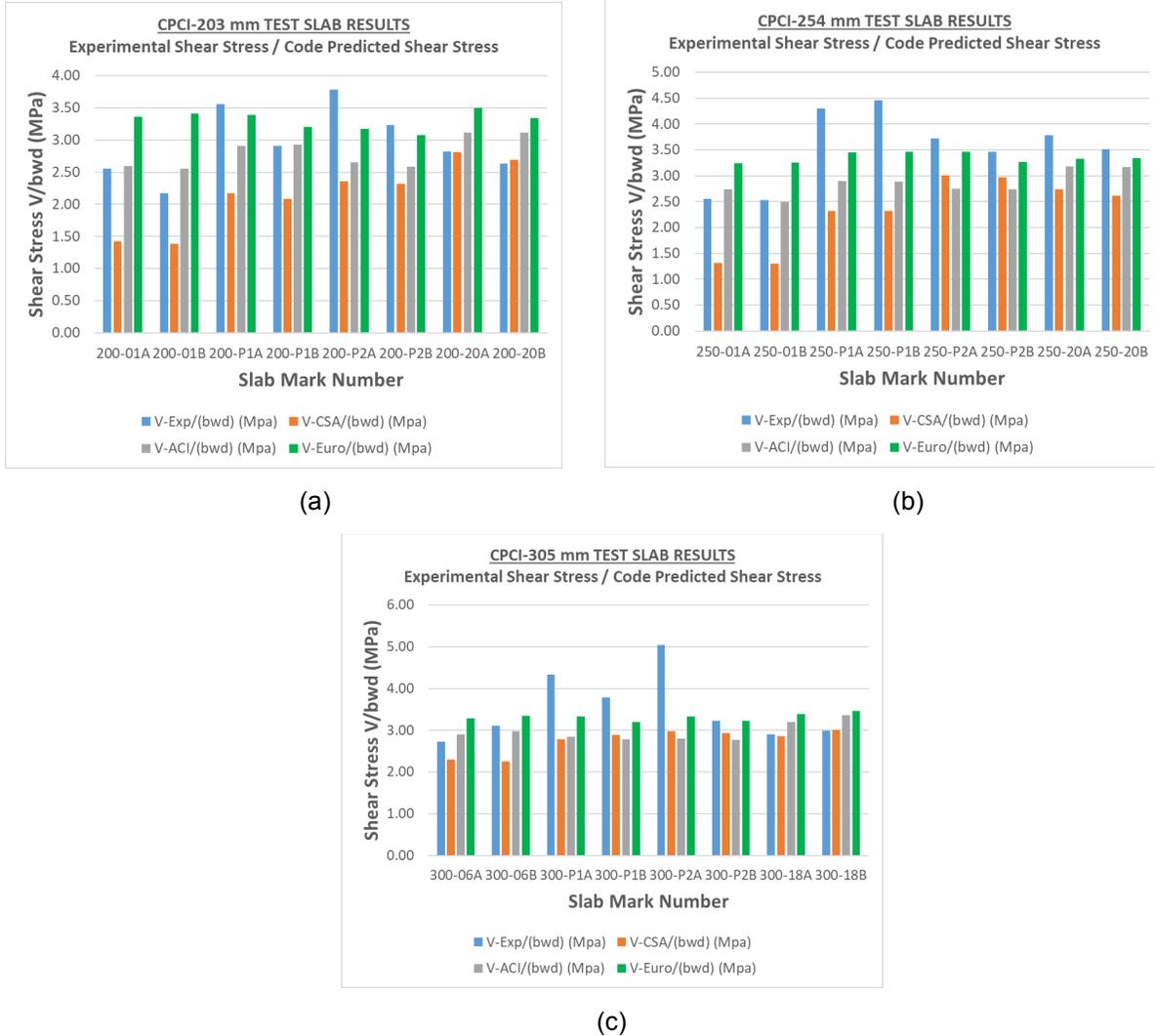


Figure 5 – Experimental & Code Predicted Shear Stresses, (a) 203 mm slabs, (b) 254 mm slabs, (c), 305 mm slabs

## 6 Conclusions

Based on the performed tests and analyses, the following conclusions can be drawn:

1. It has been demonstrated that the traditional North American approach to design of PHC slabs for web-shear resistance is missing some important variables that affect the slab capacity; namely

the effect of horizontal internal shearing stresses within the transfer zone, the geometry of the slabs over the depth and the vertical location of the strands; more prestressing does not necessarily result in increased web-shear capacities.

