Shear Strength of Precast Prestressed Concrete Hollow Core Slabs

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Abstract

Since early eighties, the precast prestressed concrete hollow core slab cross sections with non-circular voids became gradually popular, first in 400 mm thick slabs, then in 500 mm thick slabs. However, it is evidenced that this type of deeper slab sections have subjected to initial web shear cracking when they are provided longer supports and resist for higher line loads acting close to supports. Therefore, the objective of this study is to review the equations specified in American Concrete Institute (ACI), Eurocode 2 (EC2) and Canadian Standards Association (CSA) specifications to evaluate the shear strength of a member having no transverse reinforcement as in the case of hollow core slabs.

For this purpose, the experimental test data on hollow core slabs are collected from past experimental programs and detailed finite element analyses are performed. Based on experimental and numerical results, it could be concluded that the evaluation of shear strength by the equations specified in ACI, EC2 and CSA specifications are conservative for the slab cross sections with circular voids while ACI and EC2 predictions are not conservative for deeper slab sections with flat webs. However, CSA predictions for all types of hollow core slab sections are more conservative than ACI and EC2 predictions.

Keywords: Hollow core slab; Shear strength; Prestressed concrete; Precast members; Circular Voids; Flat weds

1. Introduction

Prestressed hollow-core concrete slabs were developed in the 1950s, when long-line prestressing techniques evolved, and for more than 30 years the type of units produced changed little. These slabs made of high-strength concrete, are prefabricated concrete members with large hollow proportions. In practice, they are interconnected after assembly by joint grouting compound. In comparison with conventional concrete members, this type of concrete slabs has a lot of economical advantages, especially in saving material, energy and in reducing weight of transportation. Outstanding features are quality control, schedule time and costs. Additionally, formworks which are used to produce in-situ concrete are saved in application of these slabs. The first prestressed hollow core slabs were 150, 200 or 265 mm in thickness. They were provided with circular voids. Since the early eighties, slab cross sections with non-circular voids became gradually popular, first in 400 mm thick slabs, then in 500 mm thick slabs. These deeper slab units are increasingly used in industrial buildings, office buildings and also in domestic architecture where it is needed to have large open parking spaces on the ground floor. As a subsequent of that these types of hollow-core slabs were developed to resist the higher loads and to support for longer span. Typical cross-sections of the slabs with circular voids and non circular voids (flat webs) are shown in Figure 1. In a section with non circular voids, the inner webs have a constant thickness over a depth of *h*/3. The outermost webs are only slightly tapered due to the non-verticality of the outer edges.

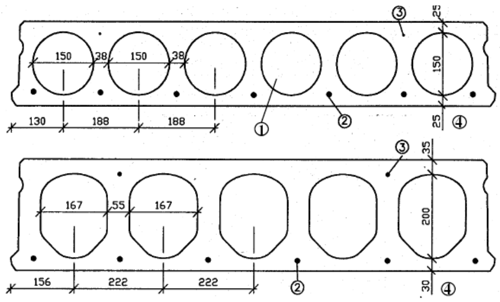


Figure 1. Typical slab cross-sections with circular and non-circular voids

These deeper slab sections provided longer supports or resisted for higher line loads acting close to supports, subjected to the initial web shear cracking. Subsequently, It was realized that resistance of the slabs with flat webs against web shear failure was considerably lower than shear strength evaluated by the equations specified in ACI (2005) and EC2 (2005) specifications. The experimental studies by Pajari (2005) and Hawkins and Ghosh (2006) have alos illustrated that web-shear cracking strengths in end regions can be less than strengths computed by traditional equations specified in ACI (2005) and EC2 (2005) specifications.

Therefore, the first part on the study is going to compare the shear design approaches of 3 different specifications: ACI (2005), EC2 (2005) and CSA (2001). The equations specified in ACI (2005) and EC2 (2005) specifications to evaluate the shear strength are based on predicting diagonal cracking loads by considering stresses at the centroid while the equation in CSA (2001) is based on the Simplified Modified Compression Field Theory (Vecchio et al.(1986), Collins (1997) and Angelakos et al. (2001)) which considers the post-cracking shear strength of a member. This part of study also presents the comparison of observed shear strength from experiments with the code predicted shear strength. The second part of the study presents the results of finite element analyses of typical hollow core slabs with wide range of depths to explore why the deeper slabs subject to web shear cracking at lower loads than the shear strength evaluated by the equations specified in ACI (2005) and EC2 (2005) specifications. For this purpose, 220, 300, 400, and 500mm deep sections are selected and all details of these sections are taken from a manufacturer of prestressed hollow core slabs.

