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Abstract

The rehabilitation of structures is a complex process that involves many considerations. This paper presents results of a feasibility study on the use of precast prestressed concrete prisms (PCPs) as a near-surface mounted reinforcement for rehabilitation of cracked one-way slabs. The PCPs were cast with high-strength concrete and were concentrically prestressed by one 9 mm diameter stress-relieved seven-wire steel strand. Six identical one-way slabs with a length of 3000 mm, a width of 600 mm and a depth of 200 mm were tested. They were first loaded until cracking, then were fully unloaded and subsequently rehabilitated with four PCPs before loading them to failure. Deflected profiles and crack widths of the slabs before and after rehabilitation were compared in this research. This rehabilitation method reduced the magnitude of deflection and crack width of the samples. The cracks induced in the first phase of loading before PCP rehabilitation appeared fully closed after the PCP rehabilitation until five times the original cracking loads of the slabs.

Keywords: prestressed concrete prisms, high strength concrete, near-surface mounted reinforcement, rehabilitation, one-way slab

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Introduction

Strength and serviceability degradation due to environmental effects and increased loads due to traffic are common issues for all aging structures around the world. The most important concern for engineers is thus to improve durability and lifespan as well as to reduce maintenance costs of all structures. Maintenance includes all aspects required to keep a structure performing safely, in terms of load-carrying capacity, serviceability and durability. As a result, reinforced concrete structures often need to undergo rehabilitation and/or strengthening during their lifetimes.

Due to the sheer number of aging concrete structures worldwide, the demand for the repair and strengthening of outdated structures is increasing. It is not always applicable and economical to replace a current dysfunctional structure with a new one. Therefore, engineers and researchers are always looking for the most appropriate methods for retrofitting structures. Usually, strengthening an existing structure is more economical compared with the construction of a new one. Some traditional strengthening techniques used to improve performance of concrete structures during their service life include the application of overlays, increasing the number of supports to reach shorter spans for flexural members, using shotcrete, post-tensioning and using external steel or fibre-reinforced polymer (FRP) plates. Other feasible strengthening methods for existing concrete structures include using externally bonded reinforcement and near-surface mounted (NSM) reinforcement. When using NSM, reinforcing bars are embedded in grooves cut into the existing concrete to protect against thermal, environmental and mechanical damage. Other advantages of the NSM method include improved durability and a more optimized stress-sharing mechanism and fatigue performance. Grooves are saw-cut, in-concrete elements at a depth that avoids any damage to the main internal reinforcement. After the grooves are cleaned, epoxy is injected into them before the installation of the NSM reinforcement. This research uses precast

prestressed concrete prisms (PCPs) as NSM reinforcement. PCPs are precast high-strength concrete prismatic members concentrically pretensioned by reinforcement; in this study, the reinforcement was steel strand.

The use of PCP dates to the 1960s for reinforcing both concrete pavements (Dyckerhoff and Widmann Kommandigesellschaft 1963) and water tanks to delay cracking (Mikhailov 1958). Mikhailov (1958) reported the late formation of cracks in PCP-reinforced tanks, with a slow rate of widening. He also confirmed that cracks started to widen after a strain of about 0.0003 in concrete, and deformation further increased after the failure of the PCPs. The bond between concrete and prestressed prisms was examined by Evans and Parker (1955). They found that if the surface between the concrete and PCPs is rough, an adequate bond connection will be achieved to prevent any slippage. The authors also state that cracks are more prone to appear in reinforced concrete rather than in PCP-reinforced concrete. Similarly, Ozell found a good bond mechanism between prestressed prisms and cast-in-place concrete (Ozell 1957). Both studies specified a rough surface between the PCP surfaces and cast-in-place concrete as a requirement for a sufficient bond mechanism and monolithic action.

In the second half of the 1960s, nearly 40,000 linear meters of PCPs were produced for use in prestressed pressure pipes (Hoppe 1963). However, in the late 1970s, the production of the bars stopped, most likely because of improvements in post-tensioning processes, and no further reference to the production of such bars could be found in the literature. However, experimental work using PCPs in T-beams is reported by Hanson (1969), Zia et al. (1976), Chen and Nawy (1994) and Nawy and Chen (1998). Research was reported by Vogel and Svecova (2011) and Davoudi et al. (2008a) on the behaviour of PCP prestressed with carbon fibre-reinforced polymers

(CFRP). The presence of PCPs prestressed either with steel or CFRP increased the first cracking moment of the beams and reduced the crack width.

