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### REHABILITATION OF UNBONDED POST-TENSIONED SLABS WITH DIFFERENT BOUNDARY CONDITIONS

By

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# REHABILITATION OF UNBONDED POST-TENSIONED SLABS WITH DIFFERENT BOUNDARY CONDITIONS

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In Phase-I, a total of six unbonded post-tensioned (PT) slab specimens were tested. Three were simply supported two-way slabs with two-way post-tensioning (Specimens PTS-1, PTS-2, and PTS-6), whereas three other one-way slabs were tested with different boundary conditions (Specimens PTS-3, PTS-4, and PTS-5). The specimens were loaded to develop extensive cracks. Each of the specimens was then repaired with carbon fiber-reinforced polymer (CFRP) sheets using two different patterns. In Phase-II, the repaired specimens (PTS-1CR, PTS-2CR, PTS-3CR, PTS-4CR, PTS-5CR, and PTS-6CR) were tested again to reach their ultimate loads. The Phase-I and Phase-II research focused on the study of cracking patterns, reinforcing bar strains, tendon stresses, as well as the pressuredeflection and ultimate strength behavior of unbonded PT slabs. The investigation was also extended to the repair of these slabs with CFRP and the evaluation of the efficiency of CFRP repair of unbonded PT slabs. The research revealed that proper placement of CFRP sheets effectively restrained crack opening and crack growth and increased the flexural strength, stiffness, and deflection capacity of unbonded PT slabs, whereas there were modest increases in tendon stress.

### **KEYWORDS**

Boundary conditions; carbon fiber-reinforced polymers; post-tensioned concrete; rehabilitation; repair; slabs; strengthening; unbonded tendons.

#### INTRODUCTION

In recent decades, a large number of structures, which have aged, were built with one- and two-way PT. In most cases, the PT tendons were unbonded (PTI 2011). Some of the problems that existing buildings and infrastructures with

unbonded post-tensioning face today are excess loading, inadequate maintenance, and a lack of periodic repair and strengthening (PTI 2011). Some form of external reinforcement is needed to repair and strengthen these structurally deficient buildings and infrastructures. Many of the slabs can also be repaired or retrofitted by using external PT techniques and fiber-reinforced polymer (FRP) composites. The external post-tensioning, however, is often challenging for one-way slabs, due to the obstruction of one-way beams, and for two-way slabs, due to the limited clear story height of office and residential buildings. Replacing old strands with new internal strands is more difficult and cumbersome, even though the new strands are smaller. An addition of new tendons and a new layer of concrete could be an option; however, this makes the structure heavier, which contradicts the design philosophy of prestressed structures-namely, the pursuit of relatively light, crack-free, long-span structures. An FRP repairing and retrofitting system, particularly a carbon FRP (CFRP) system, is a suitable and convenient solution embracing such a philosophy. The FRP system saves time and costs. Also, it does not require significant alteration to the original floor slabs.

A handful of research programs on flexural and shear strengthening of the prestressed concrete members using FRP composites have been conducted in recent decades (for example, Meier and Kaiser 1991; Chakrabarti 1995; Chakrabarti et al. 2002; Di Ludovico et al. 2005; Chakrabarti 2005a, 2005b; Rosenboom et al. 2007; Ibrahim Ary and Kang 2012a; Kang and Ibrahim Ary 2012b). All of these tests focused on the study of bonded pre-tensioned and unbonded PT concrete beams. In particular, only limited research was conducted on unbonded PT slabs with FRP (Michaluk et al. 1998; Chakrabarti et al. 2007, 2009). Therefore, the behavior of unbonded PT members strengthened with FRP remains poorly understood, and standard configurations and formal procedures are yet to be established. Given this gap, an extensive experimental

