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Punching Shear Behaviour of Externally Prestressed Concrete Slabs

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Abstract

The use of externally post-tensioned fibre reinforced concrete decks in highway bridge structures is seen as a viable option in the move towards the design and construction of high-performance structures. However, with the thin unreinforced deck slabs that may result, punching shear is a potential concern. An experimental program is described in which the punching shear behaviour of externally prestressed slabs is investigated, both with plain and fibre reinforced concrete specimens. Results indicate that significant improvements in strength, ductility, energy absorption and non-brittleness of failure can be achieved with fibre reinforcement. Nonlinear finite element analysis procedures are used to model the specimens, and reasonably accurate simulation of behaviour are obtained. Design code procedures are found to be unconservative in estimating the punching shear strength of these elements, whereas a commonly used analytical model is found to be overly conservative.

CE Database subject headings: Analysis, Bridge deck, Design, Fibre reinforced concrete, Finite element, Prestressed, Punching, Shear, Slabs

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Introduction

Although high-performance concrete has been used extensively in bridges in Canada and the USA since the mid 1990s, its application has been motivated primarily by concerns over durability. Consequently, the high compressive strength available has remained underutilized in bridges. An efficient use of the strength of high-performance concrete requires moving towards thinner cross section components. In many cases, a significant impediment to doing this is the presence of reinforcing steel, which imposes dimensional requirements related to clear cover and spacing between bars. The deck slabs of many box girder bridges, however, are prestressed in two horizontal directions; reinforcing steel is provided primarily for crack control. In such cases, it is possible to consider replacing the reinforcing steel by steel fibres, which would create opportunities for reducing the thickness of these components.

Punching shear is an important failure mode of bridge deck slabs. The contribution of an active, known precompression force (as opposed to a passive compressive force created by restraint) and of the presence of steel fibres to the punching resistance of thin slabs is not well understood. This paper presents the results of an experimental and analytical study undertaken to characterize the effect of these two factors on the punching shear behaviour of thin concrete slabs.

Previous Studies

Since the late 1950s, numerous experimental investigations have been undertaken to study the behaviour of laterally restrained slabs which, relative to unrestrained slabs, typically exhibited a large increase in load-carrying capacity. Wood (1961), for example, reported strengths greater than 10 times those predicted by yield-line theory. Taylor and

Hayes (1965) identified the effects of lateral edge restraint on the punching shear strength of slabs, and concluded that code requirements for punching shear needed to be revised to take the restraint effect into consideration. Aoki and Seki (1974) studied shear behaviour in the vicinity of concentrated punching loads; special attention was given to ‘arching action’, the mechanism which explains the tendency of restrained slabs to fail in punching rather than in flexure.

Mufti et al. (1993) investigated the possibility of producing a concrete deck slab that is entirely steel-free and capable of matching the load levels resisted by conventionally reinforced decks. Hassan et al. (2002) also studied the effect of prestressing on the punching strength of steel-free deck slabs. A system of external unbonded prestressing bars was used as the lateral confinement system, rather than the steel straps used by Mufti et al. (1993). The positive effects of prestressing were also previously reported by He (1992), who tested a 1/4-scale slab-on-girder bridge segment.

In many applications, the inclusion of steel fibres in concrete has proven to be beneficial to the over-all mechanical behaviour of the structure. Fibre reinforced concrete (FRC) shows an improvement in toughness, ductility, energy absorption, cracking resistance, and tensile strength (Shaaban and Gesund, 1994). In traditionally reinforced slab-column connections, with and without shear reinforcement, Swamy and Ali (1982) found that fibre reinforcement changed the shape of the punching failure surface and that the ensuing failure was more gradual and ductile.

