

Drilled-in shear reinforcement for concrete thick slabs: modelling aspects

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Abstract

In order to evaluate the performance of drilled-in bonded shear reinforcement, two series of large-scale slabs were tested before and after shear strengthening. Experimental results indicate that the 2006 Canadian Standard and the 2010 *fib* Model Code provisions overestimate the shear strength of such strengthened slabs by about 29% if the drilled-in shear reinforcement is assumed to be totally effective. The main goal of this research is to develop a method for predicting the shear strength of slabs with bonded shear reinforcement. A non-linear finite element model using the program VecTor2 was used to study parameters influencing the slab behaviour up to failure. Results showed that both the behaviour and ultimate strength of slabs are predicted well by the model while taking account of a stress-slip relationship for the bonded reinforcing bars. Moreover, the experimental observations during the loading corroborate the cracking pattern predictions.

1 Introduction

Thick slabs are a commonly used structural form for small and medium span bridges. For many such bridges, it was typically assumed that the concrete was able to fully resist the shear stresses and therefore, shear reinforcement was not required. However, due to the increase of traffic loads combined with material degradation, it appears that some of these thick slabs may need to be strengthened in shear. In this study, the shear strengthening method consists in steel rebars introduced into vertical pre-drilled holes with different anchorage systems, including epoxy adhesive and external mechanical anchorage. By comparing this method with other post-installed shear strengthening methods [1, 2, 3, 4] for narrow beams elements, the proposed method has the advantage to be fully effective on wide and thick elements such as slabs.

Two series of tests performed on thick slabs were conducted up to shear failure [5, 6, 7]. Results showed that while shear-strengthened slabs can exhibit failure loads 46% higher than original unstrengthened slabs, they showed failure loads 29% lower than the *fib* model code 2010 and CAN-CSA S6-06 code prediction values [8, 9]. One of the main objectives of the current research is to adequately predict the increase in shear strength of thick slabs strengthened by various methods and to provide a basis for a strengthening design method in light of the experimental and numerical results. Finite elements (FE) models were developed with VecTor2 [10] software. This paper presents FE models and numerical results of slabs with drilled-in bonded shear reinforcement. These results are compared with the experimental results of unstrengthened and strengthened slabs. The predictions show that the bond-slip behaviour of the reinforcing bar-epoxy-adhesive-concrete interface is a key parameter influencing the slab behaviour and the efficiency of the strengthening method.

2 Review of experimental slabs tests

Experimental tests were performed on two series of slab slices (beams), identified as the PP and the BC series [5, 6, 7]. The numerical study will focus on 6 slabs identified as PP3-U1, PP3-U2, PP3-R1, PP3-R2, BC1 and BC2. The slab properties and strengthening details are summarized in Table 1 and Figure 1. These slabs span 4m, and have a height of 750mm height and a width of 610mm. Two unstrengthened slabs (PP3-U1 and PP3-U2) and 2 strengthened slabs (PP3-R1 and PP3-R2) with the same overall dimensions were tested. For the slabs PP3-R1 and PP3-R2, the chosen spacing ratio of shear reinforcement, s_w/d_v , is close to the maximum value of 0.75 allowed by 2006 Canadian Standard for conventional stirrups. Slabs BC1 and BC2 have the same overall dimensions as slab PP3. However, they were strengthened in shear with a smaller shear reinforcement spacing ($s_w/d_v=0.61$). The shear strengthening method used for slabs PP3-R1, PP3-R2 and BC2 consists of vertical post-installed

reinforcing bars in pre-drilled holes and anchored by epoxy adhesive. The BC1 specimen has standard stirrups as prescribed by 2006 Canadian Standard and was therefore the only specimen with shear reinforcement installed before concrete casting. All strengthened slabs in shear contain two 15M (200 mm² per bar) reinforcing bars. Slabs PP3 and BC contain flexural reinforcement ratios of 1.17% and 1.65%, respectively. The shear spans “ a/d ” are 2.87 and 2.88 for slabs PP3 and BC, respectively.

The concrete cylinder compressive strengths (f_{cc}) presented in Table 1 were obtained on the day of testing of the slabs (ASTM-C39). The maximum aggregate size is 19mm. The yield strengths for the flexural reinforcement are 468MPa and 508MPa for slabs PP3 and BC, respectively. Properties of shear reinforcement used for slabs PP3-R1 and PP3-R2 slabs are: f_y of 480MPa; strain hardening strain ϵ_{shs} of 0.023; ultimate strength f_u of 690MPa; ultimate strain ϵ_u of 0.140. The properties of shear reinforcement used for slabs BC1 and BC2 are: f_y of 448MPa; ϵ_{shs} of 0.006; f_u of 633MPa; ϵ_u of 0.180. The Young modulus of steel is taken as 200GPa.