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Specimens	PT tendons per unit width $A_{ps}$ , in. <sup>2</sup> /in. (mm <sup>2</sup> /mm)	Tensile mild steel per unit width $A_{ps}$ , in. <sup>2</sup> /in. (mm <sup>2</sup> /mm)	Compressive mild steel per unit width $A_{ps}$ , in. <sup>2</sup> /in. (mm <sup>2</sup> /mm)	$f_c',$ psi (MPa)		
PTS-1, PTS-1CR	16-1/4 in. strands each way 0.00554 (0.14)	4 x 4-4/4 WWM 0.01 (0.254)	6 x 6-10/10 WWM 0.0024 (0.06)	5931 (40.9)		
PTS-2, PTS-2CR	16-1/4 in. strands each way 0.00554 (0.14)	6 x 6-10/10 WWM 0.0024 (0.06)	6 x 6-10/10 WWM 0.0024 (0.06)	5963 (41.1)		
PTS-3, PTS-3CR	16-1/4 in. strands in span direction 0.00554 (0.14)	25-No. 3 at top and 9-No. 3 at bottom each fixed end 0.0265 (0.673)	6 x 6-10/10 WWM 0.0024 (0.06)	5726 (39.5)		
PTS-4, PTS-4CR	16-1/4 in. strands in span direction 0.00554 (0.14)	26-No. 3 at top and 9-No. 3 at bottom each fixed end 0.0276 (0.7)	6 x 6-10/10 WWM 0.0024 (0.06)	5959 (41.1)		
PTS-5, PTS-5CR	16-1/4 in. strands in span direction 0.00554 (0.14)	27-No. 3 at top and 9-No. 3 at bottom each fixed end 0.0265 (0.673)	6 x 6-10/10 WWM 0.0024 (0.06)	5362 (37)		
PTS-6, PTS-6CR	16-1/4 in. strands each way 0.00554 (0.14)	6 x 6-10/10 WWM 0.0024 (0.06)	6 x 6-10/10 WWM 0.0024 (0.06)	5223 (36)		

### Table 1—Summary of steel reinforcement and measured concrete strength for specimens

Note: WWM is welded wire mesh;  $f'_c$  is concrete compressive strength; concrete pouring dates are different.

research program was conducted on the application of CFRP for the rehabilitation of unbonded, PT one- and two-way slabs and repaired slabs using CFRP in this study.

The objectives of this experimental research are 1) to observe the general behavior of unbonded PT slabs before and after the application of CFRP with different boundary conditions; 2) to understand the relationship between internal reinforcement (mild steel and unbonded PT tendons) and externally bonded CFRP; 3) to observe and record crack propagation, strain, pressure, and deflection during testing; and 4) to quantitatively compare the ultimate strength of nonrepaired and repaired slabs.

#### MATERIALS

Quality concrete with a design compressive strength of 5000 psi (34.5 MPa) was used. Concrete mixtures were prepared according to ASTM C-94. The slabs were cured in their forms for 24 hours and then removed and continuously cured for at least 28 days. The concrete was proportioned using portland cement and fly ash with a water-cementitious material ratio (w/cm) of 0.34 (weight per volume ratio), resulting in a slump of about 3 in. (76 mm). The average concrete compressive strength of at least three specimens measured on the test date for each specimen is indicated in Table 1. The average measured concrete strength was 5690 psi (39.2 MPa), which is typical for PT slabs. Quarter-inch diameter seven-wire strands were used in each direction of the two-way slabs and in the span direction of one-way slabs. These were Grade 270 ASTM A-416 strands with a specified ultimate strength  $f_{pu}$  of 270 ksi (1860 MPa) and cross-sectional area  $A_{ps}$  of 0.036 in.<sup>2</sup> (23.2 mm<sup>2</sup>). The individual prestressing strands were inserted through 9/32 in. (7 mm) inner diameter plastic tubes. This process eliminated any bonding between the strands and the concrete.

Two different types of non-prestressed mild steel were used: 1) welded wire mesh (WWM) produced in accordance with ASTM A-185; and 2) ASTM A-615 deformed reinforcing bars. For the tension mild steel of two-way slabs, the WWM with a specified yield strength of 60 ksi (414 MPa) was used, whereas Grade 60 No. 3 ( $d_b = 3/8$  in. [9.5 mm]) reinforcing bars were used as tension reinforcement of the one-way slabs.

For strengthening, CFRP sheets were used. Three different types of CFRP sheet materials were applied: 1) CF130 high tensile carbon; 2) CF530 high-modulus carbon; and 3) CF160 high-modulus carbon. All the material properties indicated in Table 2 were obtained from the manufacturer (Structural Group, Inc. 2002). The second type (CF530) had measured values of ultimate tensile strength  $f_{u,frp}$  of 580 ksi (4000 MPa) and modulus of elasticity  $E_{frp}$  values of 54,000 ksi (372,300 MPa). The first and second type (CF160) with an ultimate strength

		Tensile modulus of	Design tensile strength,	Ultimate tensile
Specimens	Туре	elasticity, ksi (MPa)	ksi (MPa)	strength*, ksi (MPa)
PTS-1CR, PTS-2CR	CF130 high tensile carbon	33,000 (227,600)	550 (3790)	620 (4280)
PTS-5CR, PTS-6CR	CF530 high modulus carbon	54,000 (372,400)	550 (3790)	580 (4000)
PTS-3CR, PTS-4CR	CF160 high tensile carbon	33,000 (227,600)	510 (3520)	620 (4280)

### Table 2—Properties of carbon fiber-reinforced polymer (CFRP) used for specimens

\*Provided by manufacturer's design guide (Structural Group, Inc., 2002).