2. Evaluation of Shear Strength

2.1. ACI specification

The use of shear reinforcement is generally not feasible for hollow core slabs and, therefore, the shear strength, particularly of deep slabs, may be limited to the shear strength of the concrete. Section 11.4 of ACI (2005) gives the requirements for evaluating the shear strength of concrete. The provisions of the section 11.4.3 are likely to be used if shear is a controlling factor in the design of hollow core slabs. In Section 11.4.3, the factored shear force V_u is limited to the lesser of φV_{ci} and φV_{cw}, where V_{ci} is the flexure-shear cracking strength and V_{cw} is the web-shear cracking strength. For simply supported hollow core slabs, the shear cracking strength of the web adjacent to the support usually control the design of the unit, unless the design loading includes heavy, non-uniform loads. The φ value for shear calculations is 0.75. The nominal shear strength provided by concrete (web-shear cracking strength) V_{cw} is given in ACI (2005) as:

$$V_{cw} = (0.29\sqrt{f_c^{'}} + 0.3 f_{pc}) b_w d + V_P \tag{1}$$

Where $f_{pc}=P/A$, $d= y_t+e$ but not less than 0.8h, b_w is the width of the section at the centroidal axis and V_P is the vertical component of the prestressing force.

2.2. Eurocode 2 specification

In prestressed single span members without shear reinforcement in regions uncracked in bending (where the flexural tensile stress is smaller than f_{ctk}), the shear resistance should be limited by the tensile strength of the concrete using expression (6.4) in EC2 [2005] as:

$$V_{Rd,c} = \frac{Ib_w}{S}\sqrt{(f_{ctd})^2 + \alpha_1 \sigma_{cp} f_{ctd}} \tag{2}$$

where I is the second moment of area, b_w is the width of the cross-section at the centroidal axis, S *is* the first moment of area above and about the centroidal axis, α_l equal to $l_x/l_{pt2} \leq 1.0$ for pretensioned tendons and otherwise it equals to 1, l_x is the distance between the section considered from the starting point of the transmission length and the section considered at the distance of half of the slab thickness, l_{pt2} is the upper bound value of the transmission length of the prestressing element according to Expression (8.18) in EC2 (2005), σ_{cp} is the concrete compressive stress at the centroidal axis due to axial loading or prestressing ($\sigma_{cp} = N_{Ed} / A_c$ in MPa, $N_{Ed} > 0$ in compression) and f_{ctd} is defined as the design tensile strength.

2.3. CSA specification

The equation for the evaluation of shear strength in CSA (2001) is based on the Simplified Modified Compression Field Theory (SMCFT) which considers the post-cracking shear strength of the member. Factored shear strength V_c shall be determined by clause 11.3.4 in CSA (2001) as:

$$V_c = \phi_c \lambda \beta \sqrt{f_c^{'}} b_w d_v \tag{3}$$

From the clause 11.3.6.4 in CSA (2001), β is defied as:

$$\beta = \frac{0.4}{(1+1500\varepsilon_x)} * \frac{1300}{(1000+S_{Ze})} \tag{4}$$

The longitudinal strain ε_x at mid-depth of the cross-section can be computed from Eq. (5)

$$\varepsilon_x = \frac{M_f / d_v + V_f - A_P f_{po}}{2(E_p A_p + E_c A_{ct})} \tag{5}$$

Where M_f and V_f shall be taken as positive quantities and M_f shall not be taken less than $(V_f - V_p)/d_v$