The various approaches developed to predict the ultimate punching shear strength of concrete slabs can be divided into two categories: empirical equations, based on statistical analysis of tests results; and rational mechanistic models. The mechanistic

approach idealizes the failure mechanism, system geometry, and material properties to build a mathematical model that seeks to predict the punching shear strength. Hewitt and Batchelor (1975) proposed such a model to predict the behaviour of restrained circular slabs that fail in punching, based on an extrapolation of the work by Kinnunen and Nylander (1960). The model was further developed by Newhook (1997) for application to laterally restrained deck slabs of slab-on-girder bridges.

Little has been reported on the punching shear behaviour of externally-prestressed FRC slabs, either from an analytical or experimental perspective.

Experimental Program

An experimental program was undertaken comprising seven square externally-prestressed slab specimens. Four were constructed using steel fibre reinforced concrete (SFRC); the remaining three, serving as control specimens, were constructed of plain concrete. None contained conventional internal reinforcement, and all were externally post-tensioned in two directions.

Details of the test specimens are provided in Fig. 1. The nominal dimensions of the slab and loading plate were 1500 x 1500 x 127 mm and 200 x 200 x 50 mm, respectively. To minimize the effect of local stress concentrations at the points of application of the post-tensioning forces, sixteen C130x10 channels, each 204 mm in length, were cast integrally along the specimen edges as shown.

The specimens were externally post-tensioned using eight 32-mm diameter Dywidag bars in each direction. The force in each Dywidag bar was transmitted first to a vertical steel beam comprised of two C310x45 channels, and then to the specimen through a 204 x 170 x 38 mm plate bearing against a 32-mm-diameter round bar welded

to the C130x10 channels. [For specimen P-1, the loading plates acted directly against the specimen.] The post-tensioning system details are shown in Fig. 1. The points of application of axial load were chosen to facilitate the post-tensioning process and to create a condition as close as possible to a uniform axial stress state without interfering with the applied vertical load. Linear elastic finite element analysis indicated that a near uniform compressive stress was achieved across the widths of the specimens at the centre.

The slabs rested on continuous roller supports all around, giving simple-spans of 1350 mm in each direction. The transverse loading system for the specimens consisted of a vertical monotonically-increasing concentric load applied to a 200 mm square steel loading plate. The load was supplied by a 5300 kN capacity testing machine in displacement-control mode.

Linear variable differential transducers (LVDTs) were used to monitor the vertical and horizontal displacements of the specimen. The vertical displacement was measured using six to eight LVDTs; four were placed at the corners, mounted on the top surface of the slab and aligned with the centreline of the rollers, and two (four for F-1, F-3, F-4 and P-2a) were attached on opposite sides of the Baldwin machine head. The top and bottom horizontal LVDTs were used to monitor axial displacements and rotations of the edges; these were placed at the mid-span of each side, 9 mm away from top and bottom surfaces. Two electrical resistance strain gauges, with 5 mm gauge lengths, were applied to each Dywidag bar to measure the bar strains during post-tensioning and testing. These strains were used to monitor bar forces during the experiments. In addition, loads cells were

employed at the anchored ends of two of the Dywidag bars to corroborate forces determined from the strain gauge readings.

The concrete used to construct each test specimen was custom-batched with a targeted 28-day compressive strength of 50 to 60 MPa. Mix proportions are given in Table 1. To maintain an adequate workability for the fibre-reinforced concrete mix, the gravel/cement ratio was reduced, and the super plasticizer content was increased. Higher cement content was needed to reach the intended concrete strength. The actual concrete compressive strengths, as determined on test-days using standard cylinder tests, are reported in Table 2. Also given are the fracture energies for those specimens constructed from SFRC. [Note: The fracture energies were determined using notched prisms (150 x 150 x 530 mm) tested under 4-point bending according to the Italian Standard UNI-11039.]

Dramix RC-80/50-BP steel fibre was used in the amount of 1.0% by volume, for SFRC specimens F-1, F-2, and F-3. For Specimen F-4, the same steel fibre type was used in the amount of 1.5% by volume. This high-carbon wire fibre with hooked ends has a nominal length (L) of 50 mm and a nominal diameter (d) of 0.60 mm. The tensile strength was 2000 MPa, according to manufacturer specifications.