The behaviour of beams reinforced with prisms was compared to the behaviour of prestressed concrete beams. Chen and Nawy (1994) worked on thirteen rectangular, simply supported beams reinforced with prisms. Both beams and prisms were cast with high-strength concrete with a compressive strength of 84.4 to 98.5 MPa. The prisms were 38.1 x 76.2 mm, 38.1 x 101.6 mm and 76.2 x 76.2 mm and were prestressed with one 9.5 mm steel strand; the last set had a cross-section of 114.3 x 76.2 mm with two 9.5 mm steel strands. Although conventional concrete-reinforced beams exhibit a trilinear load-deflection behavior, PCP-reinforced beams indicated a bilinear performance. The cracking of prisms considerably changed the behavior of the reinforced concrete beams, and the load-deflection diagram experienced a substantial reduction in slope only after the cracking of the prisms. Nawy and Chen (1998) studied the behavior of four continuous concrete T-beams under flexure, reinforced with steel prestressed prisms. All beams had a length of 5,791 mm, with two equal, separate spans of 2,743 mm. The prisms fit each span, since they were 2,743 mm long with a 50 x 50 mm cross-section. Both beams and prisms were cast with high-strength concrete with a strength variation of between 84 and 94 MPa and with 9.5 mm diameter steel strands with a strength of 1862 MPa in the prisms. They were prestressed to $0.65f_y$, with an approximate prestressing loss between 23% to 34%. According to the results, all four beams demonstrated full moment redistribution over loading points and interior support location with the full establishment of plastic hinges. Their findings supported earlier research results that PCPs postpones cracking in the flange region in the negative moment zone.

Davoudi et al. (2008a) tested 6 continuous concrete T-beams, with 2 of these beams reinforced with CFRP prestressed PCP, 2 CFRP reinforced beams, and 2 steel reinforced beams.

The PCP had a cross-section of 50 x 50 mm and used $0.32f_{\text{rpu}}$ prestressing level. The results confirmed that prior to cracking of the PCP, the beams reinforced with PC had comparable deflection to the steel reinforced beams, while the CFRP reinforced beam had deflections 2.5 times larger. The research by Vogel and Svecova (2011) CFRP prestressed PCP reinforced simply supported beams. Half of the beams was subjected to fatigue loading. The bond strength on the surface of the PCP was measured in the experiment and it was found to be 0.23 MPa.

Based on the available literature, PCP were not used before for structural rehabilitation. However considering that each study available in the literature concluded that the presence of PCP reduced, or delayed the onset of cracking, this study investigated if the prisms are effective in closing existing cracks.

The main objective of this pilot research study is to investigate the performance and efficiency of PCPs for repair of cracked reinforced concrete one-way slabs. Figure 1 provides schematic view of strain and stress distributions for two slabs: Slab A and Slab B. Slab A is reinforced with steel reinforcement, while Slab B is reinforced with PCP in addition to the same amount of steel reinforcement as Slab A. The primary objective of using PCP for rehabilitation is to provide better crack control. Before cracking of concrete, behavior of both slabs is the same. After cracking of the slabs, in Slab A only the resultant force in the steel reinforcement remains in the tension zone, while in Slab B in addition to the resultant in the tension reinforcement, there is also a resultant of the forces in the PCP. To calculate this force, load-strain diagram was developed for each prism prestressing level and cross-sectional size. The strain that was calculated from strain profile of the section was then used to find the force in the PCP. The additional force in the tension zone of members with PCP reduces the strains in the tensile zone, reducing the curvature of the section and hence reducing deflections of these beams. The cracking load depends on the level of

prestress in the PCP, and its material. The cross-sectional size of the prisms limits the amount of prestressing in the prisms (Davoudi et al. 2008b). The effect of using these prisms on post-rehabilitation serviceability behavior of reinforced concrete slabs, specifically deflection and crack widths will be of main relevance to the findings of this research.

The objective of this research is to investigate the feasibility of using near surface mounted precast PCP for rehabilitation of concrete structures. This method uses materials and strengthening method that are commonly known to engineers and widely accepted. Some advantages of this method are that the self-weight of the structure does not increase, the strengthening system does not protrude above or below the structure, and the materials that are used have the same thermal properties as the original structure.