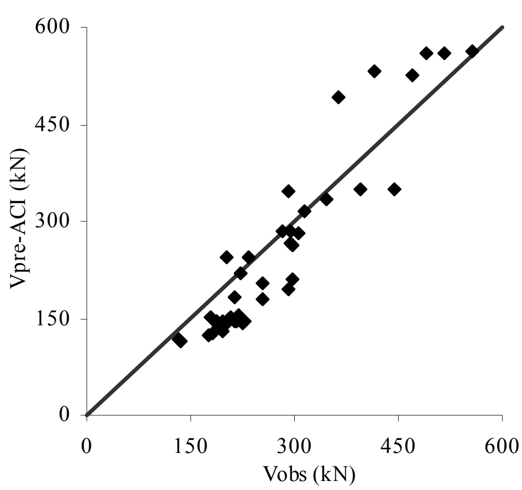
$$S_{Ze} = \frac{35 S_z}{15 + a_g} \tag{6}$$

However, S_{Ze} shall not be taken as less than $0.85 S_Z$ and S_Z shall be taken as d_v. a_g is maximum size of coarse aggregate and effective web width shall be taken as the minimum concrete web width within the depth. The prestressing force may be assumed varying linearly from zero to full development in the transfer length which is assumed to be 50 times the diameter of strand as in ACI (2005) Specification. The resistance factor for concrete, φ_c is taken as 0.65 while for low density concrete it is equal to 1.

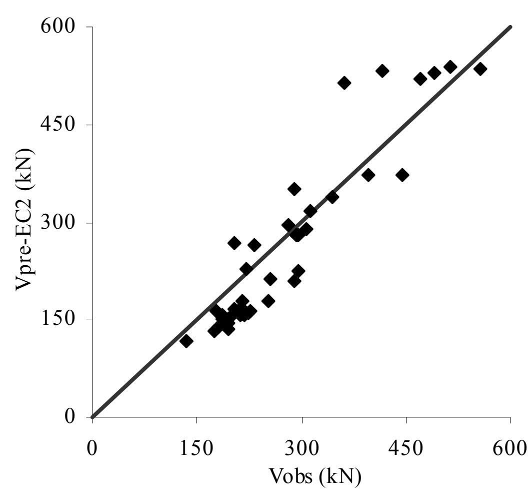
3. Comparison of Code Predictions with Experimental Data

Main objective of this part of the study is to validate the accuracy of evaluation of shear strength of precast prestressed concrete hollow core slabs by the equations specified in ACI (2005), EC2 (2005) and CSA (2001). For this purpose, test data from forty four specimens are selected from the research report by Pajari (2005). It is also important to note that those specimens were simply supported, isolated (not a part of a floor) and loaded with transverse uniformly distributed line loads. The test specimens which have excluded, are only those in which the slabs had grouting at the loaded end, some important data as the measured strength were missing, the shear span (distance from support to the nearest line load) was lesser than 2.4 times the slab thickness and the slippage of strands was greater than that acceptable in the Finnish quality control for type approved slabs. Altogether, 15 different nominal geometries for concrete cross-section were identified in the accepted test specimens.

Figure 2 (a), (b) and (c) illustrate the comparison of shear strength values obtained from the tests with the predicted shear strength by ACI (2005), EC2 (2005) and CSA (2001), respectively. In these figures, V_{obs} refers to the shear strength obtained from the test (shear force at support) while V_{pre} refers to the predicted shear strength by the code equation.



(a)



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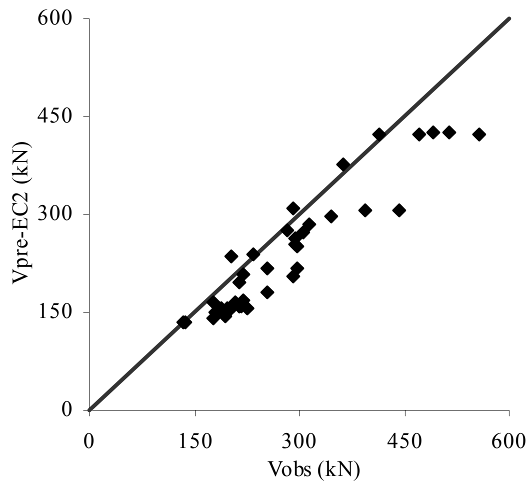


(c)

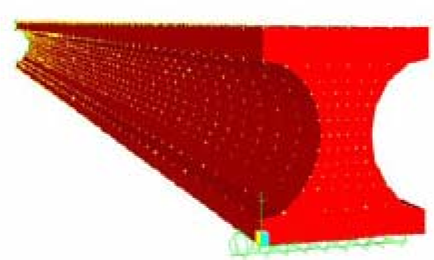
Figure 2. Comparison of observed shear strength with the predicted shear strength by (a) ACI, (b) EC2 and (c) CSA.