A primary test objective was to investigate the influence of steel fibres in enhancing the punching shear capacity of the slabs. In addition, the influence from the amount and pattern of prestressing used was also to be studied. The nominal prestressing forces initially applied to the specimens are given in Table 3.

In practice, for shear punching of the bridge slabs due to heavy trucks, the load distribution under the rubber tires could create curved boundaries. However, in this study,

for the sake of simplicity and comparison, as with the previous similar tests, a square loading plate was utilized in the test setups. Further studies are recommended to evaluate the potential influence of the loading conditions on the slab shear punching behaviour.

Test Results

All specimens failed in a punching shear mode, except Specimen P-3 which failed in a punching-flexural mode. The measured load-deflection responses are given in Fig. 2. The maximum load capacity, energy dissipation, and post-peak ductility and failure mode for each specimen are reported in Table 4.

Specimens P-1 and P-2a failed in a highly brittle punching mode. In each case, the punching failure was attained when a cone of concrete completely detached from the underside of the slab (e.g., see Fig. 3(a)) with no visible or audible warning. The suddenness of the failure was reflected in the load-deformation responses recorded (see Fig. 2(a)). With these specimens, the shear fracture on the compressed surface of the specimen occurred immediately adjacent to the loading plate. On the tension side, cracking was aligned with the inner face of the plate, with some irregularities at the corners. The average angle of inclination of the shear failure plane, measured from the horizontal, was 15° for Specimen P-1 and 12° for Specimen P-2a. The failure mode of Specimen P-3 was interpreted as a combined flexural-punching shear failure in which both flexural cracks (i.e., cracks underneath of the slab extending from the corners to the center) and punching shear cracks (i.e., cracks from near the edge of the loading plate on the top diagonally extending to the underside of the slab) were observed to form simultaneously at failure. On the compression surface, several longitudinal flexural

cracks formed approximately at mid-span, while punching shear cracks formed immediately adjacent to the loading plate.

All SFRC specimens (F-1, F-2, F-3, and F-4) failed by punching shear. The failures were characterized by cracks forming around the loading plate periphery followed by penetration of the loading plate. On the tension surface, no distinguishable shear fracture cone was evident although some longitudinal and diagonal cracks were observed (see Fig. 3(b)). Audible signs of distress heard during the loading process were attributed to the fibres debonding and pulling out across the widening cracks. The formation of a complete failure surface for Specimens F-1, F-2, and F-3 was gradual and occurred over a few minutes resulting in a moderately ductile failure. Specimen F-4 experienced a more brittle load-deformation response with a sudden drop in load capacity occurring just after the peak, as seen in Fig. 2(b). Considering the high prestress to which it was subjected, the more brittle response of Specimen F-4 might be due to its relatively lower concrete compression strength. With all SFRC specimens, the loading plate penetrated approximately 15 mm at failure.

In general, as the transverse load increases and the slab flexes and cracks, an in-plane expansion of the slab will occur. If this expansion is restrained, increased axial compression forces will be induced thus giving rise to ‘membrane action’ or ‘arching action’. In the specimens tested, the lateral straining was passively resisted by the post-tensioning system; this resulted in an increase in force in the post-tensioning bars and hence in the prestress on the slabs. At ultimate load, the increase in in-plane axial force was as much as 50% or more (see Table 3), significantly influencing load capacity and behaviour. The variation of the in-plane load corresponding to the applied vertical load is

illustrated for all the specimens in Fig. 4; the in-plane axial forces began to diminish once the specimen entered the post-peak regime.

