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Design of concrete members strengthened in shear with vertical bonded bars

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Abstract: Due to design using older codes, material degradation and increased traffic loads, numerous reinforced concrete bridge members, such as beams and slabs, have insufficient shear capacity and need shear strengthening. An efficient shear strengthening technique consists of adding drilled-in vertical reinforcing bars within the existing concrete member and to bond these bars to the concrete with high-strength epoxy adhesive. However, the shear design method prescribed by the Canadian Highway Bridge Design Code for members with stirrups may overestimate the shear capacity of shear strengthened members with epoxy-bonded bars. This paper presents the effect of bonded shear reinforcement on shear resistance mechanisms and summarizes a proposed shear design method developed for reinforced concrete members with epoxy-bonded shear reinforcement.

1 Introduction

Several situations may require shear strengthening of reinforced concrete bridge members, such as shear capacity reduction caused by material degradation, increase in loading, construction defects or deficient older code requirements. Several shear strengthening techniques studied by researchers and proposed by codes consist to install reinforcing bars or reinforcing material on the lateral faces of a member (Adhikary & Mutsuyoshi, 2006; Al-Mahmoud et al., 2009; Colalillo & Sheikh, 2014; Ferreira et al., 2016; Hellberg & Eryd, 2018; Lechner & Feix, 2016; Mofidi et al., 2016). Even if these techniques have been proven to be efficient in beams, their efficiency on large structures like thick concrete slabs and wide beams is limited. To overcome this limitation, some researchers suggested to insert drilled-in reinforcing bars and anchor the bars to the concrete with high strength adhesives or mechanical anchorages (Bédard, 2018; Cusson, 2012; Fernández Ruiz et al., 2010; Fiset et al., 2019; Hellberg & Eryd, 2018; Inácio et al., 2012; Provencher, 2010; Valerio et al., 2011). Fig. 1 shows the section of a wide structural thick slab strengthened in shear with bars inserted in drilled holes from the top face of the bridge and anchored with an adhesive. This technique offers many practical advantages and in particular, its efficiency on wide members and the limited impact on traffic during the strengthening process.

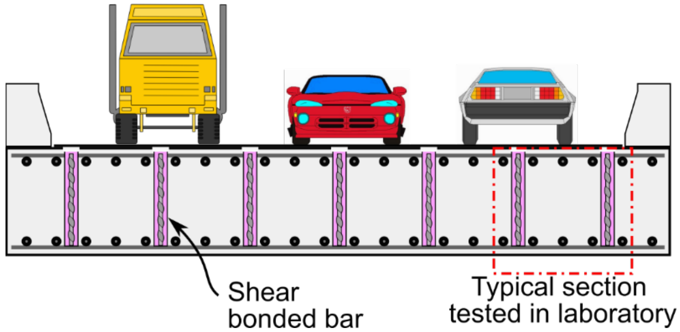

Fig. 1 – Typical reinforced concrete thick slab bridge section strengthened in shear with reinforcing bars inserted from the top face into drilled holes and anchored with an adhesive

2 Tests overview

The shear behaviour of wide concrete members has been experimentally investigated by many researchers in the past. Table 1 and Fig. 2. present the main properties and the shear capacity of similar members strengthened in shear with vertical bars bonded to the concrete with high-strength adhesives. Fig. 2 also compares the strengthened member shear capacity, $V_{strengthened}$, to the one of a reference member without shear reinforcement, $V_{without}$. As indicated in the figure, although similar, the members exhibited different values of longitudinal transverse reinforcement spacing ratio, s/d_v (s is the spacing of transverse reinforcement measured along the longitudinal member axis and d_v the effective shear depth).

Table 1: Main properties of shear strengthened members presented in Fig. 2

Author	s/d_v	h (mm)	f'_c (MPa)	ρ_v (%)	d_b (mm)	f_y (MPa)
Valerio (2009)	1.05	350	55.0	0.06	8	530
Provencher (2010)	0.75	750	35.6	0.14	16	480
Bédard (2018)	0.67	750	39.6	0.16	16	449
Bédard (2018)	0.61	750	40.2	0.17	16	449
Cusson (2012)	0.60	750	34.5	0.17	16	448

In Table 1: h the member thickness, f'_c the average concrete compressive strength, ρ_v the bonded shear reinforcement ratio, d_b the bonded bars diameter and f_y the average bonded bars yield strength.

