

Shear capacity of dry-cast extruded precast, prestressed concrete hollow-core slabs

Karl A. Truderung, Amr El-Ragaby, Mohamed Mady, and Ehab El-Salakawy

- There are currently multiple code methodologies for shear design; however, only the European standard EN 1168 addresses web-shear design specifically for prestressed concrete hollow-core slabs.
- This paper presents the experimental results of eight precast, prestressed concrete hollow-core slabs tested until failure to evaluate their shear capacity against the shear equations in the Canadian standard CSA A23.3-14, the American code ACI 318-14, and the European standard EN 1168.
- EN 1168 was the most accurate predictor of the web-shear capacity of the tested precast, prestressed concrete hollow-core slabs for the range of tested variables, including the amount of prestressing.
- ACI 318-14 also provided accurate predictions for the web-shear capacities of the tested slabs, without the extensive calculations required by EN 1168.

A common structural engineering challenge involves designing a floor or roof system to maximize the span, minimize the overall thickness, and restrict the deflections under service loads to acceptable limits. Precast, prestressed concrete hollow-core slabs offer the designer an economical solution to meet this challenge. Precast, prestressed concrete hollow-core slabs are most often used for floor areas supporting uniformly distributed loads, such as office, school, and residential occupancies (for which service load stresses, deflections, and ultimate moment most often govern the design). Large concentrated loads may also be imposed on precast, prestressed concrete hollow-core slabs, as in the case of steel angle slab hangers framing a floor opening, heavy storage or equipment loads, or in a transfer condition, such as precast, prestressed concrete hollow-core slabs supporting multistory load-bearing walls over a clear-span underground parking structure. In these cases, significant shear forces may govern the precast, prestressed concrete hollow-core slab design, rather than service load stresses, deflections, or ultimate moment.

Precast, prestressed concrete hollow-core slabs are a unique structural element, as they are typically extruded into their final shape by a machine using dry-cast (zero-slump) concrete. Therefore, the addition of shear reinforcement within the concrete is not possible; precast, prestressed concrete hollow-core slabs must resist internal shear forces by using high-strength concrete combined with prestressing strands.

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Despite the widespread usage of precast, prestressed concrete hollow-core slabs in Canadian construction, their shear capacity has not been specifically addressed in the Canadian Standard CSA A23.3-14, *Design of Concrete Structures*.¹ The standard has a general design approach used for shear design of all types of elements. In that approach, the procedure for evaluating concrete shear resistance V_c is based on the modified compression field theory developed by Vecchio and Collins.² This theory has been proved to be a successful model in predicting shear resistance of diagonally cracked reinforced concrete elements subjected to in-plane shear and normal stresses. In other words, the modified compression field theory predicts the shear resistance of already-cracked concrete elements, assuming that a significant amount of shear resistance is developed after cracking. However, precast, prestressed concrete hollow-core slabs with high levels of prestressing won't typically crack in flexure near the support but will most likely fail immediately after the formation of the first diagonal web-shear cracks. However, unlike the model in EN 1168, *Precast Concrete Products—Hollow Core Slabs*,³ the design model given in CSA A23.3-14 does not take into consideration an elastic analysis of web-shear stresses or the significant effect of horizontal shear stresses on shear capacity caused by the variation of prestressing forces in the transfer zone. Also, the existing CSA A23.3-14 model has no methodology to predict and classify the expected mode of shear failure.

A series of eight full-scale hollow-core slabs were tested in shear from two different manufacturers using the same nominal dimensions with varying levels of prestressing and support bearing lengths. The experimental results were compared with the predictions of the Canadian, American, and European concrete design standards and codes. The test results and comparisons emphasized the need to introduce a specific equation for evaluating the elastic web-shear capacity of precast, prestressed concrete hollow-core members into the upcoming edition of CSA A23.3. It is expected that including a web-shear capacity equation together with the current postcracking shear capacity equations would help improve the accuracy in shear strength prediction for such members and provide a more uniform level of conservatism to all slab prestressing levels.

Background

There are a number of possible failure modes that a precast, prestressed concrete hollow-core slab may experience under concentrated loads. If a concentrated load is applied close to the support, it may trigger a flexural crack. Following the initiation of a flexural crack, if the sudden tensile demand on the bonded portion of strands between the crack and the support exceeds the bond resistance of the strands, a strand anchorage failure may occur. Another possible failure mechanism is the formation of a diagonal web-shear crack, which is triggered when the principal tensile stress in the precast, prestressed concrete hollow-core webs exceeds the tensile resistance of the concrete.

If a concentrated load is applied farther away from the support, it may trigger a flexural crack. If the strands are well-anchored to the concrete, a flexural failure may occur through yielding and eventual rupture of the strands.

Because precast, prestressed concrete hollow-core slabs use prestressing to increase the threshold to reach flexural cracking, if a concentrated load is large enough to crack a slab in bending, it follows that there will be a correspondingly high level of shear stress in the vicinity of the initial crack. If the strands are well anchored, the vertical crack will propagate diagonally and form a diagonal flexure-shear crack. In this case, further increasing the load will eventually cause a failure of the uncracked compression flange of the slab, a flexure-shear failure. Evaluation of a single value for shear resistance at a given section of a member, based on the lesser of the calculated design web-shear or flexure-shear capacity, may not always be relevant because it does not simultaneously take into account the other failure modes. For instance, at a given location near the support, if a slab does not crack in flexure under the loading conditions that would initiate a web-shear failure, then it is not possible to achieve a flexure-shear failure and the flexure-shear capacity would be meaningless, even if it happens to be less than the calculated web-shear capacity. In general, an anchorage, flexure, flexure-shear, or web-shear failure is possible for shallow precast, prestressed concrete hollow-core slabs loaded adjacent to the support with low to medium levels of prestressing; however, deeper precast, prestressed concrete hollow-core slabs loaded adjacent to the support with medium to high levels of prestressing will tend to fail in web shear unless the load is far enough from the end, in which case a flexure or flexure-shear failure is also possible.