Discussion

Comparing the results for both the normal concrete (P-1, P-2a, P-3) and fibre concrete (F-1, F-2, F-3 and F-4) specimens, it is seen that the level of prestress influenced all aspects of behaviour, including cracking, deflection, stiffness, and failure mode. The influences on ultimate load and absorbed energy are represented in Table 4. As one might expect, increasing the prestress level enhanced the ultimate load capacity and stiffness but had a negative effect on ductility. This conclusion is also clearly verified by the comparison of deformation and load capacities responses for Specimens P-1, P-2a and P-3 in Fig. 2(a). The increases in the ultimate load and stiffness can be attributed to the increased membrane action which, in turn, was affected by the level of the in-plane axial stress.

Comparing the behaviour of Specimen P-1 to that of Specimen F-1, and of Specimen P-2a to that of Specimen F-3, for the same prestressing levels, adding steel fibres ($V_f = 1\%$) resulted in the transition from a brittle to a ductile punching shear failure, and improved the post-cracking behaviour and post-peak load-carrying capacity. The improvements in the strain energy absorption capacity can be attributed to the energy absorbed in debonding, stretching and pulling-out of the fibres after cracking. In addition, the presence of fibres prevented the propagation of the shear crack from the tension zone to the compression zone (top surface), impeding the formation of the failure cone observed in Specimens P-1 and P-2a; Specimens F-1 and F-3 remained intact after failure. However, the ultimate punching resistance (P_{max}) increased by only 10%. It is

observed that fibre inclusion has less of a beneficial effect on ultimate load when combined with in-plane axial restraint. [Swamy and Ali (1982) reported an increase of 40% in the ultimate resistance corresponding to 1% of fibre volume in unrestrained specimens.]. As for Specimen F-4, with 1.5% fibre, the test results indicate only a 5% increase in the ultimate punching resistance compared to that of Specimen F-1 (similar concrete strength and prestressing levels but 1% fibre). This may be due to significant bond deterioration of the steel fibres at the crack locations caused by the shear punching failure mechanism under high prestressing loads.

Varying the axial stresses in one direction had only a minor influence on the general behaviour of the specimens tested. For Specimen F-1 and Specimen F-2, an inplane axial stress of approximately 6.0 MPa was applied in one direction, while the inplane axial stresses in the orthogonal direction were 6.0 MPa and 4.5 MPa, respectively. Both specimens experienced similar load-deformation behaviour, strain energy absorption capability, and failure mode. The failure load for F-1 increased by only 5% and the energy absorption capacity remained virtually unchanged.

In-plane axial force applied to the slabs can induce significant moments at large deformations especially for ductile slab such as those with fibres. Furthermore, in long term, creep and shrinkage of the concrete could result in reduction of the in-plane axial force, which would influence the punching shear capacity of the post-tensioned slabs. Further studies are recommended to evaluate these effects.

Numerical Modeling

The specimens were modeled for nonlinear finite element analysis (NLFEA) using program VecTor3. This program, developed specifically for the analysis of reinforced concrete solids, is based on the smeared rotating crack approach. Reinforced concrete is modeled as an isotropic material before cracking, and as an orthotropic material afterwards. The material models and constitutive relations utilized are based on the Disturbed Stress Field Model (DSFM) (Vecchio, 2000), which is an extension of the Modified Compression Field Theory (MCFT) developed by Vecchio and Collins (1986). The program is based on an iterative total stress and strain formulation in which secant moduli are defined and progressively refined according to current local stress-strain states.

Due to symmetry, only one-quarter of the typical test specimen needed to be modeled. The finite element mesh employed, shown in Fig. 5, consisted of 2039 constant-strain hexahedron elements and 2648 nodes. For better distribution of the imposed vertical load and to prevent local failure, the loading plate was also modeled. All nodes across the axes of symmetry were restrained against in-plane displacement. Nodes corresponding to the location of the roller supports (bottom layer at 75 mm from the edges) were restrained in the z-direction with ‘compression-only’ vertical truss members, thus permitting ‘lift-off’ in the modeling. Vertical loading was modeled as an imposed downward displacement at the node corresponding to the center of the loading plate. Axial stresses were applied as concentrated loads in both orthogonal directions, distributed over nodes along the edges. These in-plane axial forces were increased in fixed proportion to the applied vertical displacement. The constant increase rate of in-

plane axial force for each slab was determined based on the average increase rate of the prestressing force, corresponding to the vertical displacement, measured during the test.