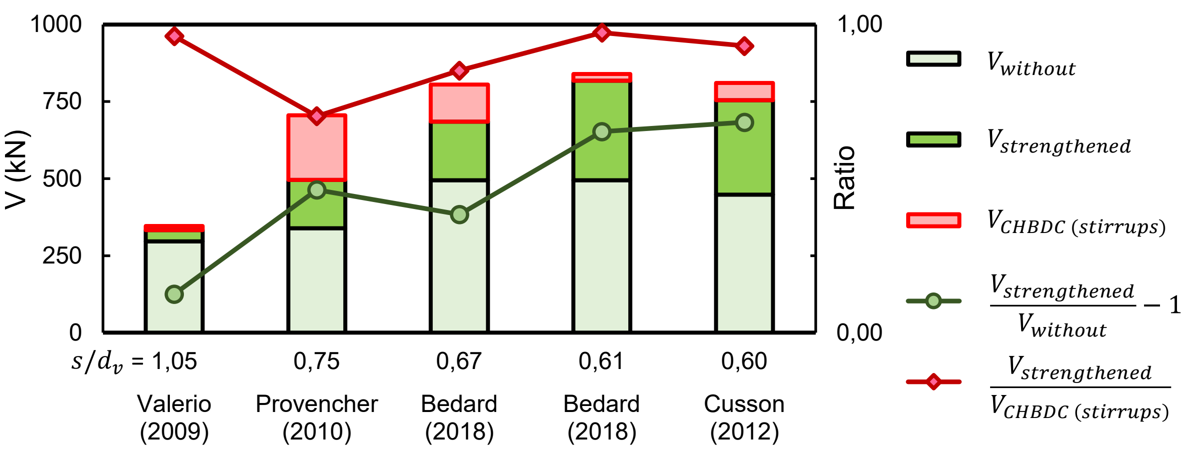

Fig. 2 – Shear capacity of shear strengthened members with bonded bars tested by several authors

As expected, the shear strengthening technique with bonded bars increases the shear capacity of all members ($V_{strengthened} > V_{without}$). Also, the reduction of the spacing ratio, which corresponds to an increase in the amount of shear reinforcement, results in a larger shear capacity. Valerio (2009) tested a spacing ratio larger than the maximum ratio ($s/d_v = 0.75$) permitted by the Canadian Highway Bridge Design Code (CHBDC) (CSA-S6, 2019) for stirrups and observed a limited shear capacity increase of 13% ($V_{strengthened} / V_{without} - 1 = 0.13$). By comparison, a significant increase in shear capacity of 46%, was observed by Provencher (2010) for a member with a spacing ratio of 0.75. The largest capacity increase was observed by Cusson (2012) for a member with the smallest spacing ratio of 0.60 among the tested specimens

Fig. 2 also compares the measured capacity of shear strengthened members to the CHBDC nominal predictions assuming that the shear reinforcement consists of typical stirrups, $V_{CHBDC\ (stirrups)}$. It can be observed that for all members, the shear capacity of the strengthened member is lower than the predicted capacity using the CHBDC provisions assuming stirrup reinforcement. This overestimation reaches 30% ($1- V_{strengthened}/V_{CHBDC\ (stirrups)}$) for the member tested by Provencher (2010) with a spacing ratio s/d_v of 0.75. By reducing the spacing ratio s/d_v, this overestimation progressively decreases to not more than 7% for the member tested by Cusson (2012). On the other hand, the shear capacity is overestimated by only 4% for the member with the largest spacing ratio s/d_v of 1.05. Therefore, it appears that expecting that a member with bonded shear reinforcement will behave like a member with stirrups is a very unsafe assumption for design. The spacing ratio of bonded shear reinforcement plays a significant role in the shear strengthened member behaviour and capacity, although this spacing ratio s/d_v is not the only contributing factor.

To better understand factors affecting the behaviour of a shear strengthened member, Fig. 3 compares the response of two members tested by Cusson (2012), the first one strengthened with bonded shear reinforcement and the second one designed with conventional stirrups respecting the CHBDC provisions.