Current codes and standards

The Canadian, American, and European concrete design standards and codes differ in their approach for the shear design of prestressed concrete members. CSA A23.3-14 provides one set of equations for the shear design of prestressed members. There currently is no independent equation for evaluating the elastic web-shear capacity of a member, only the postcracking model equations, because the code assumption is that the concrete will be cracked in service. However, prestressed hollow-core slabs are designed to be uncracked in flexure under service loads, so the assumption of cracked concrete is very unlikely.

The European code EN 1992:2004, *Eurocode 2: Design of Concrete Structures—Part 1-1: General Rules and Rules for Buildings (EC2)*,⁴ is similar to the American Concrete Institute's (ACI's) *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)*⁵ in that there are separate equations used for evaluating the web-shear and flexure-shear capacities of prestressed members. The overprediction of web-shear capacities for some types of precast, prestressed concrete hollow-core slabs using the EC2 equations was confirmed by extensive shear tests performed by Pajari,⁶ resulting in separate web-shear equations specific to precast, prestressed concrete hollow-core slabs and a quali-

ty assurance test to confirm the as-cast web-shear strength, as noted in EN 1168. These specific web-shear equations in EN 1168 account for the effect on precast, prestressed concrete hollow-core slab shear resistance (positive or negative) due to internal horizontal shear stresses within the transfer zone. In addition, the EN 1168 web-shear equations also account for the effect of varying the strand height (positive or negative) on precast, prestressed concrete hollow-core slab shear resistance. The design implications of EN 1168 are counterintuitive to the way precast, prestressed concrete hollow-core slabs have been designed for decades in North America.

CSA A23.3-14 and ACI 318-14 both recognize the benefit to shear capacity of prestressing (axial compression on the cross section). The implication to the designer is that adding more strand will always increase the shear resistance. However, neither CSA A23.3-14 nor ACI 318-14 captures 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 equations for certain types of geometric cross sections will show that shear resistance cannot always be enhanced without limit by simply adding more strands. A peak shear capacity may be reached, and adding more strand can 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 precast, prestressed concrete hollow-core slab. EN 1168 captures these important effects on slab shear resistance.

A missing but important variable in the current EC2 equations for evaluation of flexure-shear capacity, is that it does not account for the effects of the bending moment applied to the section under consideration, while the ACI 318-14 flexure-shear and CSA A23.3-14 shear equations both account for the bending moment when calculating shear resistance.

Hawkins and Ghosh⁷ found in their review of the ACI shear equations that the equations for the web-shear and flexure-shear capacities of prestressed concrete members in ACI 318-08⁸ yield a conservative design if applied for precast, prestressed concrete hollow-core slabs with depths up to 320 mm (12.5 in.), but the equation for web-shear capacity becomes unconservative for precast, prestressed concrete hollow-core slabs exceeding 320 mm in depth. To address these concerns, ACI introduced a requirement that precast, prestressed concrete hollow-core slabs with a depth exceeding 320 mm (12.5 in.) require minimum shear reinforcement, or that the maximum shear force not exceed half the web-shear capacity.

Research significance

Because shear failure of a concrete member without shear reinforcement is typically sudden and brittle in nature, it is

critical that the actual behavior of such members be understood and accurately predicted in cases where shear strength governs the member design. After reviewing previous research (Hawkins and Ghosh,⁷ Pajari,⁸ Yang,⁹ Micallef,¹⁰ Bertagnoli and Mancini,¹¹ Pajari,¹² Cheng and Wang,¹³ Palmer and Schultz¹⁴), the authors found that there are limited studies regarding the shear capacity of precast, prestressed concrete hollow-core slabs. To date, there is no universally accepted model for precast, prestressed concrete hollow-core slab shear strength prediction.

The aim of this research program is to address this concern by testing a series of full-scale precast, prestressed concrete hollow-core slabs to failure in shear and compare the test results with the predicted capacities based on the Canadian, American, and European concrete design standards and codes.^{1,3,4,5} Specific variables considered in the testing program include the effect of the length of bearing, the level of prestressing, and the variation between slab producers.

Details of experimental program

The precast, prestressed concrete hollow-core slabs tested in this study were cast as part of two local manufacturer's regular production, using a long-line automated slab extrusion process. In this process, the manufacturer casts approximately 150 m (500 ft) of slab per line of form, using a hollow-core extruder. The slabs were cast using zero-slump concrete, covered with an insulated tarp, steam cured, and saw cut after the concrete had reached the strength required to cut the strands, typically 18 hours later. Slabs that shared a common strand pattern were cast successively on the same line of production to ensure uniformity in the concrete for each slab.

Test specimens

A total of eight precast, prestressed concrete hollow-core slabs measuring 4575 mm (15 ft) long, 1216 mm (48 in.) wide, and 305 mm (12 in.) deep were tested until failure. The precast, prestressed concrete hollow-core slabs were provided by two suppliers, six of them from one supplier and the last two from the other. The slabs were labeled 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 labeled P1 or P2, respectively. The level of prestressing used by the precast, prestressed concrete hollow-core suppliers was indicated with L, M, or H, which denote low (minimum), medium, or high (maximum) prestressing level (jacking force/slab area), respectively. The last letter of each slab name represents the length of bearing: A denotes 63 mm (2.5 in.) and B denotes 38 mm (1.5 in.). **Table 1** shows the test matrix for the precast, prestressed concrete hollow-core slabs tested for this paper.

The nominal cross-sectional dimensions of the tested slabs are shown in **Fig. 1**. The as-cast properties of the test slabs, including average height, total web width, slab gross area,

Table 1. Slab identification for tested specimens

Slab	Producer	Level of prestressing		Bearing length*			
		Code	Jacking force/slab area, MPa	Code	Bearing length, mm		
P1-L-A	1	L	3.61	A	63		
P1-L-B				B	38		
P1-M-A		M		6.88	A	63	
P1-M-B					B	38	
P1-H-A		H			10.21	A	63
P1-H-B						B	38
P2-M-A	2	M	6.8			A	63
P2-M-B						B	38

Note: 1 mm = 0.0394 in.; 1 MPa = 0.145 ksi.

* Measured from end of slab to face of bearing pad.

and the gross moment of inertia for each slab, were calculated based on the measured section dimensions. **Table 2** presents an overview of the geometric and section properties for the nominal and actual slabs.