To model the post-cracking tensile behaviour of fibre reinforced concrete, a cursory version of the Variable Engagement Model, developed by Foster et al. (2006), was used. As an example, an analysis was implemented using the foster model to estimate tension stress-strain response of a single fibre reinforced concrete element with material properties similar to that of the specimen F-1. Fig. 6 illustrates the results. For the sake of comparison, the tension response of a plain concrete with identical compression strength is also plotted. Except for the tension modeling, all other material modeling was done using the default options of the program.

The load-deformation responses determined from the finite element analyses are compared to the experimentally observed behaviours in Fig. 7. In general, the ultimate load capacity, pre- and post-cracking stiffness, ultimate ductility and energy absorption capacity were calculated with reasonable accuracy. In all cases, the failure modes were reasonably captured. The ratio of the experimental to calculated strengths had a mean of 1.01 and a coefficient of variation of 10% (see Table 5). Improved simulations might be expected once a more refined model for the post-cracking tensile stress response of SFRC can be developed and implemented into the finite element formulation.

Codes and Analytical Model Predictions

ACI 318-99 (Clause 11.12.2.2) has been used to estimate the punching shear strength of prestressed slabs according to the following equation:

$$V_c = \left(0.083 \sqrt{f'_c} + 0.3 f_{pc} \right) A_o d + V_p \quad (1)$$

where for the specimens in this study with square loaded areas,

V_c = available shear strength; N, $b_o = (4b + \pi d)$; mm, d = effective slab depth (need not be taken less than $0.8t$ for prestressed concrete slabs, which is the case for all the test specimens); mm, t = slab thickness; mm, f'_c = concrete cylinder strength; MPa, $\beta_p = 3.5$ (for all specimens); f_{pc} is the average value of axial stress in the two directions, which shall not be less than 0.9 MPa or greater than 3.5 MPa, and V_p = the vertical component of all effective prestress forces crossing the critical section which is zero for the slabs in this study.

Canadian standard CSA A23.3-04 (Clause 18.12.3.3) was also employed to determine the punching shear strength of the prestressed slabs, according to the following equation:

$$V_c = \left(\beta_p \phi_c \sqrt{f'_c} \sqrt{1 + \frac{\phi_p f_{cp}}{0.33 \phi_c \sqrt{f'_c}}} \right) b_o d + V_p \quad (2)$$

and where f_{cp} is the average value of axial stress in the two directions not to exceed 3.5MPa, and b_o is the perimeter of the critical section. Similar to the ACI equation, the second term in Eq. (2) represents the vertical component of the prestressing force, which is zero for all the specimens considered here. For the analysis, the resistance factor for concrete, ϕ_c and ϕ_p were taken as unity, β_p as 0.33, and d as $0.80t$. Table 6 summarizes the strength of the test slabs, determined accordingly.

It is important to note that the design values for f_{pc}/f_{cp} in the ACI and CSA codes are limited due to lack of test data for higher values. Further, f'_c , the concrete strength in both codes, cannot exceed 35 MPa. The main purpose of employing the ACI and CSA equations in this study is to evaluate their applicability for the specimens tested with higher concrete strength and higher in-plane axial stress. Therefore, these two limits have

not been considered in the calculation; the exact values of concrete compression strength and in-plane axial stress have been employed in Eq. 1 and Eq. 2.