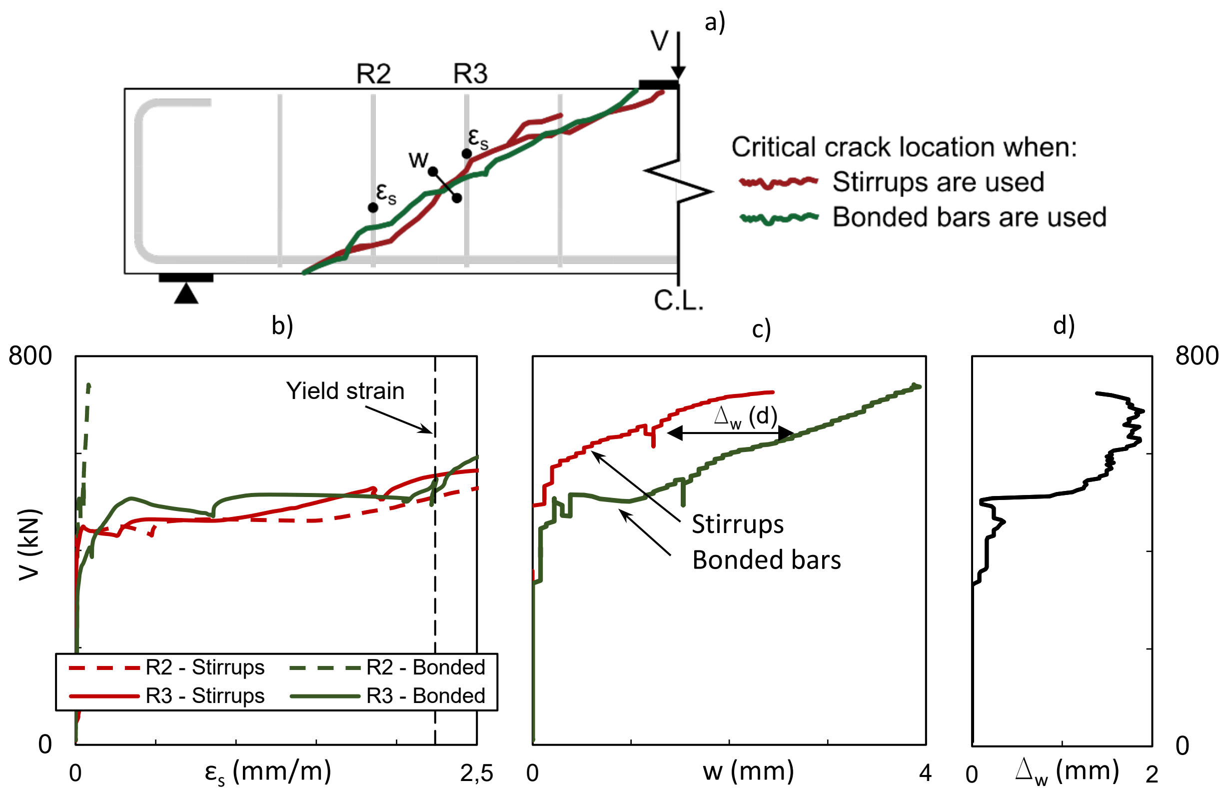

Fig. 3 – a) Critical shear crack location at shear failure for two similar members tested by Cusson (2012) with $s/d_v = 0.60$, one with conventional stirrups and one with bonded bars (half the member illustrated), shear, V, versus b) shear reinforcement strain, ε_s, c) crack width, w, and d) increased crack widths, Δ_w, for bonded reinforcement

Fig. 3a shows that the critical shear crack locations are very similar for both members. The cracks in both members cross three shear reinforcement locations. Both critical shear cracks cross the transverse reinforcement at location R2 close to its bottom extremity and at location R3 approximatively at its mid-height. Looking at the measured strains in the transverse reinforcement at locations R2 and R3, shown in Fig. 3b, the response of the bonded bars differs from the response of stirrups. For the reinforcement at R3, both the bonded bars and the stirrups reached their yield strain at a shear, V, of about 550 kN. However, the bonded bars were activated at a smaller shear value (V = 350 kN for the bonded bar compared to V = 430 kN for the stirrup reinforcement). At location R2 there was a very gradual increase in the strain of the bonded bar compared to the rapid increase of the stirrups strain. At location R2, the stirrups reached yielding at about the same shear value than the stirrups at location R3, while the measured maximum bonded bars strain was 0.1 mm/m at maximum shear, which is far below the yield strain of 2.24 mm/m. The difference in behaviour between the two types of transverse reinforcement can be attributed to their different anchorage mechanisms. For a bonded reinforcing bar, the critical crack location defines its embedment length (distance from end of bonded bar to the crack location) and hence can restrict its ability to develop significant strains if there is a short embedment length. On the other hand, conventional stirrups are considered fully anchored and fully active. Thus, the contribution to shear capacity of bonded bars are typically lower than the contribution of stirrups (Fiset et al., 2014).