Material properties

Concrete The design concrete strength for the slabs produced by the two manufacturers was 28 MPa (2600 psi) at 18 hours and 45 MPa (6500 psi) at 28 days. For each set of 2 slabs sharing a common strand pattern (level of prestressing), a total of 12 cylinders were cast: 4 were tested at one day, 4 were tested at 28 days, and 4 were tested on the slab test day. Crushed limestone aggregates with a nominal maximum aggregate size of 20 mm (¾ in.) were used for the test slab

concrete. The angular surface profile of the crushed limestone aggregate provides a strong interlock between the aggregates and the cement paste, enabling the use of high-strength concrete for the typical hollow-core slab. **Table 3** provides a summary of all concrete strength values for each tested slab. The compressive strengths on the date of the tests were significantly higher than the 28-day design strengths. This is likely due to the suppliers aiming to ensure they meet the 18-hour strength on a regular basis, which tends to result in exceedance of the design 28-day strength.

Reinforcement The strands used for the test slabs were 9, 13, or 15 mm (0.375, 0.50, or 0.60 in.) seven-wire, 1860 MPa (270 ksi) low-relaxation strands. Figure 1 shows the nominal strand locations.

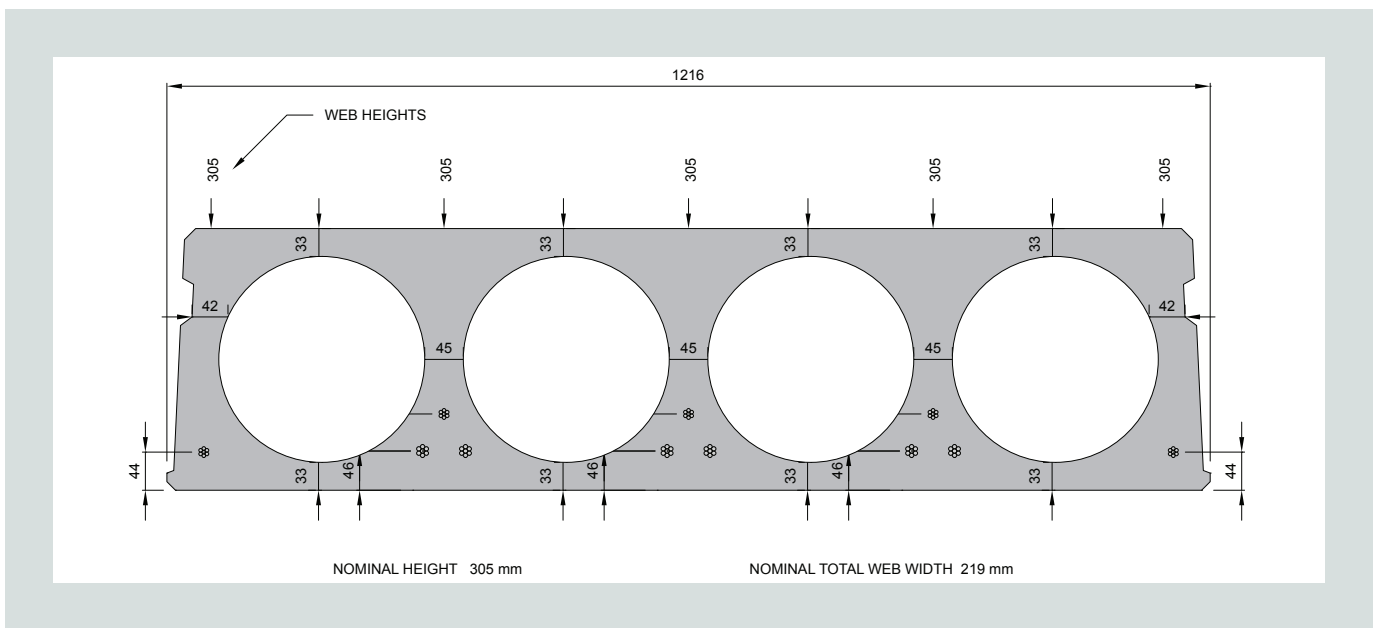


Figure 1. Nominal geometry and reinforcement details. Note: All dimensions are in millimeters. 1 mm = 0.0394 in.

Table 2. As-built slab geometry and section properties

Slab	Average height, mm	Total web width, mm	Slab gross area A_g , mm ²	Gross moment of inertia I_g , mm ⁴
Producer 1	305	219	180,161	2.153E+09
P1-L-A	305	244	200,800	2.280E+09
P1-L-B	305	242	200,128	2.295E+09
P1-M-A	304	252	206,993	2.304E+09
P1-M-B	302	242	203,193	2.284E+09
P1-H-A	303	257	211,552	2.340E+09
P1-H-B	305	247	208,379	2.353E+09
Producer 2	305	231	180,414	2.149E+09
P2-M-A	303	229	188,725	2.180E+09
P2-M-B	305	234	190,843	2.227E+09

Note: 1 mm = 0.0394 in.

Test setup and instrumentation

Because there are no guidelines in CSA A23.3-14 or ACI 318-14 on full-scale shear testing of precast, prestressed concrete hollow-core slabs, the load tests in this research program closely followed the standard hollow-core shear test given in annex J of EN 1168. As a result of concerns about overprediction of web-shear capacity in some cases using the equations given in EC2, the EN 1168 test setup was designed specifically to result in a web-shear failure to allow European producers to verify slab capacities by full-scale testing.

The test slabs were made up of full-width 1220 mm (48.0 in.) elements with a nominal slab length of 4.575 m (15.00 ft), satisfying the condition of 4000 mm (13.00 ft) or 15 times the slab depth, whichever is longer.³ The support conditions emulated a pin at the loaded end of the slabs and a roller at the opposite end so as to eliminate any axial forces that may be caused by rotation and restraint of the slab at the support. The slabs were loaded with a stiffened 254 mm (10 in.) deep I-section steel spreader beam across the full width of the slab, with the load applied at a distance of 763 mm (30 in.) from the centerline of the end support of the slabs (satisfying the condition of 2.5 times the slab depth or 600 mm [24 in.], whichever is greater³).