Furthermore, it is important to highlight that the predicted shear strength by ACI (2005) and CSA (2001) codes are evaluated using the material safety factors of 1.0 while the predicted shear strength by EC2 (2005) is evaluated using the characteristic tensile strength for this comparison.

It is clear from the comparison that the shear strength values predicted by ACI (2005) and EC2 (2005) for the shallow sections with circular voids are mostly conservative, but they are not conservative for the deeper sections with non circular voids. However, CSA (2001) predicts conservative estimation of shear strength values of all the sections selected for this comparison.

4. Finite Element Modelling

In order to investigate the stress distribution close to the support under a symmetrically loading condition in different slab units and in turn to validate the assumption that the shear stress would reach its maximum value at the neutral axis down to the flexural steel, made in deriving the equations to evaluate the shear strength in ACI (2005) and EC2 (2005) using the Mohr's circle of stresses, the four finite elements models of slab units: 220mm, 300mm, 400mm and 500mm in depth selected from the experimental program are modelled in SAP 2000 program. Figure 3 illustrates the 3D view of a model with circular voids and the loading arrangement.



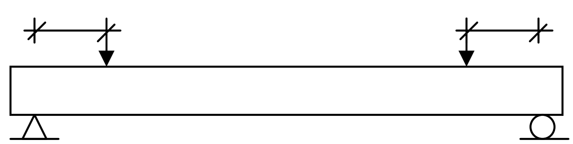


Figure 3. 3D view of a model with circular voids and the loading arrangement

Due to symmetric loading arrangement, half of the beam was modelled. The end details are critical of this type of slab units. Typically, these slabs sit on the small pads only 50mm long right at the ends of the slabs. Hence, all beams were modelled with roller support at one end (50mm distance away from edge of the beam) while all translational degrees of freedom except the vertical translation at other end of the model were restrained.

A vertical line load, corresponding to the experimental failure load, is placed on the top of the model at a distance of five times d_v from the support. The spans of the beams are selected to have same span over effective depth ratio of 25. The prestressing force is transferred in the model as shown in Figure 4. The transfer length is defined as the length required building up the full prestressing force in the concrete. As suggested in the ACI (2005) specification, the transfer length is taken as 50 times the strand diameter.

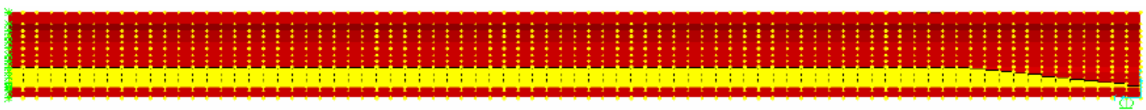


Figure 4. Transferring of prestressing force.

Concrete is modelled as a homogeneous material which results more general behaviour of the beam with the properties reported in the experiments. Modulus of elasticity of the unconfined concrete is calculated using the equation as:

$$E_c = 3320\sqrt{f_c^{'}} + 6900 \text{ MPa} \tag{7}$$

The reinforcements in hollow core slabs are consisted of only longitudinal strand. Each of strands built of seven wires of low relaxation strands are modelled as cable elements using the properties as reported in the experimental program assuming full interaction with concrete.

4.1. Results of finite element models

Figure 5 (a), (b) and (c) exhibit distributions of the direct axial, shear and principle tensile stress components in the model with a section of 300 mm in depth and including the circular voids at the corresponding failure load as reported in experiment program. Figure 6 also shows the distribution of the three stress components of the model with 400mm in depth and consisting of non circular voids.