Note: CF160 (7.14 kips/in.;1.6 kN/mm) has twice the thickness of CF130 (3.57 kips/in.; 0.8 kN/mm).

Table 3—Dimensions for test specimens

Specimens	l <sub>1</sub>	<i>l</i> <sub>c1</sub>	l <sub>2</sub>	l <sub>c2</sub>	Boundary condition
PTS-1, PTS-1CR,					
PTS-2, PTS-2CR,	9 ft 0.5 in. (2756 mm)	8 ft 10 in. (2692 mm)	9 ft 0.5 in. (2756 mm)	8 ft 10 in. (2692 mm)	Pin-pin
PTS-6, and PTS-6CR					
PTS-3, PTS-3CR	10 ft 8 in. (3251 mm)	8 ft 8 in. (2642 mm)	8 ft 8 in. (2642 mm)	8 ft 8 in. (2642 mm)	Fixed-fixed
PTS-4, PTS-4CR	9 ft 10 in. (2997 mm)	8 ft 9 in. (2667 mm)	8 ft 8 in. (2642 mm)	8 ft 8 in. (2642 mm)	Fixed-pin
PTS-5, PTS-5CR	8 ft 10 in. (2692 mm)	8 ft 10 in. (2692 mm)	8 ft 8 in. (2642 mm)	8 ft 8 in. (2642 mm)	Pin-pin

Notes:  $l_1$  is slab length in span or one direction;  $l_2$  is support center-to-support center length in span or one direction;  $l_2$  is slab length in transverse or other direction; and  $l_2$  is support center-to-support center length in transverse or other direction.

 $F_{u,frp}$  of approximately 7.14 kips/in. (1.6 kN/mm) had twice the thickness of the first type (CF130) with an  $F_{u,frp}$  of approximately 3.57 kips/in. (0.8 kN/mm). The ultimate strengths  $f_{u,frp}$  in Table 2 were calculated as  $F_{u,frp}$  times the unit width of 1 in. (25.4 mm), divided by the CFRP thickness (for example, 0.0058 in. [0.146 mm] for CF130; 0.0115 in. [0.292 mm] for CF160 per ply, where the thickness of the CFRP impregnated with epoxy [saturant] was used). The design strengths in Table 2 were determined as the average ultimate strength minus three standard deviations of the measured values of  $f_{u,frp}$ .

### **TEST SPECIMENS**

An experimental program was divided into two phases: 1) Phase-I; testing control specimens of three two-way PT slabs and three one-way PT slabs; and 2) Phase-II; testing the same specimens after repairing using CFRP sheets. Table 3 summarizes the dimensions and boundary conditions of each specimen. Six control slabs were nonrepaired specimens labeled as PTS (Post-Tensioned Slab Specimen). Of these six, three were two-way slabs (PTS-1, PTS-2, and PTS-6) and three were one-way slabs (PTS-3, PTS-4, and PTS-5). Note that testing of PTS-6 (additional two-way slab specimen) was planned and conducted after the completion of testing of the first four specimens, and that all the damaged specimens were repaired with CFRP sheets and retested. The six repaired slabs were labeled as CR (for



Fig. 1—Fixed-fixed condition (PTS-3).

example, PTS-1CR; Post-Tensioned Slab No. 1 with CFRP Repair). The two-way slab specimens (PTS-1, PTS-2, and PTS-6) were simply supported on four sides of the slab. The one-way slab specimens had three different boundary conditions on two span ends: 1) PTS-3 and PTS-3CR had fixed conditions on both ends; 2) PTS-4 and PTS-4CR were simply supported on one end and fixed on the other end; and 3) PTS-5 and PTS-5CR were simply supported one-way slabs. The test installations to achieve the designated boundary conditions are as shown in Fig. 1 and 2.



Fig. 2—Pin-support portion of PTS-4.



Fig. 3—Test setup for two-way slabs under uniformly distributed area loads or pressure. (Note: 1 ft = 305 mm; 1 in. = 25.4 mm.)

Slab thickness was 3 in. (76 mm) for all specimens. The two-way slabs (PTS-1, PTS-1CR, PTS-2, PTS-2CR, PTS-6, and PTS-6CR) had a footprint of 9 ft 0.5 in. x 9 ft 0.5 in. (2.76 x 2.76 m) (Fig. 3) and were internally reinforced using mild steel and unbonded PT tendons in each direction as indicated in Table 1. The clear span length in each principal direction was 8 ft 10 in. (2.7 m) for PTS-1, PTS-1CR, PTS-2, PTS-2CR, PTS-6, and PTS-6CR (Fig. 4). Mild steel



Fig. 4—Draped tendon profiles for test specimens. (Note: 1 ft = 305 mm; 1 in. = 25.4 mm.)