Fig. 3c presents the crack width, w, measured at the member mid-height for members with bonded bars and with stirrups. For these members, it was experimentally observed by Cusson (2012) that the shear cracking occurred at a similar shear value of about 350 kN. At that loading, the diagonal crack was well controlled by the stirrups at location R2 while it crossed both R2 and R3 transverse reinforcement in the member with bonded bars. For the member with stirrups, the shear crack reached the member mid-height at a shear value of about 480 kN and further increases in shear resulted in widening of the shear crack until shear failure. For the member with bonded bars, the shear crack opened rapidly at a shear, V, of about 510 kN with a crack width of about 1.2 mm, which is significantly larger than the crack width in the member with stirrups for the same shear loading. Further increases in shear progressively widened the critical shear crack in the member with bonded bars and this crack width remained larger than the one in the member with stirrups. Fig. 3d presents the difference between the crack widths measured in the two different members ($\Delta_w = w_{\text{bonded bar}} - w_{\text{stirrup}}$). At a shear value, V, of 510 kN, Δ_w reached 1.2 mm. Then Δ_w increased to 1.5 mm at a V of 560 kN and did not vary much up to the shear failure (Δ_w = 1.4). Since the critical crack width in the member with bonded bars is larger than the one in the member with stirrups, it is expected that the member with bonded bars experiences lower aggregate interlock and a lower concrete contribution to shear capacity, V_c..

3 Shear capacity design method

The shear capacity, V_r, determined according to the CHBDC considers the concrete contribution, V_c, and the shear reinforcement contribution, V_s, to the shear capacity with Eq. [1].

[1] $$V_r = V_c + V_s \leq 0.25\phi_c f'_c b_v d_v$$

In this equation, Φ_c is the resistance factor for concrete, b_v is the effective width of the web and f'_c is the concrete compressive strength.

3.1 Concrete contribution to shear capacity

The concrete contribution to shear capacity, V_c, is given by Eq. [2].

[2] $$V_c = 2.5\beta\phi_c f_{cr} b_v d_v$$

In this equation, β is a factor accounting for the shear resistance of concrete and f_{cr} is the cracking strength of concrete. As presented in Fig. 3c, the crack width in members with bonded bars is larger than the one in similar members with stirrups. Considering that the difference in crack width Δ_w contributes to reducing aggregate interlock, the factor β proposed by Bentz and Collins (2006) for members with shear reinforcement must be adapted for members with bonded bars as given by Fiset (2019):

$$\beta = \frac{0.4}{1 + 1500\varepsilon_x + 1.5\left(\frac{35}{15 + a_g}\right)p\Delta_w} \quad [3]$$

In this equation, ε_x is the longitudinal strain at mid depth of the member specified by the code. Considering reinforcing steel typically used in north America (yield strength, f_y, of 400 MPa), the difference in the crack width, Δ_w, can be determined for vertical bars with Eq. [4] (Fiset, 2019).

$$\Delta_w = 1.1\left(u_1 + \frac{\sqrt{d_b}}{30}\left(1 + 1.5\frac{\Sigma\ell_a}{\Sigma\ell_d}\right) - \frac{d_b}{3f_c'^{(2/3)}}\right) \geq 0 \quad [4]$$