A 190 mm (7.5 in.) wide steel plate was welded to the bottom flange of the spreader beam to apply a concentrated load across the full width of the slab. A layer of plaster was placed between the slab and the steel plate to prevent load concentration and ensure that the load was uniformly distributed across all webs during loading. **Figure 2** shows a photo of the test setup.

Figure 3 outlines the bearing details at the loaded end of the slabs for the two lengths of bearing tested. The length of

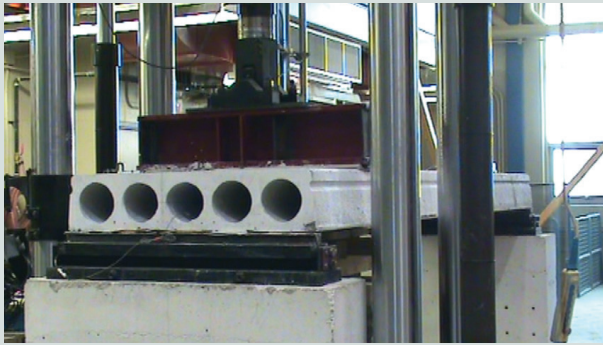
Table 3. Concrete compressive strengths for tested precast, prestressed concrete hollow-core slabs

Slab	Average compressive strength at testing f'_c , MPa	Age at testing, days
P1-L-A	80.0	72
P1-L-B	83.6	86
P1-M-A	67.9	81
P1-M-B	65.7	60
P1-H-A	81.0	55
P1-H-B	87.0	92
P2-M-A	63.2	115
P2-M-B	63.8	101

Note: 1 MPa = 0.145 ksi.

bearing (defined as the distance from the end of the slab to the inner face of the bearing pad) at the loaded end was 63 mm (2.5 in.) for slabs in series A using a continuous 50 mm (2 in.) bearing pad centered on 76 mm (3.0 in.) of slab overlap onto the steel support plate below, representing industry-standard bearing length. A short bearing length of 38 mm (1.5 in.) was chosen for slabs in series B to represent potential reduced bearing on-site using a continuous bearing pad placed on a 38 × 38 mm (1.5 × 1.5 in.) continuous steel bar.

The bearing length at the other end of the slab was 63 mm for all tests, similar to the loaded end of slabs in series A. All results from this testing program are applicable to rigid slab support conditions only. They should not be extrapolated or applied to cases where the supports are flexible, as in a beam.

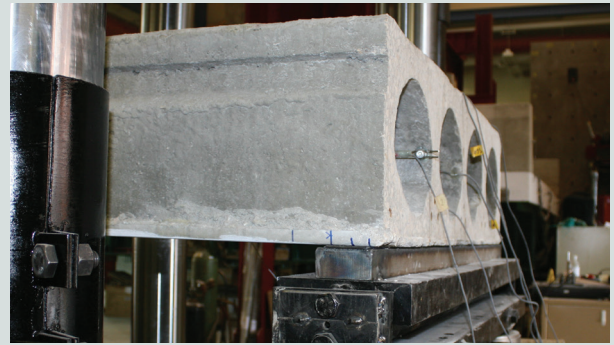


Front view



Side view

Figure 2. Typical setup for hollow-core slab shear tests.



Type A: Standard bearing length, 63 mm



Type B: Reduced bearing length, 38 mm

Figure 3. Bearing details at test end. Note: 1 mm = 0.0394 in.

The end slip of all strands was measured prior to testing and ranged from 1.0 to 3.0 mm (0.04 to 0.12 in.). This magnitude of end-slip did not affect the code-predicted transfer lengths, given that the recorded values were so small.

The slabs were initially loaded up to 70% of the predicted failure load for two successive cycles, while the load was increased to failure during the third (final) cycle, as required by annex J of EN 1168. A 5000 kN (1100 kip) machine was used to apply the load under a load-controlled rate of 20 kN/min (4.5 kip/min).

Test results and analysis

Shear resistance diagrams

Using measurements of the as-built slab geometry and strand locations, spreadsheets were developed to evaluate the predicted capacities as calculated using the Canadian, American, and European design standards and codes.^{1,3,4,5} The spreadsheets were used to develop the shear-force and shear-resistance diagrams for each slab using all three publications. In order to ensure accuracy for comparison of predicted slab capacities, the strand transfer lengths were evaluated for all strand diameters using the detailed equations applicable to each publication, rather than using a simplified assumption for transfer length based on a fixed multiplier of strand diameter. The load was modeled as a 190 mm (7.5 in.) long uniformly distributed line load across the full slab width (rather than a simplified concen-

trated load, to account for the width of the loading beam) and was incrementally increased until the maximum shear force reached the shear capacity of the slabs within the shear span (between the end of the slab and the applied load).

Figure 4 shows an example of the developed charts for slab P1-M-A, which include the theoretical shear force that would initiate a shear failure based on the code equations (due to the effects of the applied load and the member self-weight) and the code-minimum predicted shear resistance using the governing value of the web-shear or flexure-shear resistance, as applicable. The expected failure location denotes the distance measured from the loaded end of the slab to the critical section where failure is predicted to occur according to the considered code/standard.

For the analysis of test slabs, the material resistance factors and strength reduction factors in all equations were set to unity. **Tables 4 to 6** summarize the results of the rest of tested slabs.

CSA A23.3-14 predictions As expected, the code-predicted shear capacities improved with increasing levels of prestressing. The code-predicted shear capacities for slabs P1-L-A and P1-L-B (slabs with the least amount of prestressing) are lowest, and the predicted capacities for slabs P1-H-A and P1-H-B (slabs with the highest amount of prestressing) are highest. CSA A23.3-14 seems to predict almost no effect for bearing length on shear capacities. The reduction in shear resistance was marginal for the slabs with the reduced bearing compared

with those with the full bearing. There was a deviation in effective total web thicknesses between the slabs from different suppliers, which is reflected in the analytical results as well as in the experimental results (Table 4). The concrete strength does not make much difference beyond the concrete compressive strength of 64 MPa (9300 psi), which is a code-imposed upper limit to account for the brittle failure nature of high-strength concrete, in which a shear failure will pass through the aggregates rather than around the aggregates.