Experimental Program

This experimental program studied the serviceability behaviour of six 3000 mm long, 600 mm wide and 200 mm deep reinforced concrete one-way slabs under three-point bending. The program had two phases: Phase 1 focused on testing the slabs until cracking to a theoretical cracking load of 20 kN (adjusted for self-weight), while during Phase 2, the cracked slabs were repaired using four prestressed concrete prisms (PCPs) and thereafter were tested until failure. A total of twenty four 2000 mm long PCPs were cast for this project. The PCPs were cast using high-strength concrete, had square cross sections with sides of either 35 mm or 50 mm and used three levels of prestressing. This pilot study placed the prisms to maximize the effective depth of the section. In this study, the concrete cover used was smaller than the one required by current Canadian codes. To satisfy concrete cover requirements, the size of the prisms will need to be larger, or they will need to be inserted deeper in the cross section.

Prestressed Concrete Prisms

All PCPs were cast at the W. R. McQuade structural laboratory using a high-strength concrete mix that was prepared at the University of Manitoba using a mix design from earlier research work by Burg and Ost (1994). The mix design features a high value of silica fume to minimize chloride permeability and water absorption, especially in environments susceptible to deterioration. The same mix design was used in previous studies by Vogel and Svecova (2011) and Davoudi et al. (2208a; b). Further, due to the low water-cementitious ratio, a high-range water reducer is required to improve the workability of the concrete. In addition, to provide sufficient time to cast the PCPs and maintain workability, a retarder was used in the mix design. The concrete mix design can be found in Table 1. The average compressive strength of the concrete was 80.56 MPa 3 days after casting and 117 MPa at 28 days. Table 2 shows the gain in strength with time.

Table 2 uses the following nomenclature for the prisms based on their cross section size: small prism size of 35 x 35 mm prisms (SP) and large prism size of 50 x 50 mm (LP). The level of effective prestressing in the prisms is denoted at the end of the prism's nomenclature. For instance, a prism with a 35 x 35 mm cross section and 20 kN effective prestressing force is named SP-PCP-20, and a prism with a 50 x 50 mm cross section and 30 kN effective prestressing force is named LP-PCP-30, where the first two characters signify the dimension and the last two digits signify the effective prestressing force in the prism.

Table 3 presents comprehensive details of the prisms. A seven-wire stress-relieved 1860 MPa grade steel strand with a nominal diameter of 9.53 mm (conforming to ASTM A416M-16 guidelines) was used to concentrically prestress the PCPs.

Figure 2 shows the PCP prestressing bed details and dimensions. During prestressing, elongation of the steel strand was measured using a 2 mm strain gauge attached to the surface of the steel strand and connected to a data acquisition system. This experimental program used three levels of prestressing: 35 kN, 40 kN and 45 kN. Approximately 24 hours following the prestressing, the prisms were cast using high-strength concrete. The concrete cured for 14 days to provide sufficient time to develop its tensile strength before prestress release. The steel tendons were cut, and effective forces after losses were on average 20, 25 and 30 kN, corresponding to effective prestress of $0.27f_y$, $0.29f_y$ and $0.33f_y$, respectively. The amount of prestressing was kept smaller than the prestressing steel limits, due to the size of the prism cross section. The stresses in the concrete at release govern the design of these elements (Davoudi et al. 2008). Figure 3 shows the change in strand strains in PCPs with time before encasing them in the slabs. The drop in strains at 14 days indicates the prestress release day. Each prism lost approximately $160 \mu\epsilon$ due to seating in the prestressing setup and elastic shortening. The average prestress losses were 21.9% for the large prisms and 23.4% for the small prisms. These are relatively high, due to large elastic shortening and shrinkage losses. Losses of similar magnitudes were reported in the work of Nawy and Chen (1998).

Slab Details

A saw cut method was used to create grooves for installation of the PCPs. Saw cutting concrete is routinely performed when rehabilitating concrete slabs. However, to expedite this research project, Styrofoam prisms were placed inside the formwork in the areas where the prisms were to be inserted, prior to casting the slabs. The Styrofoam prisms kept the locations hollow where the PCPs were to be inserted, and they were easily removed after casting. Following the removal of the

Styrofoam, the remaining grooves were cleaned with pressurized air. Figure 5 shows the slabs with openings for the PCPs. These openings were wide enough to allow around 2 mm of space on either side of the prisms, which was required for development of a sufficient bond with the substrate; the grooves were longer than the prisms. The difference in the groove length was unintentional and may have contributed to the shear failure of the slabs; this will be discussed later.