In this equation, d_b is the bonded bar diameter in millimeters, u_1 is the bar slip at the peak of bond stress relationship measured by pullout tests (Villemure et al., 2019), $\Sigma\ell_d$ is the summation of the bonded bar development length, ℓ_d, and $\Sigma\ell_a$ is summation of the anchorage length, ℓ_a. The value of $\Sigma\ell_a$ can be taken as the difference between the bar length ℓ_{bar} and d_v ($\Sigma\ell_a = \ell_{bar} - d_v \geq 0$ for vertical bars) without exceeding $\Sigma\ell_d$. The parameter, p, represents the proportion of the member affected by Δ_w. For the design of a member with vertical bonded bars, p can be estimated with Eq. [5].

$$p = \frac{\Sigma\ell_d - \Sigma\ell_a}{s\tan\theta}\begin{cases} \leq 1 \\ \geq 0 \end{cases} \quad [5]$$

Where θ is the angle of inclination of the principal diagonal compressive stresses. For a bonded bar without mechanical anchorage, $\Sigma\ell_d$ can be determined with Eq. [6].

$$\Sigma\ell_d = \frac{f_y d_b}{2f_{by}} \quad [6]$$

Where the bond strength at the yielding of the bonded bar, f_{by}, is determined with Eq. [7] (Fiset et al., 2018). This value corresponds to the lesser of the pullout capacity (characteristic for cracked concrete $k_c = 7.0$ (CSA-A23.3, 2019)) and the adhesive bond strength, f_{b0}.

$$f_{by} = \sqrt[3]{\frac{\Omega_\theta^2 k_c^2 f_c' f_y}{40d_b}} \leq f_{b0} \quad [7]$$

The factor Ω_θ considers the reduction of the pullout capacity in diagonally cracked concrete (Fiset et al., 2018) and can be determined as following ($\Omega_\theta = 1$ when $\theta < 37°$):

$$\Omega_\theta = 0.3 + \tanh\left(\frac{10}{\theta - 25}\right) \leq 1 \quad \text{[deg]} \quad [8]$$

3.2 Bonded shear reinforcement contribution to shear capacity

As presented in Fig. 3, the bond capacity of short embedded bonded bars may limit their contribution to the shear capacity, V_s, of a member. Considering this reduction, the average capacity of a bonded bar may be reduced by an efficiency factor, η, and the average contribution of vertical bonded bars to shear capacity, V_s, may be determined as following (Fiset et al., 2017):

$$V_s = \frac{\phi_s \eta f_y A_v d_v \cot\theta}{s} \quad [9]$$

Where Φ_s is the resistance factor for steel. The efficiency factor of a bonded bar can be determined with Eq. [10], where the debonding factor k_d and the anchorage factor k_a are given by Eq. [11] and [12]. The debonding factor considers the reduction in bond strength for a large crack due to the bar debonding while the anchorage factor considers the larger capacity of a bar exceeding the bonded bar development length (Fiset, 2019).

[10] $$\eta = \frac{\ell_{bar} - \Sigma\ell_d\left(1 - 0.4k_d k_a\right)}{d_v} \leq 1$$

[11] $$k_d = 1 - \frac{1000\varepsilon_x + p\Delta_w - u_1 - 0.8}{d_b} \begin{cases} \geq 0.4 \\ \leq 1.0 \end{cases}$$

[12] $$k_a = 1 - \left(\frac{\Sigma\ell_a}{\Sigma\ell_d}\right)^{2.5}$$

4 Validation of the design method

The shear capacity of members with bonded shear reinforcement determined by laboratory tests (Bédard, 2018; Cusson, 2012; Provencher, 2010; Valerio, 2009) and numerical analysis (Bédard, 2018; Fiset, 2019) are compared in Fig. 4 to the nominal shear capacity predicted by Eq. [1] ($\Phi_c = \Phi_s = 1$). Fig. 4a compares the test values, V_{test}, to the CHBDC nominal shear capacity predictions, V_{CHBDC} (η = 1 and Δw = 0). Fig. 4b compares V_{test} to the predictions of the proposed method for bonded bars, V_{calc} ($\eta \leq 1$ and $\Delta w \geq 0$). For the adhesive used to bond shear reinforcing bars to cracked concrete, the characteristics values are: u_1 = 1.5 mm, k_c = 7.0 and f_{b0} = 14 MPa (CSA-A23.3, 2019; Fiset, 2019).