ACI 318-14 predictions The ACI 318-14 shear resistance diagram (Fig. 4) was developed based on the minimum envelope for the shear capacities assuming both modes of failure introduced by ACI 318-14: web-shear failure and flexure-shear failure. Accordingly, the location of the point of intersection with the shear force diagram determines the expected failure mode. The ACI 318-14-predicted shear capacities for slabs P1-L-A and P1-L-B (slabs with the least amount of prestressing) are lower than the predicted capacities for slabs P1-H-A and P1-H-B. It was also observed that the ACI 318-14-predicted shear resistance values were significantly higher than the corresponding CSA A23.3-14-predicted values, except for the slabs from the second producer, by a maximum of 24% in the slabs with a low level of prestressing. In addition, the predicted reduction in shear capacity due to reduced bearing is higher than the reduction percentage triggered while using the CSA A23.3-14 equations, but still marginal.

EN 1168 predictions The EN 1168 equations predicted higher web-shear capacities than CSA A23.3-14 and ACI 318-14. However, a noticeable difference is observed when comparing the predicted capacities for slabs with the lowest level of prestressing (P1-L-A, P1-L-B) with the slabs

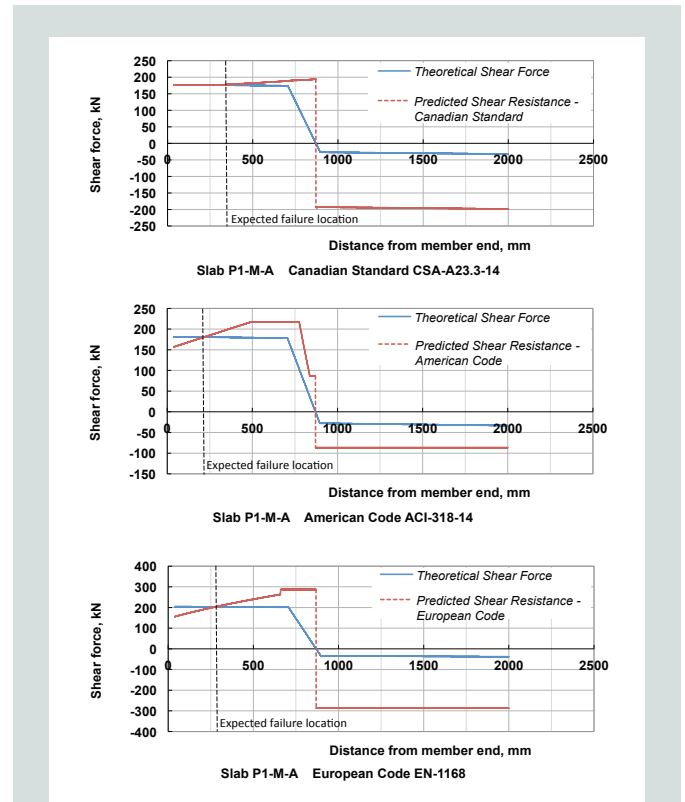


Figure 4. Shear force and predicted shear resistances for slab P1-M-A. Note: 1 mm = 0.0394 in.; 1 kN = 0.225 kip.

with the highest levels of prestressing (P1-H-A, P1-H-B). The predictions were within a maximum spread of 3% and did not show the dramatic predicted increase in shear capacity that CSA A23.3-14 and ACI 318-14 showed for in-

Table 4. Experimental and predicted failure loads using CSA A23.3-14

Slab	Observed failure mode	Predicted failure location,* mm	b_w , mm	V_{exp} , kN	V_{c-pre} , kN	V_{exp}/V_{c-pre}
P1-L-A	Web shear	301	244	175	149	1.18
P1-L-B	Web shear	282	242	199	144	1.38
P1-M-A	Web shear	292	252	275	176	1.56
P1-M-B	Web shear	272	242	228	174	1.31
P1-H-A	Web shear	290	257	188	185	1.02
P1-H-B	Web shear	271	247	186	187	0.99
P2-M-A	Web shear	293	229	297	175	1.70
P2-M-B	Web shear	274	234	194	176	1.10
Mean value						1.25
Standard deviation						0.26

Note: b_w = total web width at the height within the slab depth for which the shear stresses are calculated; V_{c-pre} = code-predicted shear capacity as calculated at the predicted failure location; V_{exp} = calculated shear force at the predicted failure location due to effect of applied load plus self-weight of the slab unit. 1 mm = 0.0394 in.; 1 kN = 0.225 kip.

* Measured from slab end.

Table 5. Experimental and predicted failure loads using ACI 318-14

Slab ID	Observed failure mode	Predicted shear failure mode	Predicted failure location,* mm	b_w , mm	V_{exp} , kN	V_{c-pre} , kN	V_{exp}/V_{c-pre}
P1-L-A	Web shear	Web shear	216	244	175	187	0.94
P1-L-B	Web shear	Web shear	197	242	199	190	1.04
P1-M-A	Web shear	Web shear	216	252	275	180	1.52
P1-M-B	Web shear	Web shear	197	242	228	168	1.36
P1-H-A	Web shear	Web shear	215	257	188	207	0.91
P1-H-B	Web shear	Web shear	197	247	186	209	0.89
P2-M-A	Web shear	Web shear	216	229	297	165	1.80
P2-M-B	Web shear	Web shear	197	234	194	167	1.17
Mean value							1.11
Standard deviation							0.33

Note: b_w = total web width at the height within the slab depth for which the shear stresses are calculated; V_{c-pre} = code-predicted shear capacity as calculated at the predicted failure location; V_{exp} = calculated shear force at the predicted failure location due to effect of applied load plus self-weight of the slab unit. 1 mm = 0.0394 in.; 1 kN = 0.225 kip.

* Measured from slab end.

creasing the strand amounts. Again, the effect of the bearing length resulted in only a marginal difference to the predicted capacities.

It is worth mentioning that in all three considered design standards and codes, the reduced percentages of shear capacity due to reduced bearing length are smaller than the observed reduction in experimentally obtained capacities in most cases (except for slabs P1-H-A and P1-H-B).