It is clear from Figure 5 that the maximum shear and tensile stresses are developed at the centroid of the section as it is assumed in deriving the code equations that the shear stress would reach its maximum value at the neutral axis. Since, the maximum tensile stress developed at the web has reached to its peak value, web shear crack could be developed leading in shear failure of the slab rather than having a flexural failure because of rapid propagation of the crack after the initiation of the web tensile crack in the web.

Unlikely previous results, Figure 6 indicates that the maximum shear and tensile stresses are not developed at the centroid of the section. They are more concentrated towards the bottom of the beam because any deformed section close to the support is no longer remained in plane with deeper section consisting of flat webs. Usually, this region is called a disturb region as observed between the support and the loading point and it violates the concept of plane section remained in plane and that it is perpendicular to the longitudinal axis of the slab. Therefore, at lower loads than the evaluated shear strength of such members by the ACI (2005) and EC2 (2005) specifications, web tensile crack could be initiated due to higher concentration of shear stresses.

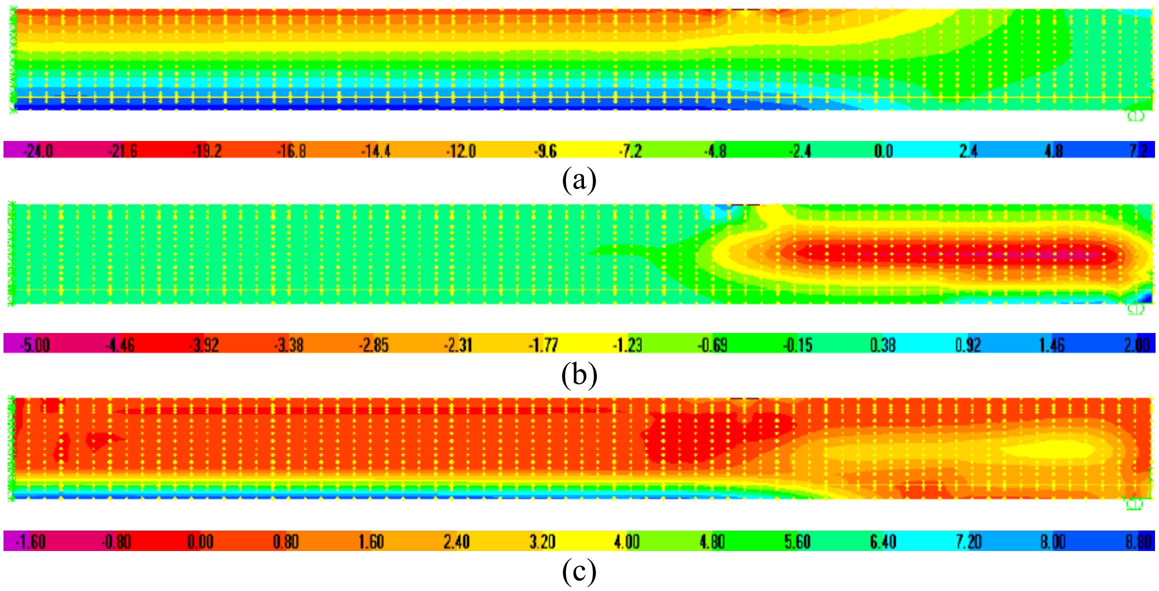


Figure 5. Distribution of (a) direct stress, (b) shear stress and (c) principle tensile stress in MPa of 300mm deep section with circular voids