Table 1 Details of slab specimens

Slabs	Shear Reinforcement	d [mm]	s_w [mm]	s_w/d_v	A_{s_w} [mm ²]	f_{cc} [MPa]
PP3-U1	None	698	-	-	-	35.8
PP3-U2	None	698	-	-	-	33.2
PP3-R1	Bonded	698	470	0.75	400	34.0
PP3-R2	Bonded	698	470	0.75	400	37.2
BC1	Stirrups	694	380	0.61	400	33.3
BC2	Bonded	694	380	0.61	400	34.5

d : Effective depth to main tension reinforcement; s_w : Spacing of transverse shear reinforcement; d_v : Effective shear depth, taken as $0.9d$; A_{s_w} : Shear reinforcement area within a distance s_w

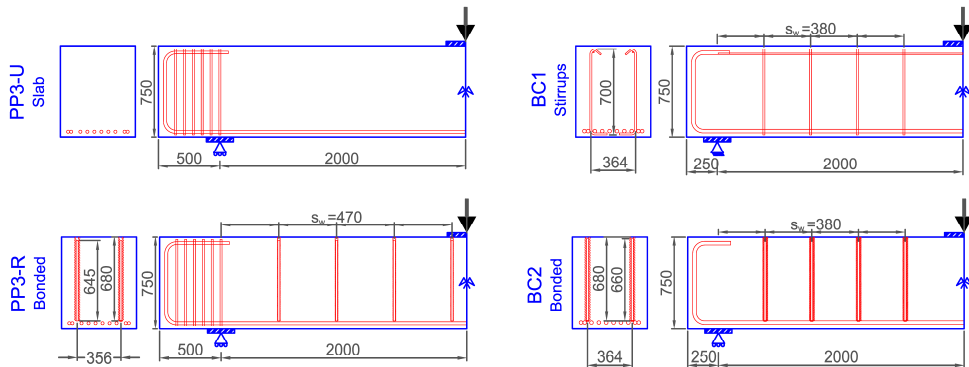


Fig. 1 Slabs specimens (unit: mm)

3 Numerical model

3.1 Methodology

Before investigating the influence of various parameters on the behaviour of thick slabs, the numerical model was validated. VecTor2 offers several options in terms of material behaviour. The selected model was first validated using the test results from unstrengthened slabs. The effect of element size on the results was also examined with elements of 15mm to 70mm nominal size. Once this validation performed, other slabs could be modelled and compared with the experimental results.

3.2 Materials

VecTor2 software uses two dimensional finite elements to analyse concrete structures with rotating smeared cracks based on the Modified Compression Field theory (MCFT) and Disturbed Stress Field Model (DSFM) [11, 12]. Many options are available to model the material behaviour. The basic options were initially selected which consist of a tri-linear stress-strain relationship for steel as shown

in Figure 2a. The pre-peak and post-peak concrete compression behaviour (Figure 2b) is modelled according to Hoshikuma et al. [13]. The cracked concrete behaviour includes compression softening. For the tensile behaviour of concrete (Figure 2c), the stress-strain relationship is linear up to the tensile strength (f_t). Beyond this point, the tension softening effect is represented with a linear law driven by the cracking energy G_f . The tension stiffening effect is also included according to the model of Lee et al. [14]. All equations and references can be found in the VecTor2 Reference Manual [10].

VecTor2 gives the opportunity to define a bond-slip model as a trilinear law (Fig 3a). Pullout tests were carried out at Université Laval to determine the full bond-slip behaviour of the concrete/epoxy-adhesive/steel reinforcing bar interface. The concrete and reinforcement used for these tests were similar to those used for the slab tests. The tri-linear relationship used in VecTor2 is as follows: first elastic branch up to 30.0MPa bond strength, τ_1 , and 0.20mm slip, s_1 ; maximum bond strength, τ_2 , of 30.9MPa at 1.00mm slip, s_2 ; residual bond strength, τ_f , of 5.0MPa at 4.73mm slip.