Predicted and experimental failure loads

Using the developed spreadsheets, the failure load and failure location based on each design standard and code were predicted (Tables 4 to 6). The following information was included:

- V_{exp} , which is the calculated shear force at the predicted failure location due to effect of applied load P_{Exp} plus self-weight of the slab unit
- V_{c-pre} , which is the code-predicted shear capacity as calculated at the predicted failure location
- V_{exp}/V_{c-pre} , which is the ratio of experimentally observed shear capacity to the code-predicted shear resistance (used as a measure for level of accuracy of the predicted values using code provisions)

CSA A23.3-14 predictions Although the Canadian method is based on a postcracking failure model, CSA A23.3-14 was a reasonably good predictor of slab web-shear capacity for the tested slabs; however, the results were all conservative.

Essentially none of the experimental to predicted capacities were below unity. However, there is a nonuniform level of conservatism being applied to predicted shear capacity of the slabs. CSA A23.3-14 underpredicts the shear capacity for some precast, prestressed concrete hollow-core slabs, especially at the medium level of prestressing (slabs P1-M-A and P2-M-A), where the ratio of experimental to predicted capacity was as high as 1.70. In general, CSA A23.3-14 was conservative for all tested slabs, with an average experimental-to-predicted capacity ratio of 1.25 and a standard deviation of 0.26.

ACI 318-14 predictions Despite its simplicity, ACI 318-14 was a reasonably good predictor of slab web-shear capacity for the tested slabs. In some cases, the ratio of experimental to predicted capacity was below unity, ranging from 0.89 to 1.80, with an average value of 1.11 and a standard deviation of 0.33. ACI 318-14 was successful in predicting the failure mode for all tested slabs.

EN 1168 predictions EN 1168 was also a reasonably good predictor of slab web-shear capacity for the tested slabs. In some cases, the ratio of experimental to predicted capacity was below unity, ranging from 0.83 to 1.51, with an average value of 0.96 and a standard deviation of 0.25. The median of EN 1168 predictions were the closest to unity, with the smallest standard deviation.

It is worth mentioning that the variation in the obtained results can be referred to the nature of the shear strength of the concrete in hollow-core slabs, which is not necessarily homogenous in all directions due to the way these slabs are

Table 6. Experimental and predicted failure loads using EN 1168

Slab ID	Observed failure mode	Predicted shear failure mode	Predicted failure location,* mm	b_w , mm	V_{exp} , kN	V_{c-pre} , kN	V_{exp}/V_{c-pre}
P1-L-A	Web shear	Web shear	281	244	175	212	0.83
P1-L-B	Web shear	Web shear	256	242	199	214	0.93
P1-M-A	Web shear	Web shear	281	252	275	211	1.30
P1-M-B	Web shear	Web shear	256	242	228	193	1.18
P1-H-A	Web shear	Web shear	281	257	188	219	0.86
P1-H-B	Web shear	Web shear	256	247	186	216	0.86
P2-M-A	Web shear	Web shear	281	229	297	196	1.51
P2-M-B	Web shear	Web shear	256	234	194	194	1.00
Mean value							0.96
Standard deviation							0.25
Note: b_w = total web width at the height within the slab depth for which the shear stresses are calculated; V_{c-pre} = code-predicted shear capacity as calculated at the predicted failure location; V_{exp} = calculated shear force at the predicted failure location due to effect of applied load plus self-weight of the slab unit. 1 mm = 0.0394 in.; 1 kN = 0.225 kip. * Measured from slab end.							

extruded using a dry-cast method. Unlike the flexure failure of such slabs, variation of slab capacities in shear failure, which is brittle in nature, is expected and the obtained measurements are reasonable.

Effect of level of prestressing on failure loads Contrary to what would be expected by CSA A23.3-14 or ACI 318-14, slab P2-M-A, with a medium level of prestressing, achieved the highest experimental shear resistance of 297 kN (66.8 kip), far higher than that of the slabs with the maximum number of strands. Another important observation is regarding the experimental shear failure loads for P1 slabs from supplier 1: there is a noticeable peak in tested shear resistances for slabs with medium prestressing levels (P1-M-A and P1-M-B), and the average experimental shear failure load of slabs P1-L-A and P1-L-B (187.5 kN [42.15 kip]) is the same as the average experimental shear failure load for slabs P1-H-A and P1-H-B (slabs with the highest level of prestressing). Slabs P1-H-A and P1-H-B each had a second layer of strands above the bottom layer.

While EN 1168 greatly reduced the spread in predicted web-shear capacities for the full range of prestressing levels, it was surprising to see such a large jump in capacity for slabs with medium levels of prestressing. Nonetheless, the testing program has proved that it is an incorrect assumption that increasing the strand amounts will always increase the web-shear resistance. It is universally accepted that the web-shear resistance is directly proportional to $b_w d$, where b_w is the total web width at the height within the slab for which the shear stresses are calculated and d is the depth of the reinforcing strands measured from the concrete compression fiber. Given

that the slab depth is constant, it would be expected that slabs would have a linear increase in web-shear capacity with increased values for b_w .

A review of the variation in web widths for the P1 slabs cannot explain this observation in the observed range of tested shear forces over the range of prestressing levels. The maximum variation in web widths from the P1 slabs ranges from 2.1% to +3.9% (compared with the median web width for P1 slabs of 247.3 mm [9.736 in.]). Similarly, the P2 slab web widths were actually smaller than the median web width for the P1 slabs (approximately 5.0% to 7.0% less). Finally, the variations in concrete compressive strength for each slab do not explain the observed higher shear capacities for slabs with medium levels of prestressing either because the slabs with the highest web-shear capacities had concrete compressive strengths of approximately 80% of the values of slabs with low and high levels of prestressing for the P1 slabs.

Effect of length of bearing on failure loads As expected, except at the low level of prestressing, reducing the length of bearing resulted in a reduction in the shear capacity. Even with as little as 38 mm (1.5 in.) of bearing there were no premature localized failures adjacent to the support triggered by the reduced bearing length where the slab failed near the bearing end before the full shear capacity of the slab was reached. This observation will be comforting for designers of precast, prestressed concrete hollow-core slabs faced with a potentially reduced as-erected bearing length on-site, when the precast, prestressed concrete hollow-core slabs are bearing on rigid supports, such as a concrete wall.