2. In spite of the fact that the Canadian Standard<sup>1</sup> is based on a post-cracking shear capacity model, it was a reasonable predictor of the shear resistance. In general, the Canadian Standard was conservative for all tested slabs, however the level of conservatism is not uniform. Further calibration of the code equations for shear are required to correct this under-prediction of shear capacity for slabs with lower levels of prestressing.
3. Given its simplicity, the American code equations for shear resistance are a good predictor of shear capacity for the range of slab depths, level of prestressing and bearing lengths tested.
4. The European EN-1168 Code was also a good predictor of shear capacity, however in some cases the predicted results were unconservative. In addition, the predicted shear capacities were not much different when the level of prestressing was varied from low to high amounts.
5. In terms of statistical comparisons, the European code had the closest mean experimental-to-predicted results of 1.03 and the smallest standard deviation on the experimental-to-predicted results of 0.21 based on the results of the twenty-four test slabs. Statistically the coefficient of variation is similar for all three which indicates consistency in testing.
6. The test results clearly indicated larger experimental shear resistances for slabs with medium levels of prestressing, which confirms that more prestressing did not equate to a higher shear capacity for the tested slabs.
7. In spite of the short lengths of bearing used in testing the PHC slabs, all slabs were able to reach their full shear capacity, even with as little as 38 mm of bearing; no localized bearing failures were triggered adjacent to the bearing surface for these test slabs. There weren't significant reductions to the shear capacity even with the reduced bearing lengths.

## Acknowledgements

The authors are grateful for funding and technical support provided by the Canadian Precast/Prestressed Concrete Institute (CPCI), the CPCI Technical Committee, the University of Manitoba Grants Research Program (URGP) and the Natural Science and Engineering Research Council of Canada (NSERC). In addition, the authors would like to thank the hollowcore slab producers and the hollowcore extruder suppliers who contributed towards this research project, as well as the technical staff of the McQuade Heavy Structural Laboratory in the Department of Civil Engineering at the University of Manitoba and Dr. Matti Pajari – VTT Technical Research Centre of Finland.

## References

- ACI Committee 318. (2005). "Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-05)," American Concrete Institute, Farmington Hills, Michigan.
- ACI Committee 318. (2008). "Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08)," *American Concrete Institute*, Farmington Hills, Michigan.
- ACI Committee 318. (2014). "Building Code Requirements for Structural Concrete ACI 318-14 and Commentary ACI 318R-14." American Concrete Institute, Farmington Hills, Michigan.
- CSA. (1994). "Design of Concrete Structures," CSA-A23.3-94, *Canadian Standards Association*, ON, CA.
- CSA. (2004). "Design of Concrete Structures," CSA-A23.3-04, *Canadian Standards Association*, ON, CA.
- CSA. (2014). "Design of Concrete Structures," CSA-A23.3-14." *Canadian Standards Association*, ON, CA.
- Celal, M. S. (2011). "Shear Behaviour of Precast/Prestressed Hollow-Core Slabs," M.Sc. Thesis, Department of Civil Engineering, University of Manitoba, Winnipeg, Manitoba.
- Deutsche Norm. (2008). "Precast Concrete Products–Hollow Core Slabs (includes Amendment A1:2008)." DIN EN 1168-08, *CEN-European Committee for Standardization*, Brussels, Belgium.
- Deutsche Norm. (2005). "Precast Concrete Products–Hollow Core Slabs." DIN EN 1168-05, *CEN-European Committee for Standardization*, Brussels, Belgium.
- Hawkins, N. M. and Ghosh, S. K. (2006). "Shear Strength of Hollow-Core Slabs", *PCI Journal*, Vol. 51, No. 1, pp. 110-114.

- Pajari, M. (2005). "Resistance of Prestressed Hollow Core Slabs Against Web Shear Failure," *Research Notes* 2292, VTT Building and Transport, Kemistintie, Finland, April, 69 p.
- Truderung, K. A. (2011). "Shear Capacity of Dry-Cast Extruded Precast/Prestressed Hollow-Core Slabs," M.Sc. Thesis, Department of Civil Engineering, University of Manitoba, Winnipeg, Manitoba.