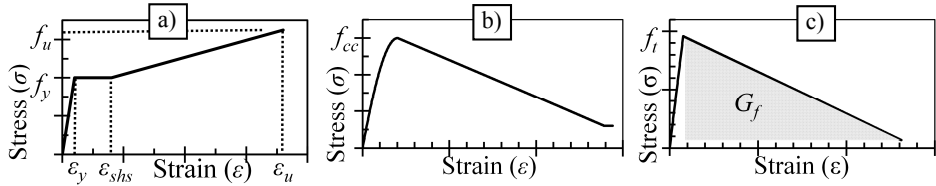


Fig. 2 Steel relationships (a) and concrete relationship in compression (b) and in tension (c)

3.3 Geometry and model

Figure 3b shows the boundary conditions and the final mesh used for slab PP3-R. The other slabs were modelled with similar meshes taking account of the different shear reinforcement details. All slabs were modelled with 2D membrane elements. Longitudinal reinforcing bars and shear reinforcement were modelled with discrete truss elements. Smearred reinforcement was used to model stirrups over the supports (slabs PP3). For slabs with bonded shear reinforcement, contact elements were used to model the bond interface with the epoxy adhesive. Otherwise, truss elements were perfectly linked to the concrete element nodes. Taking into account the symmetry of geometry and loading, half of the slab was modelled. Boundary conditions were imposed as follows, horizontal displacements are restrained at mid-span and vertical displacements are restrained at the support. The plates at the support and loading location were modelled using steel elements.

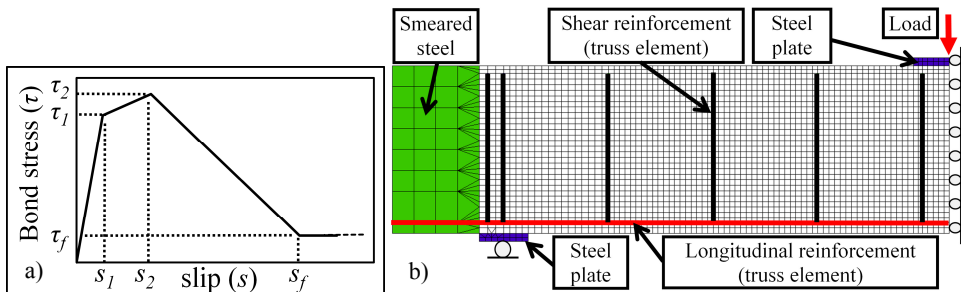


Fig. 3 Bond-slip model for epoxy (a) and final model of slabs PP3-R1 (b)

4 Results and discussion

4.1 Model validation

To validate the model, analyses were performed on the unstrengthened slab PP3-U1. Two types of failure modes can be expected for the shear-span to depth ratios used. Unstrengthened slender beams usually fail after the main shear crack opens and propagates. This propagation results in a rapid decrease of load carrying capacity with splitting cracks along the longitudinal reinforcement. The Canadian Standard uses a section model for this behaviour. However, with a relatively small shear span-to-depth ratio a concrete strut action can develop from the loading point to the supports which can enable the beam to carry load after major shear cracking occurs. For this mechanism, both crushing of the concrete strut and the rupture of the flexural reinforcement may lead to failure. This second mechanism is typically analysed using strut-and-tie model action. Because the geometry of slabs are close to

the limit where “slender” beams are considered to be “deep” beams (at approximately $a/d=2.5$), it can be anticipated that the slab behaviour can exhibit both strut-and-tie behaviour and sectional behaviour.

Figure 4 presents the effect of mesh size on the predicted failure modes. The left hand side vertical axis presents the ratio of the predicted shear capacities (sectional model) between a specified mesh size and reference mesh size of 15mm (single thick line). The right hand side vertical axis presents the ratio of the predicted shear capacities associated to the arch action (strut-and-tie model) between a specified mesh size and reference mesh size of 15mm (double thin lines). The effect on shear capacity ratio is presented with solid lines whereas ratio of displacements at failure is presented with dashed lines. For meshes with elements smaller than 35mm, the element size has very little influence on the load at which the main shear crack propagates and the value of the associated deflection. By comparing with the 15mm element size, the shear strength capacity is increased by 7% and 15% for 35mm and 70mm element size respectively. From Figure 4, it can be also observed that best results are obtained with 25mm element size or smaller. This observation is in accordance with recommendations relative to aggregate interlock. To best represent the aggregate interlock phenomena, authors [10] recommend a mesh with element overall dimensions close to the aggregate size, and not bigger than twice the aggregate size.