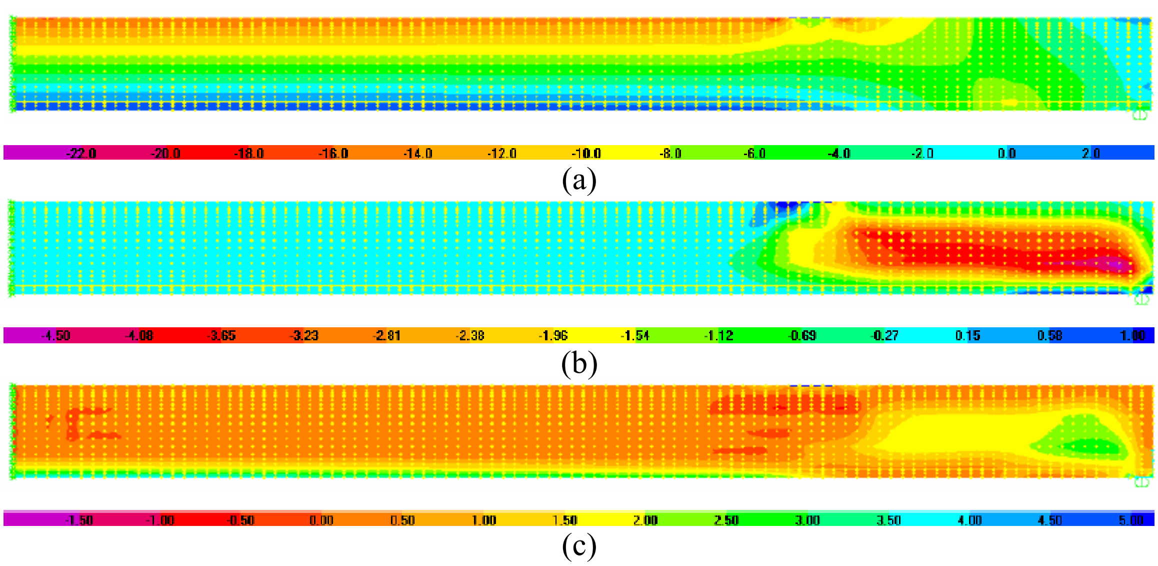


Figure 6. Distribution of (a) direct stress, (b) shear stress and (c) principle tensile stress in MPa of 400 mm deep section with flat webs.

5. Conclusion

ACI (2005), EC2 (2005) and CSA (2001) specifications propose equations to evaluate the shear strength of a member which have no transverse reinforcement. To check the validity of these equations, finite element analyses and 44 experimental tests on precast prestressed hollow core slabs with thickness varying from 220 to 500 mm have been performed. Based on the results the following conclusions can be drawn.

- According to the experimental test data of 265 and 320 mm deep sections with circular voids, ACI (2005), EC2 (2005) and CSA (2001) specifications predictions are conservative. Finite element analyses illustrate that 220,300 mm deep sections with circular voids follow the assumption that plain section remains in plain at the section where the shear forces are high and that the maximum principle tensile stress occurs at the mid depth of the slab. Therefore, the results of finite element analyses give strong support that code predictions are conservative for these types of sections with circular voids.
- As the slab depth becomes larger with flat web, the stress distribution becomes non linear with tensile and shear stresses concentrating towards the bottom of the beam. So, Morch (1902) prediction that the shear stress would reach its maximum value at the neutral axis down to the flexural steel is not going to be valid for deeper precast prestressed concrete hollow core slabs with flat webs. The maximum value of shear stress is much higher the predicted shear stress by code equations and the maximum principal tensile stress is not at centroid of the section .As a result of that, the equations in ACI (2005) and EC2 (2005) specifications, derived from the Mohr's circle of stress and based on the assumption made by Morch (1902), estimate the non conservative strength values for deeper prestressed hollow core slabs with flat webs.
- CSA (2001) prediction on the shear capacity is based on the Simplified Modified Compression Field Theory and it estimate the conservative shear strength values for all kind of sections used in this study. It is because the modified compression field theory, used in the CSA (2001) to calculate the shear stresses at each level, treat concrete as a diagonally cracked material and interface shear stress, often called aggregate interlocking, is estimated by average tensile stress. The interface shear plays an important role in the determination of the shear strength of the members without transverse reinforcement. The stress strain relationship for the concrete indicates that average tensile stresses, after concrete diagonally cracked, are comparatively lower than the tensile stress at the first crack.
- Compressive strength of concrete used in precast prestressed hollow core slabs is comparatively high. High strength concrete member are smoother than in normal strength concrete members with cracks propagating through coarse aggregate particles rather than around them. So the tensile strength at cracking of the members with high strength concrete may be lower than the tensile strength at cracking used in the specifications like ACI (2005) and EC2 (2005). This also should be considered in the design of prestressd hollow core slabs.

6. References

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