The grooves were sand-blasted prior to the installation of the prisms to provide a rougher surface area and a better bond mechanism between the two concretes and the adhesive. Figure 6 displays the installation of the prisms in the slabs. The PCPs were installed in an overhead position when the slabs were on the supports, to simulate in-situ repair. Three sides of the PCPs were smeared with a layer of high viscosity epoxy and then inserted into the groove. A small bead of the epoxy was also applied to all three sides of the groove. The epoxy used for this application was a rapid setting, high modulus epoxy adhesive sold under the trade name of Dural Fast Set Gel. A temporary shoring system was used to keep the PCPs in the grooves while the epoxy was curing. The slabs were tested seven days after installation of the PCPs.

Loading Conditions and Instrumentation Details

Figure 7 shows a typical sample of the simply supported slabs tested in three-point bending. The testing was performed in two phases. During the first phase, the steel-reinforced slabs were loaded to a load of 20 kN. Only deflection at the mid-span was measured in this test phase. The first cracking occurred between 6 kN and 10 kN in all slabs. The cracks were traced, and the crack width was measured. The slabs were then fully unloaded and prepared for the second phase of testing.

For the second phase of this project, the now cracked slabs were fully unloaded, and while remaining at the supports, were repaired using the PCP. After the installation of the PCPs, the slabs

were tested until failure. The load was applied monotonically using displacement control, at a rate of 0.5 mm/min.

During the testing process, the applied load was measured using a load cell attached to an actuator. Four linear variable displacement transducers were used to measure the deflection of the slabs at different sections along the beam length. For each slab, six electrical strain gauges were mounted on the longitudinal steel reinforcement; one strain gauge was installed on the concrete surface at mid-span in the compression zone. Another strain gauge was also located at the mid-span of the seven-wire steel tendon in each PCP. PI-gauges were used to monitor the concrete strain and measure crack width throughout the test. Figure 7-b shows a photograph of the test setup.

Results and Discussion

The slabs were subjected to monotonic loading until failure, and the performance of all slabs is described in detail. Throughout the entire test, crack initiation and propagation was marked, and modes of failure were recorded.

Phase 1 – Cracking of the Slabs

The first phase of this testing was set up to provide a baseline behaviour of the slabs prior to rehabilitation. The load-deflection curves and crack widths from this phase of the program provide the behaviour of the control samples. In this phase, the slabs were loaded until the load of 20 kN was reached, and all cracks were marked. The first cracking occurred between the 6 kN and 10 kN loads in all slabs.

Figure 8 shows the load-deflection behaviour of slab LP-S-30 at mid-span under the load point in Phase 1. Early stage hairline cracks were visible at mid-span under the load point at around 10 kN, followed by cracking at quarter span. Slab load-deflection curves exhibited bilinear behaviour in the cracking phase with a change of stiffness at concrete cracking. Once a crack formed at around 10 kN at mid-span, the deflection started to increase rapidly. The loading was stopped at 20 kN. There was one linear variable displacement transducer on each side of the slab at mid-span. Figure 8 shows that there is good agreement between deflection readings on both sides. Finally, a permanent deflection of less than 2.0 mm was observed at the termination of the cracking phase. The beams were fully unloaded after this phase was completed.

Phase 2 – Ultimate Load Testing of Repaired Slabs

Each slab was repaired using four prisms with similar levels of prestressing and was left to cure for seven days. The slabs were tested under three-point bending, with the same load point and support locations as in Phase 1, until failure. Table 5 provides calculated nominal values and experimental ultimate loads for all slabs along with their modes of failure and strain in concrete and longitudinal reinforcement. The ultimate loads were calculated using strain compatibility analysis, and equivalent rectangular stress block at ultimate limit state. The force in the PCP was calculated taking into consideration the effective prestress of each prism. An assumption was made that all prisms in the same slab had the same level of prestressing. A load-strain graph was prepared for each level of prestressing and cross section size, which made the use of strain compatibility analysis straightforward. The beam ultimate load was calculated assuming flexural failure.