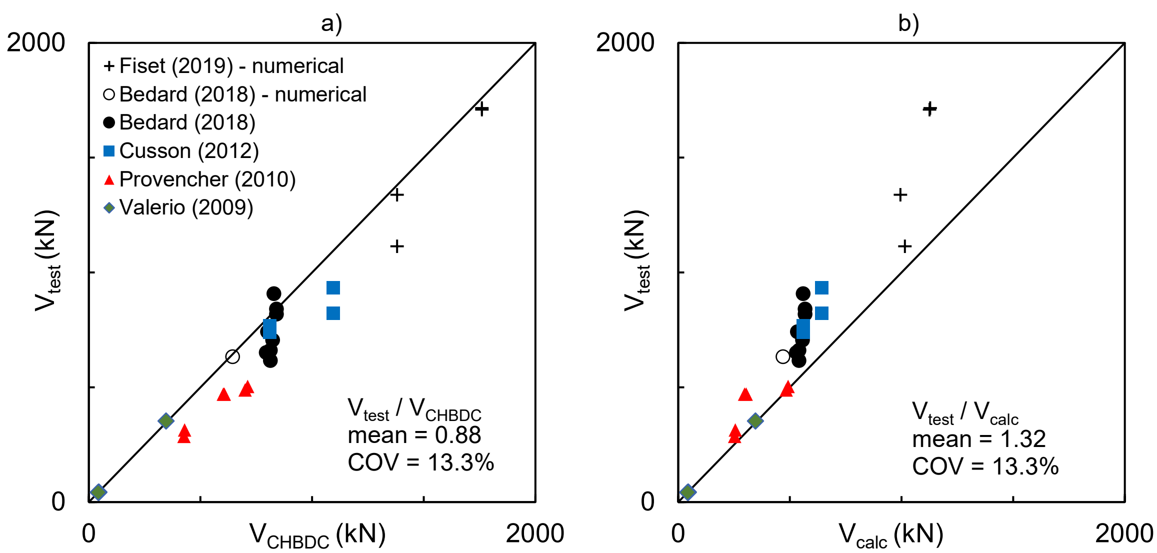

Fig. 4 - Shear capacity correlation between test and predicted values, a) CHBDC method considering stirrups (V_{CHBDC}) and b) proposed method, V_{calc}, considering bonded bars

It can be seen in Fig. 4a that if the predictions assume stirrups (corresponding to V_{CHBDC}) there is a considerable overestimation of the shear capacities of almost all members with bonded bars (V_{CHBDC} > V_{test}). On the other hand, the proposed method presented in Fig. 4b presents safe shear capacity predictions for all strengthened members (V_{calc} < V_{test}). For design, the global resistance factor, Φ, applied to the nominal shear capacity prediction can be determined as following (Wight & MacGregor, 2012):

[13] $$\phi = \text{mean} \times \exp\left(-0.75\mathrm{B} \times \text{COV}\right)$$

Where “mean” is the average ratio between the shear failure test value and the predicted shear nominal value, and “COV” is the coefficient of variation. Considering values given in Fig. 4 and a reliability index, B, of 3.5, one can determine the required Φ value of 0.62 for V_{CHBDC}, which is below the material factor Φ_c = 0.75 and Φ_s = 0.90 prescribed by the code. On the other hand, Φ = 0.93 can be applied to the proposed model for shear design to respect the reliability index of 3.5, which is above Φ_c and Φ_s prescribed by the code CSA-S6 (2019). Therefore, the proposed method for the design of members with bonded shear reinforcement is considered safe using the current values of Φ_c and Φ_s specified by the CHDBC.

5 Conclusions

The shear design method presented in this paper consists of adding vertical reinforcing bars in holes drilled into the existing concrete member and to bond these bars to the concrete with a high-strength epoxy adhesive. Tests results showed that considering the bonded bars as stirrups to predict shear capacity with current design codes can lead to a very unsafe design. If was found that a member strengthened with bonded shear reinforcement behaves differently than a member with conventional stirrups. Both the concrete and the bonded shear reinforcement contributions (V_c and V_s) to the shear capacity are reduced compared to a member with stirrups due to larger shear cracks, resulting in smaller shear stresses on the crack interfaces, and the potential bond failure. An adapted shear design method considering these two phenomena is proposed in this paper. Comparison between predictions and test results showed that the proposed method provides safe and accurate predictions of the members shear capacity.

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