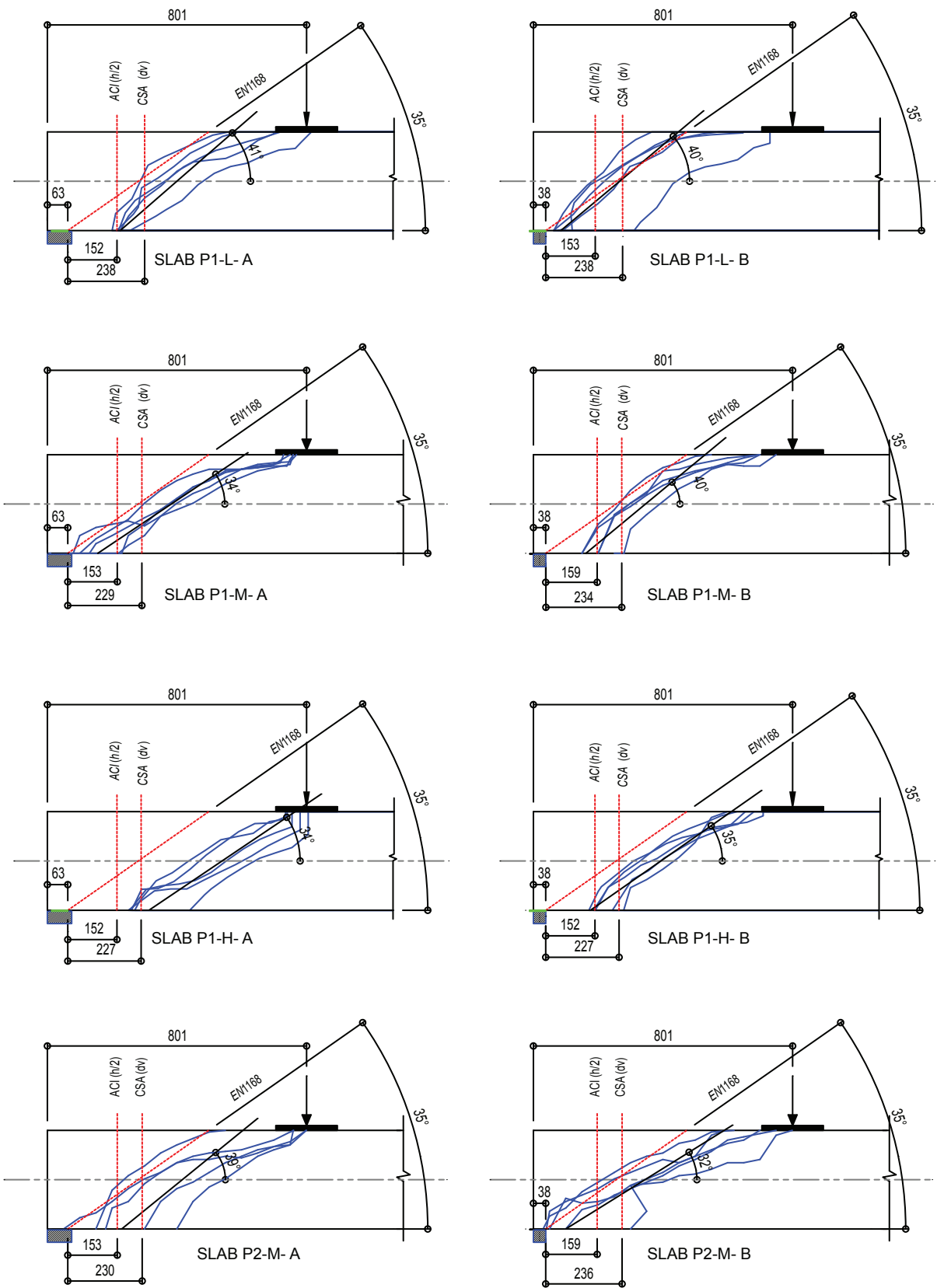


Figure 5. Crack profiles for tested slabs. Note: All dimensions are in millimeters. 1 mm = 0.0394 in.

Slab failure profiles

Predicted and experimental locations of critical section

After each slab was tested, the crushed and spalled concrete of the tested ends was removed to enable measurement of the overall crack profiles at each web. These lines of shear failure for all webs in the slab were accurately measured by vertical and horizontal coordinate points from a datum line. **Figure 5** shows the crack profiles of all webs compiled and plotted on one elevation for comparison of the individual web crack profiles within each test slab. Together with the presented web-shear crack profiles, the specified location of critical section for shear failure according to provisions of each design standard and code was also highlighted on every slab elevation.

In general, EN 1168 was the closest and most realistic in predicting the location of web-shear failure when compared with experimental crack profiles. Uneven or scattered and variable-angle web-shear cracking lines could be an indication of shear lag (the failure of individual webs before or after other adjacent webs in the same slab), which would normally be reflected on a discrepancy in level of measured load at which each web cracked. It is sometimes difficult to classify the type of observed failure for modes other than a web-shear failure, where a combination of flexural, bond, and flexure-shear failure modes across the width of the slab are observed at failure. Another possibility for the location of the observed failure surfaces is the possibility of local variations in concrete compressive strength in a specific web or along the length of the slab. A locally lower value of concrete strength can sometimes trigger the initial failure.

Inclination of web-shear cracks One other observation, which was derived from the crack profiles presented in Fig. 5, was the angle of web-shear failure. The average angles of inclinations were obtained by plotting a straight line from the bottoms of the slabs that is approximately parallel with the slope of all web crack lines at the midheight level of the slabs. The observed crack angles (measured between the plotted cracks and bottoms of the slabs) ranged from 32 to 41 degrees. It should be noted, however, that the observed failure profiles are not one continuous angle but rather a variable angle along each crack. In addition, it is difficult to identify which web triggered the overall failure and some of the variation in the angles and locations of the failure profiles may be due to the rapid redistribution of load to other undamaged webs or possibly to some differential warping between the end supports before loading or even to the bottom of the slab being slightly concave upward at the bearing locations from the shape of steel casting bed.