Strain in the Flexural Reinforcement

Figure 9 illustrates the strain in the nonprestressed steel reinforcement for both testing phases. The strains in Phase 1 constitute the first segment of the curve with loading and unloading, followed by reloading in Phase 2. The strain in this figure represents the values for all tested slabs at the mid-span under the load point. The graphs show that cracking in Phase 1 started at 10 kN. In Phase 2, the strain in all slabs changed linearly with load, up to the cracking loads of the PCPs, which varied from 50 kN to 65 kN for different prestressing load levels. Strain values at this stage for all slabs were similar, due to the fact that the transformed cross-sectional properties of the slabs were comparable. When the prisms cracked, strain in the flexural reinforcement experienced a rapid increase, with an approximately linear relationship, and continued linearly up to the yielding load. When the nonprestressed steel reinforcement yielded at load levels around 100 kN to 139 kN, depending on the level of prestressing in the PCPs, the strain changed considerably in the nonprestressed steel bars with a very small change in load, and a plateau was formed for most of the slabs.

Figure 10 demonstrates the load-strain relationship for all four PCPs embedded in slab SP-S-30 at mid-span. PCP 1 and PCP 2 were two middle interior prisms, while PCP 3 and PCP 4 were two external prisms along the sides. As can be seen from Figure 10, the strain in all PCPs changed linearly with load up to 70 kN, which was the cracking load for three PCPs (PCP 1, PCP 2 and PCP 3). PCP 4 was located at the exterior edge of the slab and cracked at a load level close to 60 kN, and strain gauge readings were lost at 63 kN. Furthermore, the effective prestressing strain in these prisms was around 2800 micro-strains. For the slab LP-S-30, shown in Figure 10, the maximum total strain in the prestressing strands was 8070 micro-strains at failure. Therefore, about 75% to 82% of the ultimate tensile strain of seven-wire steel strands was used before failure.

Strains in the steel reinforcement before and after rehabilitation are plotted in Figure 11 for slabs reinforced with large and small PCPs. It is evident for all slabs that strain values were reduced due to the presence of PCPs. Lower strain leads to smaller deflections and crack widths. The installation of PCPs helped the slabs retain their stiffness to the same levels as before the cracking of the concrete. Slab stiffness only changed after the cracking of the PCPs, which varied from loads of 50 kN to 65 kN, depending on the prestressing load levels and the yielding of nonprestressed steel reinforcement.

Load-Deflection Response

Figure 12 provides load-deflection behaviour for all slabs at mid-span under the load point. The figure contains deflection data for both loading phases of the slabs. The deflections in Phase 1 were plotted starting at the origin of the axial system, while the second phase deflection originated from the value of the permanent deformation in the system after unloading at the end of Phase 1. It needs to be pointed out that prior to the prism installation, the cross section of the slabs was 8% smaller, and as a result, the original stiffness is marginally smaller compared to that of the repaired slabs. Nevertheless, the load-deflection comparison between the two phases plotted in Figure 12 for the same slabs clearly shows that each slab in Phase 2 was able to retain its original stiffness up to loads three times the theoretical cracking load from Phase 1.

The plot in Figure 12 shows that in Phase 1, the slab deflections followed a linear trend prior to the cracking of the concrete at around 10 kN, and then the stiffness of the slab decreased until the load was terminated at 20 kN. The slabs were fully unloaded, and permanent deformations between 1.5 and 2 mm were measured. The highest stiffness after cracking in this phase was recorded for slab LP-S-30, while the smallest stiffness was recorded in this phase for slab LP-S-20.

The load-deflection graphs for Phase 2 originate from the point of permanent deformation measured at the end of Phase 1. Interestingly, after the rehabilitation, the stiffness of slab LP-S-20 was no longer the smallest in the group. It is clear from Figure 12 that the apparent stiffness, as observed from the load deflections, remained constant until the load of between 50 kN and 65 kN, depending on the size and prestress level of the prisms used in the rehabilitation. The slabs rehabilitated with small prisms cracked earlier compared to the slabs rehabilitated with large prisms. The nonlinearity of the load-deflection curve after the cracking of the PCPs was associated with the presence of nonprestressed steel reinforcement and their yielding at load levels from 100 kN to 139 kN for all slabs. Deflection increased at a faster rate after the yielding of nonprestressed steel reinforcement, as a result of lower values of stiffness after prism cracking.