Fig. 5—Tendon and mild steel layout for PTS-6.

wire mesh measuring  $7 \times 7$  ft (2.13 x 2.13 m) was placed at the compression surface of each slab, mainly to prevent damage during transportation (Fig. 5). The amount of mild steel, which varied for each specimen, is provided in Table 1. Two different sizes of the WWM mild steel were used (Fig. 6): 1) 4 x 4 - 4/4 with cross-sectional areas  $A_s$  of 0.01 in.<sup>2</sup> (6.45 mm<sup>2</sup>) per unit inch width; and 2)  $6 \times 6 - 10/10$  with an  $A_s$  of 0.0024 in.<sup>2</sup> (1.55 mm<sup>2</sup>) per unit inch width. The WWM



Fig. 6—Welded wire mesh (WWM) mild steel used for two-way slabs.



Fig. 7—No. 3 deformed mild steel used for one-way slabs. (Note: 1 ft = 305 mm; 1 in. = 25.4 mm.)

of  $6 \ge 6 - 10/10$  was placed as compression reinforcement at the bottom of the two-way slabs.

The one-way slabs (PTS-3, PTS-3CR, PTS-4, PTS-4CR, PTS-5, and PTS-5CR) had various footprints depending on the support boundary conditions as indicated in Table 3 and Fig. 4. Twenty-five No. 3 ( $d_b =$ 3/8 in. [9.5 mm]) tension bars were placed at a spacing of 4 in. (102 mm) in the span direction of the one-way slabs (Fig. 7), and nine No. 3 bars were placed as tension reinforcement at a spacing of 11 in. (280 mm) at the fixed end of the one-way slabs. Additionally, the 6 x 6 – 10/10 bottom wire meshes were used for all one-way slab



Fig. 8—Tendon layout for two-way slabs.

specimens to prevent cracks during installation of the specimens. Overall, the amount of bonded steel was determined to obtain the balanced failure mode (this was done to see whether or not CFRP is effective even with a small degree of steel yielding), and the number of tendons was determined not to make any initial cracks due to excessive camber under applied PT forces.

Figure 4 shows the draped tendon profiles used for the specimens. Figures 8 and 9 show the PT reinforcement layout plan for the specimens. A total of 16 post-tensioning tendons were placed in each direction of the two-way slabs and in the span direction of the one-way slabs (Fig. 5 and 10). The spacing of uniformly distributed tendons was 6 in. (152 mm) for the two-way slabs; thus, the cross-sectional area of the tendons per unit width was  $0.006 \text{ in.}^2/\text{in.}$  $(0.15 \text{ mm}^2/\text{mm})$ . For the one-way slab, two tendons were grouped with a spacing of 2.25 in. (57 mm) between each tendon, and the two-tendon group was then uniformly distributed with a spacing of 12 in. (305 mm) between the groups (Fig. 9 and 10). PT tendons were stressed to approximately  $0.7 f_{vu}$  before transfer (that is, jacking stress  $f_{pi}$ ), resulting in approximately  $0.65 f_{vu}$  after transfer (that is, initial stress  $f_{vi}$ ). Note that the initial stress  $f_{vi}$  is almost the same as the effective stress fpe in this research, as there are minor long-term changes in tendon stress. After



Fig. 9—Tendon layout for one-way slabs. (Note: 1 ft = 305 mm; 1 in. = 25.4 mm.)



Fig. 10—Tendon and mild steel layout for PTS-3 and PTS-4.

the Phase-I test was completed, each of the six specimens was repaired with CFRP and high-strength adhesive (epoxy) in accordance with ACI 440.2R-02 recommendations (Fig. 11 and 12). Prior to placing the CFRP sheet, the substrate was cleaned and primer was applied: a steel coarse brush attached to the hand drill was used to smooth and remove concrete deposit from the top. This method provided strong bond between concrete surface and CFRP. Then, the slabs were physically repaired (for



Fig. 11—Diagonal scheme for CFRP attachment (PTS-2CR).



Fig. 12—Orthogonal scheme for CFRP attachment (PTS-1CR).

example, filling cracks and applying sealer). The two-way slabs were repaired using two different FRP-strengthening schemes: 1) a diagonal scheme; and 2) an orthogonal scheme. Two plies of CFRP fabric sheets were placed for both schemes. The diagonal scheme, which was used for PTS-2CR, is shown in Fig. 13, and the orthogonal scheme, which was used for PTS-1CR and PTS-6CR (parallel to slab edges), is shown in Fig. 14. The one-way slabs (PTS-3CR, PTS-4CR, and PTS-5CR) were strengthened with straight CFRP sheets in the slab top (note that loading is applied from the bottom) and in the bottom tension zone over a quarter of the clear span at the support (Fig. 15). Details of CFRP materials are given in Table 2.