Effect of level of prestressing The level of prestressing appears to have several effects on the failure profiles. The level of prestressing significantly affects the location of the failure relative to the slab end. Lowering the prestressing level resulted in a shift in the failure profiles away from the load and closer to the end of the slab. Also, the higher the prestressing

level on the slab, the shallower the crack angle observed at failure. CSA A23.3-14 predicted a range of 29 to 30 degrees for the crack angle based on the tested range of prestressing levels, while EN 1168 assumes a crack angle of 35 degrees for all slabs (independent of the prestressing level), which is the close to the average observed crack angle for all slabs of 37 degrees.

Effect of length of bearing The location of the failure profiles seems to be affected by the length of bearing. Slabs with reduced bearing had their shear failure profiles shifted closer to the end of the slab.

Conclusion

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

- It has been demonstrated that the traditional North American approach to the design of precast, prestressed concrete hollow-core slabs for web-shear resistance is missing some important variables that affect the slab capacity: 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.
- Although CSA A23.3-14 is based on a postcracking shear capacity model, it was a reasonable predictor of the shear resistance. In general, CSA A23.3-14 was conservative for all tested slabs, with an average experimental-to-predicted capacity ratio of 1.25 and a standard deviation of 0.26. CSA A23.3-14 does not attempt to predict the shear failure mode for the test slabs.
- In some cases, the ratio of experimental to predicted capacity using ACI 318-14 equations was below unity, ranging from 0.89 to 1.80, with an average value of 1.11 and a standard deviation of 0.33. While EN 1168 correctly captures the mechanics of internal horizontal shear stresses within the transfer zone, variable slab geometry within the slab depth, and the effect of strand height, the additional calculation effort didn't appear to be warranted for the tested slabs compared with ACI 318-14. Given its simplicity, ACI 318-14 was a reasonable predictor of shear resistance.
- EN 1168 was a reasonably good predictor of slab web-shear capacity for the tested slabs. In some cases, the ratio of experimental to predicted capacity was below unity, ranging from 0.83 to 1.51 with an average value of 0.96 and a standard deviation of 0.25.
- Both ACI 318-14 and EN 1168 were successful in predicting the failure mode for all tested slabs.
- Lowering the prestressing level resulted in a shift in the failure profiles away from the load and closer to the end of

the slab. Also, the higher the prestressing level on the slab, the shallower the crack angle observed at failure. In spite of the short lengths of bearing used in testing the precast, prestressed concrete hollow-core slabs, no localized failures were triggered adjacent to the bearing surface for these test slabs. All slabs were able to reach their full shear capacity, even with as little as 38 mm (1.5 in.) of bearing.

Because all slab shear failures in this study were triggered by a web-shear crack for the 305 mm (12 in.) deep slabs, it would be prudent to reintroduce a specific equation for evaluating the elastic web-shear capacity of precast, prestressed concrete hollow-core members into the upcoming edition of CSA A23.3. It is expected that including a web-shear capacity equation, together with the current postcracking shear capacity equations, would help improve the accuracy in shear strength prediction for precast, prestressed concrete hollow-core members, especially for those slabs with low amounts of prestressing. A web-shear resistance equation would also allow for failure modes to be checked, rather than applying a single value for shear capacity assuming the slab is already cracked. This would result in a more uniform level of conservatism to all slab prestressing ranges so that the experimental-to-predicted capacity ratios are closer to unity.

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Notation

- A_g = slab gross area
- b_w = total web width at the height within the slab depth for which the shear stresses are calculated

d = depth of the reinforcing strands measured from the concrete compression fiber

f'_c = compressive strength of concrete

I_g = gross moment of inertia

L = slab length

P_{Exp} = applied load

V_{c-pre} = code-predicted shear capacity as calculated at the predicted failure location

V_{exp} = calculated shear force at the predicted failure location due to effect of applied load P_{Exp} plus self-weight of the slab unit

θ = predicted crack angle per CSA A23.3-14

About the authors

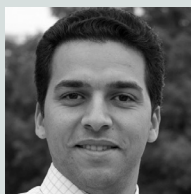


Karl A. Truderung, PEng, is a precast concrete consulting structural engineer and associate with Tower Engineering Group in Winnipeg, MB, Canada. He is a member of the Canadian Precast/Prestressed Concrete Institute

Technical Committee and a member of the CSA A23.3 Technical Committee on Reinforced Concrete Design.



Amr El-Ragaby, PhD, PEng, is an assistant professor in the Department of Civil and Environmental Engineering at the University of Wisconsin–Platteville.



Mohamed Mady, PhD, PEng, is an assistant professor in the Department of Structural Engineering at Cairo University in Egypt. Also, he is a bridge maintenance engineer in the engineering division of the Public Works of the

City of Winnipeg, MB, Canada.



Ehab El-Salakawy, PhD, PEng, FCSC, is a professor in the Department of Civil Engineering at the University of Manitoba in Winnipeg.

Abstract

Precast, prestressed concrete hollow-core slabs offer an economical solution where large spans dictate the design. There are currently multiple code methodologies for shear design; however, only the European standard EN 1168 addresses web-shear design specifically for prestressed hollow-core slabs. This paper presents the experimental results of eight 305 mm (12 in.) deep precast, prestressed concrete hollow-core slabs tested until failure to evaluate their shear capacity against the shear predictions in the Canadian standard CSA A23.3-14, ACI 318-14, and EN 1168. The results showed that EN 1168 was the most accurate predictor of the web-shear capacity of the precast, prestressed concrete hollow-core slabs for the range of tested variables, including the amount of prestressing and different support bearing lengths, with an average experimental-to-predicted capacity of 0.96. The test results and comparisons emphasized the need to introduce a specific equation for evaluating the elastic web-shear capacity of precast, prestressed concrete hollow-core members into the upcoming edition of CSA A23.3.

Keywords

Bearing length, hollow-core slab, prestressing level, shear capacity, transfer length.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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