The yielding of steel reinforcement plays a primary role in the nonlinear relationship of the load-deflection behaviour after the cracking of the PCPs. The more ductile behaviour of slabs SP-S-25 and SP-S-20 can be seen in Figure 12, as the nonprestressed steel reinforcement fully yielded in the two slabs. Table 6 summarizes the loads and deflections of the tested slabs at failure at mid-span along with the corresponding strain in the flexural reinforcement and extreme compressive strain in the concrete.

Cracking Behaviour in Phase 2

All slabs were fully unloaded before the installation of the prisms, but there was no effort placed into closing the cracks in the slabs before the PCP installation. The behaviour of members repaired by PCP is only minimally affected by the cracking of the slabs. This is because after cracking, the tension force in PCP-repaired slabs is carried by the nonprestressed reinforcement and PCP, and therefore, strains at the level of PCP reinforcement are smaller compared to strains at the same level in slabs without PCP. A more pronounced effect on the cracking behaviours is visible after

the cracking of the prisms and the yielding of the nonprestressed reinforcement. This is due to increased curvatures and reduction in tension stiffening of the prisms at loads above their cracking loads. This behaviour was confirmed by the observations from this experimental program.

Cracking of the rehabilitated slabs manifested in the development of a few very fine vertical flexural cracks at the mid-span. After the increase in load, cracks initiated at the quarter span and close to supports. Before the cracking of the PCPs, cracks in the slabs were narrow and hairline and were mostly arrested in the zone of the slab below the level of the PCP. After the prisms cracked, cracks propagated at a faster rate and moved towards the compression zone. As load increased, flexural cracks turned into flexure-shear cracks at the locations close to the end supports.

Figure 13 shows that there was a similar cracking pattern in all tested slabs. Cracking started to be visible to the naked eye at a load level of 50 kN to 65 kN, depending on the PCP prestressing load levels. A few very fine vertical flexural cracks developed at first in the mid-span. Early hairline cracks in the rehabilitated slab initiated at approximately 31% of the failure load for slab SP-S-20, 37% for slab SP-S-25, 36% for slab SP-S-30, 33% for slab LP-S-20, 39% for slab LP-S-25 and 41% for slab LP-S-30. Subsequently, several flexural-shear cracks widened in the area close to the section where the PCPs were introduced to the cross section. As the cracks stabilized, one of the inclined cracks widened and extended towards the top of the slab, ultimately causing the failure of the slab. The slabs experienced a brittle failure mode, occurring without warning shortly after the diagonal cracks developed.

The cause for the failure is not clear, but two possible causes were considered: a reduction of the cross section and a change in the amount of reinforcement at that location. The cut-off location of the PCP led to high stresses in the nonprestressed steel reinforcement and initiated a flexure-shear crack that led to the failure. In addition to the stresses at the cut-off location, the

concrete section was reduced by 8% in that location, because the grooves that were prepared for installation of the prisms were longer than the prisms. The space was filled with grout. Figure 13 shows the cracking pattern that led to the development of the failure mechanism.

To avoid such failures in the future, additional studies need to be carried out to investigate the minimum distance between the support and start/end of the prisms. In the future, the length of the grooves will also need to be as close to the length of the PCP as possible. No slip was observed between the steel reinforcement, the PCPs and the concrete in any of the slabs.

At the ultimate load, slabs LP-S-30 and SP-S-30 contained the fewest number of cracks after failure, due to the higher prestressing force provided in the PCPs. Crack spacing was measured using a tape measure, with an accuracy of ± 1 mm in all tested slabs over the cracked zone. Table 7 provides the average crack spacing and numbers of cracks formed along the length of all slabs. Based on the experimental values in Table 7, all slabs showed similar crack spacing and numbers of cracks, which demonstrates the same transfer length and bond mechanism for all of them. However, slabs reinforced with PCPs that had higher levels of prestressing showed fewer numbers of cracks and longer average crack spacing. Earlier research showed that a higher prestressing force delays cracking of the PCP and increases tension stiffening (Davoudi et al. 2008). Consequently, a reduced number of cracks was observed in slabs LP-S-30 and SP-S-30 compared to the other specimens.