Comparison reveals that more reasonable estimations were achieved using the ACI code equation than with the CSA code equation. The CSA equation overestimated the shear capacity for all the specimens except F-4 by about 20%. Both equations predicted the punching shear capacity with relatively more accuracy for Specimens P-1 and, surprisingly, Specimens F-1, F-2, and F-3. However, the code equations did not make any allowance for the beneficial influence of the fibres. For Specimen P-2a, the ACI equation also overestimated the punching failure load. The failure mode of Specimen P-3 was combined flexural-punching and both ACI and CSA equations understandably overestimated the load capacity. Overall, the ratio of experimental to calculated strengths had a mean of 1.02 and a coefficient of variation of 16% for the ACI formulation, and a mean of 0.81 and a coefficient of variation of 17% for the CSA formulation. Table 6 summarizes the results of shear capacity calculations using both the ACI and CSA equations.

The model proposed by Hewitt and Batchelor (1975) was also used to predict the punching shear capacity of the test specimens. This model is based on idealization of the slab as a circular slab that fails in punching, considering the fractured slab at failure. The main geometrical and material parameters considered in the analysis are: depth of the concrete slab, diameter or equivalent diameter of the slab, diameter or equivalent diameter of the loading area, reinforcement ratio, yield stress of the steel, and the compressive strength of the concrete. This model presumes an axisymmetric slab configuration and loading condition. Thus, for calculation purposes, the equivalent

diameter of the loaded area was taken as the diameter of the circle with the same perimeter as the square loading plate (255 mm), and the equivalent diameter of the slab was taken as the diameter of the largest circle which can be inscribed within the area of the slab (1350 mm). The resulting predicted strengths are given in Table 7. The ratio of the experimental to calculated strengths had a mean of 1.11 and a coefficient of variation of 12%. It was found that the Hewitt and Batchelor model provided slightly conservative estimates of strength in all cases except specimen P-1.

Conclusions

A series of externally post-tensioned concrete slabs, comprising specimens constructed of normal and fibre reinforced concrete, was subjected to punching shear loads in a specially devised testing rig. The test results, coupled with results of analytical and numerical investigations, suggested the following conclusions:

1. In-plane axial compressive stress, derived from external post-tensioning of the slabs, resulted in significant increases in punching shear load capacity.
2. Inclusion of fibre in the concrete also resulted in significant improvements in behaviour, particularly with respect to ductility, energy dissipation, and non-brittleness of failure.
3. Varying the prestressing in one direction, relative to the perpendicular direction, did not have a significant influence on behaviour (as deduced by comparing Specimens F-1 and F-2).
4. Reasonably accurate correlations were obtained between the nonlinear finite element analysis results and the test results.

5. While the ACI code equation provided reasonably accurate estimates for punching shear capacities of the specimens as a whole, it tended to overestimate the strengths of the plain concrete specimens and underestimate the strengths of the fibre reinforced concrete specimens.
6. The CSA A23.3 code relationship for punching strength of slabs gave generally unconservative estimates of the load capacity of the test slabs.
7. The Hewitt and Batchelor model gave generally slightly conservative estimates of the load capacity of the test slabs.

The results of the experimental investigation support the premise that use of post-tensioned, fibre reinforced, concrete is a viable option in optimizing the design of deck structures in high-performance concrete bridges.

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Notation

b_o	=	shear perimeter defined differently in different code, mm
d	=	effective slab depth, for all specimens $d = 0.8t$, mm
t	=	total slab thickness, mm
V_c	=	available punching shear strength, N
V_p	=	the vertical component of all effective prestress forces crossing the critical section which is zero for all the slabs in this study, N
f'_c	=	concrete cylinder strength, Mpa
f_{cp}	=	in-plane axial stress, prestressing, in each direction, for CSA code equations, MPa
f_{pc}	=	in-plane axial stress, prestressing, in each direction, for ACI code equations, MPa
β_p	=	in the ACI equation it is 3.5 and in the CSA equation it is 0.33 for all the specimens
ϕ_c	=	the resistance factor for concrete = 1.0 for analysis
ϕ_p	=	the resistance factor for prestressing tendons = 1.0 for analysis