For the mesh size effect on the arch action shown in Figure 4 (right vertical axis and double thin lines), there is a strong mesh influence for models with elements larger than 40mm. The predictions with the 30mm mesh size are within 5% of the predictions with the 15mm mesh size while the predictions for the ultimate strength of the arch action and the deflections are overestimated by 45% and 90%, respectively for the 70 mm element mesh size. This can be attributed to the stresses present in the compressive areas with the compressive behaviour of concrete being strongly sensitive close to the peak of the stress-strain curve. An adequate numerical model should be able to capture this material non linearity. However, the numerical model in VecTor2 uses square elements with a linear interpolation. Therefore, the presence of several elements assures a better approximation of the behaviour and the progressive crushing failure of this zone.

In light of the previous observations and comments, adequate models should be meshed with elements of size 30mm or less. Crack patterns and past experience have also showed that best results are obtained with 25mm element size meshes. Thus, all numerical models in this study were built with 25mm specified nominal size element meshes.

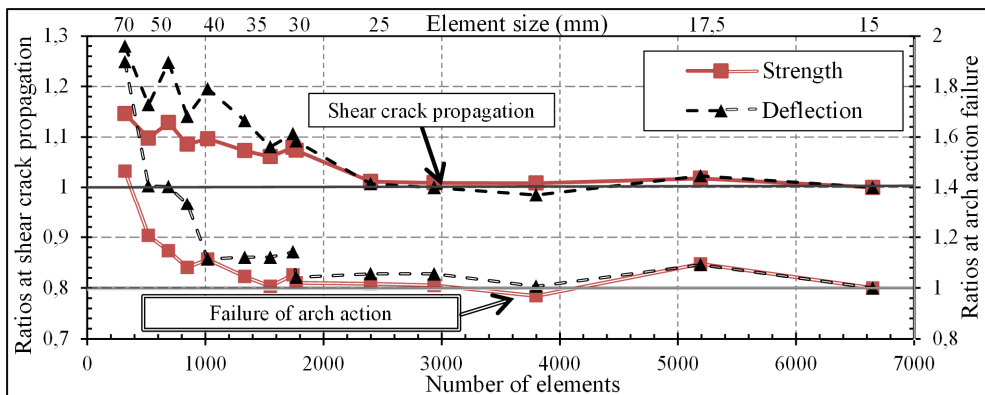


Fig. 4 Mesh size influence on shear carrying capacity and deflection for shear crack propagation and arch action, reference mesh of 15mm

4.2 Models results

Table 2 gives a summary of the experimental results, numerical results and both the Canadian Standard and *fib* Model Code predictions assuming that the drilled-in anchors are fully effective. The shear strength predictions using the provisions of both codes (V_{CSA} and V_{fib}) are in good agreement with experimental results for the unstrengthened slabs and the slabs with conventional stirrups. The average ratio of predicted shear strength to tested shear strength (V_{exp}) of these slabs is 1.090 and 1.085 for the Canadian Standard and *fib* Code, respectively. A very good average shear strength ratio and coefficient of variation of 1.035 and 0.071 was also obtained with VecTor2 for these same unstrengthened slabs and slabs with conventional stirrups. Codes provision and VecTor2 model give also good shear

strength predictions for slabs containing bonded shear reinforcement for low spacing ratio s_w/d_v of 0.61 (slab BC2). However, the code predictions are poor for slabs with spacing ratio s_w/d_v of drilled-in shear close to the maximum allowable limit of 0.75 (slabs PP3-R1 and PP3-R2). For all drilled-in bonded shear strengthened slabs, the shear ratio is 1.311 and 1.188 for the Canadian Standard and the *fib* Code, respectively. The shear strength predictions using VecTor2 for slabs PP3-R1 and PP3-R2 are very good. For slabs PP3-R1 and PP3-R2, VecTor2 predicts almost the same shear strength as the experimental results. For all drilled-in shear strengthened slabs with bonded reinforcement, the average and coefficient of variation are 1.026 and 0.020, respectively with the VecTor2 models. The load–deflection response of slabs BC1 and BC2 is also very well predicted by VecTor2.