Steel reinforcement typically provides adequate contact surface area with concrete and proportionally sufficient bond strength, due to its deformed surface texture and mechanical interlock with the concrete. Chemical adhesion is limited and not relied on as much as mechanical interlock for the bond strength of steel rebars. Prisms, on the other hand, have smooth surface areas and take advantage of the chemical bond to the epoxy, and as such, provide a satisfactory bond

with concrete. Figure 14 illustrates the development of the maximum crack widths at the mid-span. LP-S-30 had the narrowest crack width both prior to the cracking of the PCPs at 50% of the ultimate load and afterwards. SP-S-20 showed the widest crack width through the entire test, among all tested slabs, with a maximum crack width of 2.3 mm. But the load-crack width behaviour for slab LP-S-20 is not available, due to an error during installation of the Pi-gauges. The load-crack width relationship in Figure 14 indicates that a larger prestressing load and prism size corresponded to a smaller final crack width.

The load-crack width behaviour in Figure 14 is trilinear. The onset of cracks in the rehabilitated slabs did not change the stiffness of the slabs. The cracks in the slabs were not visible, and the existing cracks in the slabs were not widening prior to the cracking of the PCPs. There was a noticeable increase in crack width growth after the cracking of the PCPs at load levels from 50 kN to 65 kN. PCPs with a higher prestress level had higher tension stiffening, and hence, a smaller number of cracks. The last segment of the curve was initiated by the yielding of the nonprestressed reinforcement, as is shown, with cracks widening with a minimal increase in load.

Table 8 shows that after installation of PCPs in the slabs, cracks grew at a slower rate compared to the Phase 1 cracking. For instance, the crack width for slab SP-S-20 in Phase 1 at a load level of 20 kN was 0.21 mm, while after the installation of PCPs, the same crack width occurred at a load level of 50 kN. The same cracking behaviour and pattern was observed for other slabs.

Conclusions and Recommendations

This research program was conducted to investigate the serviceability performance of cracked one-way reinforced concrete slabs repaired using PCPs. Six full-scale one-way slabs were tested under three-point bending until failure, to inspect their deflection, crack width and flexural strength. The

initial design of all six slabs was identical, while each group of three slabs was repaired using PCPs with different cross section areas and prestressing levels. The prisms were cast with high-strength concrete and had a cross-sectional area of 50 x 50 mm or 35 x 35 mm and were concentrically prestressed with steel. The maximum applicable prestress force for PCPs at jacking is governed by the size of the PCP and the material properties of concrete to prevent the splitting of the prisms at the time of prestress release. Effective prestress levels between $0.27f_y$ and $0.3f_y$ were used in this experimental program. The following conclusions resulted from work presented in this paper:

- After rehabilitation of the slabs with PCPs in Phase 2, the apparent stiffness of the slabs remained the same as for the slabs in Phase 1 until the cracking of the prisms. The stiffness was reduced even further following the yielding of the nonprestressed reinforcement. This means that the rehabilitation scheme needs to be designed to have PCPs uncracked throughout the service life of the structure. Based on the levels of effective prestressing between $0.27f_y$ and $0.3f_y$, the prisms cracked at around 38% and 53% of the ultimate load capacity of the slabs.
- After PCP installation, the concrete slab cracks were hairline before the initiation of cracking in the PCP. These cracks were arrested at the level of the PCP until the load exceeded the cracking load of the PCP. The crack width decreased as a function of the prestressing level in the PCP from 0.2 mm at 50% of the ultimate load for slab SP-S-20 to 0.1 mm at the same load level for slab LP-S-30.
- There was no appearance of horizontal splitting cracks that would indicate bond failure along the length of the slabs throughout testing. However, some horizontal bond cracks were observed at the level of the PCPs at the time of failure.

- All tested samples failed in shear, with failure initiating at the ends of the prisms. The prism length was too short, and the abrupt change in the cross section at the cut-off of these prisms led to premature shear failure. Further work will be performed to study the effect of embedment length on the failure mode of slabs strengthened using this rehabilitation system.

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(a)

(b)

(c)

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Fig. 14 Maximum crack width at mid-span after repair with PCPs

Table 1: High-Strength Concrete (HSC) Mix Design

Parameters (/m ³)	Quantity
Cement Type I (kg)	564
Silica Fume (kg)	89
Coarse Aggs SSD ¹ (kg)	1068
Fine Aggs (kg)	593
HRWR Type F (L)	20.11
Retarder Type D (L)	1.46
Water (kg)	144
Water-Cement Ratio	0.26

¹Coarse aggregates are required to be in a Saturated Surface Dry (SSD) condition.