Table 2 Summary of results

Slabs	V_{exp} [kN]	Δ_{exp} [mm]	V_{CSA} [kN]	V_{fib} [kN]	V_{FE} [kN]	Δ_{FE} [mm]	V_{CSA}/V_{exp}	V_{fib}/V_{exp}	V_{FE}/V_{exp}	Δ_{exp}/Δ_{FE}
PP3-U1	343.2	-	380.5	399.1	345.8	3.0	1.109	1.163	1.007	-
PP3-U2	341.3	-	370.8	389.0	334.0	3.0	1.086	1.140	0.979	-
PP3-R1	490.3	-	700.8	630.9	506.8	8.2	1.429	1.287	1.034	-
PP3-R2	505.2	-	711.8	658.5	526.2	8.3	1.409	1.303	1.041	-
BC1	767.3	10.6	823.1	729.8	811.3	6.7	1.076	0.954	1.118	1.261
BC2	755.6	11.9	828	735.3	718.7	6.4	1.096	0.973	1.003	1.073
Average							1.201	1.137	1.030	1.167
Coefficient of variation							0.141	0.131	0.047	0.114

Figure 5 shows the crack patterns for slabs BC1 and BC2 and the predicted crack patterns from VecTor2 using the smeared crack model. A very good match between the FE model predictions and the experimental cracking patterns can be observed. It can also be observed that the FE model predicts fewer shear cracks for slabs with bonded shear reinforcing bars than for slabs with conventional stirrups. For the case of bonded shear reinforcement, the main shear crack location at its intersection with the reinforcing bar determines the embedded length of shear reinforcement and the maximum stress that the shear reinforcement can carry at the crack (V_s). Thus, it is necessary to have a good cracking pattern prediction in order to predict the shear capacity.

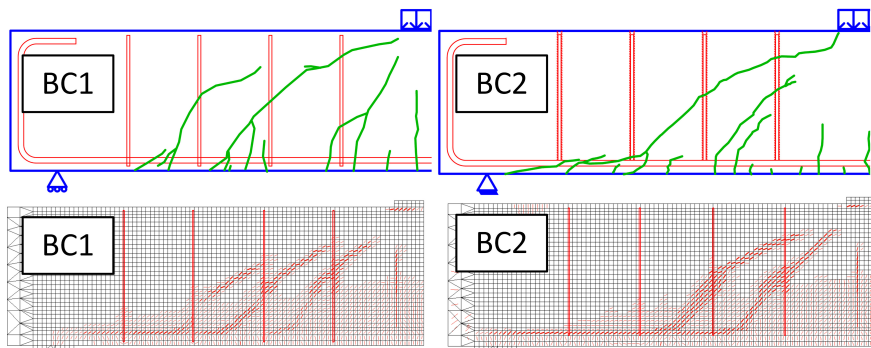


Fig. 5 Cracking pattern slabs BC1 (left), BC2 (right). Experimental (top), FE model (bottom)

5 Conclusions and future work

The main goal of this research is to develop a means of predicting the increase of shear strength of thick slabs subjected to various methods of shear strengthening. Results show that the 2006 Canadian Standard and the 2010 *fib* Model Code are suitable for predicting the results of slabs with conventional stirrups but can give unconservative results for slabs reinforced with drilled-in bonded reinforcing bars.

This paper presents finite element (FE) models used to predict the full shear versus deflection response of some of the slabs tested. One of the first steps was to validate the model based on experimental results from unstrengthened slabs and slabs containing conventional stirrups loaded up to

failure. In addition, more sophisticated models, with VecTor2 software, were used to simulate the rebar-epoxy adhesive-concrete interface were used to analyse slabs with drilled-in bonded shear reinforcement. It was concluded that for the FE analyses performed on unstrengthened slabs, there was a strong mesh size dependency for models with coarse meshes. However, models having mesh sizes smaller than 30mm give reasonably accurate results. This phenomenon can be attributed to the aggregate interlock and the accuracy of the stress distribution in the compressive zones close to the loading location. To get adequate results and cracking patterns while minimizing computing time, all models were built with elements having a maximum mesh size of 25mm.

Slabs with stirrups were modeled with truss elements perfectly bonded to the concrete while slabs with drilled-in bonded shear reinforcement were modeled with truss element linked to concrete using contact elements. The experimental observations corroborated the cracking pattern predicted by the FE model. It appears that the cracking pattern determines the embedded length of the drilled-in reinforcing bars and hence limits the steel stresses in the bonded shear reinforcement. Thus, it is important to have accurate predictions of the cracking pattern to determine the contribution of the drilled-in bonded shear reinforcement in resisting shear.

The experimental program has demonstrated the efficiency of drilled-in reinforcing bars that are bonded to the concrete, provided that the reinforcing bars are adequately anchored up to the shear failure. The research will pursue modelling of shear strengthened slabs and compare the numerical predictions with the experimental results. Based on these results, a hand calculation model taking into account of the bond-slip behaviour of the epoxy adhesive will be developed and could be introduced in design standards for the design of drilled-in bonded shear reinforcement.

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