Table 2 High Performance Concrete Strength Development with Time

Age (Days)	Compressive Strength (f'_c) (MPa) – ASTM C39/C39M-14a					
	SP-PCP-20	SP-PCP-25	SP-PCP-30	LP-PCP-20	LP-PCP-25	LP-PCP-30
3	83.10	N/A	76.30	80.90	79.80	82.70
7	108.30	92.40	86.40	104.60	N/A	109.80
14	120.20	105.80	102.40	119.40	108.50	118.70
28	125.60	110.30	107.60	122.30	113.30	123.20

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Table 3. Types of Pre-Stressed Concrete Prisms (PCPs)

Batch No	Prism	Jacking Load (kN)	Effective Pre-stress (P_{eff}) (kN)	f'_c (MPa)	Length (mm)	Dimension (mm)
1	SP-PCP-20	30	20	100	2000	35 x 35
2	SP-PCP-25	35	25	98		
3	SP-PCP-30	45	30	102		
4	LP-PCP-20	30	20	95		50 x 50
5	LP-PCP-25	35	25	104		
6	LP-PCP-30	45	30	98		

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Table 4. Reinforcement Details of the Slabs before and after Rehabilitation

Specimen	28 days Concrete Strength (MPa)	Longitudinal Reinforcement	Transverse Reinforcement	$A_s(mm^2)$	$\rho_1(\%)$	$\rho_2(\%)$
SP-S-20	42	4 No 15	9 No 15	800	1.01	1.28
SP-S-25	44					
SP-S-30	45					
LP-S-20	41					
LP-S-25	42					
LP-S-30	43					

$\rho_1(\%)$: Reinforcement ratio BEFORE rehabilitation

$\rho_2(\%)$: Reinforcement ratio AFTER rehabilitation

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Table 5. Failure Load and Modes of Failure of Slabs

Slab	Failure Load (kN)		$P_{exp.} / P_{cal.}$	Strain at Failure (10^{-6})		Mode of Failure
	Exp.	Cal.		Concrete	Nonprestressed Steel	
SP-S-20	128.0	152.5	0.84	2010	11092	Shear Failure
SP-S-25	133.1	156.0	0.85	2907	8491	
SP-S-30	137.3	156.2	0.87	2935	4103	
LP-S-20	120.5	149.1	0.80	2299	3596	
LP-S-25	126.3	150.5	0.84	2382	2279	
LP-S-30	132.3	151.2	0.87	2620	2144	

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Table 6. Ultimate Load, Deflection and Strain in Nonprestressed Reinforcement and Concrete at Mid-Span

Slab	Failure Load (kN)	Deflection At Failure (mm)	Strain at Failure (10^{-6})	
			Concrete	Steel
SP-S-20	128	30.78	2010	11091
SP-S-25	133.1	25.96	2907	8491
SP-S-30	137.3	21	2935	4103
LP-S-20	120.5	23.76	2299	3596
LP-S-25	126.3	19.23	2382	2279
LP-S-30	132.3	19.94	2620	2144

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Table 7. Average crack spacing in all slabs

Slab	Number of Cracks	Average crack spacing [mm]
SP-S-20	15	128
SP-S-25	14	132
SP-S-30	11	141
LP-S-20	14	130
LP-S-25	14	135
LP-S-30	12	148

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Table 8: Comparison of Crack Width at Mid-Span Before and After Installation of PCPs

Slab	W_{cr1} (mm)*	W_{cr2} (mm)**	$P_{W_{cr1}}$ (kN)	$P_{W_{cr1}}^R$ (kN)***
SP-S-20	0.21	0.12	20.14	50.01
SP-S-25	0.27	0.19	20	44.34
SP-S-30	0.14	0.07	21.9	57.9
LP-S-20	N/A	N/A	N/A	N/A
LP-S-25	0.05	0.03	21.66	35.35
LP-S-30	0.05	0.04	22.15	33.6

* W_{cr1} (mm): crack width before installation of prisms

** W_{cr2} (mm): crack width after installation of prisms

*** $P_{W_{cr1}}^R$ (kN): load during failure phase when crack width was equal to W_{cr1}

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