#### **TEST SETUP AND TESTING**

Two-way, simply supported, PT slabs were tested under uniformly distributed area loads or pressure. The



Fig. 13—Diagonal scheme for CFRP attachment (PTS-2CR). (Note: 1 ft = 305 mm.)



Fig. 14—Orthogonal scheme for CFRP attachment (PTS-1CR and PTS-6CR). (Note: 1 ft = 305 mm; 1 in. = 25.4 mm.)

loading frame is shown in Fig. 3 and 16. The area loads or pressure were applied upward by using a hydraulic water bag (the top of the slab at midspan was referred to as the tension side). The water pressure was gradually increased at approximately 1 psi (7 kPa) increments. One-way PT slabs were also subjected to uniformly distributed area load (water pressure), which was exerted vertically upward.

The control slabs were loaded to the ultimate (Phase-I); however, the slabs were not loaded to a point where they could become unrepairable for safety. Phase-I testing stopped when the slabs reached one of the following three criteria: 1) when excessive cracks were visually observed; 2) when the deflection in the slab reached close to L/120, where L is the span length; and 3) when the PT stress reached nearly 75 to 85%  $(0.75f_{pu}$  to  $0.85f_{pu}$ ) of the ultimate tensile strength. It was intended that the specimens would not completely fail during the Phase-I testing. The point of excessive cracking was close to the threshold of either of the other two criteria.

The same criteria were used for the repaired slabs (Phase-II), except for the first criteria. At the time the Phase-II test was stopped, the slabs were deemed semielastic. No crushing of concrete occurred until the ultimate stage. As noted previously, the cracked slabs (all six specimens) were then repaired with CFRP sheets and tested again. The same criteria to determine the ultimate load were used for the repaired slabs.

During the testing, all readings were taken using an automated data recording system. The test measurements included pressure, deflection, strain (in mild steel), and a change in PT forces. The applied pressure was monitored by the pressure gauge. The displacement gauges used to measure the deflection were linear variable differential transformers (LVDTs) with 4 in. (100 mm) of travel. Strain gauges were mounted in the mild steel at the midspan of the slab. Load cells were placed behind the anchor plates of the unbonded PT tendons to record the PT forces and stresses, and the increments of those forces and stresses during the PT and external loading. The strain gauges were attached at midspan of the slab.

### TEST RESULTS AND DISCUSSION Cracking

The compression (bottom) surface of each of the test specimens was not accessible for observation of cracks while testing was in progress. After the first crack appeared in the tension (top) surface, additional cracks were marked and recorded (Fig. 17). As expected, diagonal cracking patterns were observed in the two-way control slabs (PTS-1, PTS-2 and PTS-6). Since the square panel with the same reinforcing details in two principal directions was tested, the crack patterns were symmetrical with respect to both principal axes (Fig. 17 and 18). The symmetric



Fig. 15—CFRP attachment for PTS-3CR and PTS-4CR. (Note: 1 ft = 305 mm.)



Fig. 16—Water pressure loading test frame.

cracks observed from the two-way slabs indicate that the area load/pressure was quite uniformly applied on the slabs. For the one-way control slabs (PTS-3, PTS-4, and PTS-5), flexural cracks were focused on the tension (top) surface at the location where positive moment was the largest (for example, midspan for PTS-3 at approximately 3 ft 9 in. (1.14 m) from the simply supported end for PTS-4) (Fig. 19). After the test, the bottom surfaces of the slabs were examined. Almost no cracking was observed



Fig. 17—Crack patterns (PTS-1).

on the bottom surface of the control or repaired two-way slabs (that is, no concrete crushing was noted). In the first phase of testing of control specimens, the loads were not applied to the collapse level for safety reasons. It is noted that special safety precautions are essential in the testing of

an unbonded PT system. Also, the use of a hydraulic water bag warrants further precautions.

For the repaired slabs (PTS-CR specimens), cracks were not visible on the tension (top) surface during testing, as the surface was covered by CFRP. In some of the exposed areas, the old cracks opened up. No debonding of the CFRP sheets was observed at the ultimate loading stage.

### Pressure-deflection behavior

The comparison of deflection values between control (nonrepaired) and repaired specimens is shown in Table 4 and Fig. 20 to 25. In general, the control slabs behaved linearly during the initial stages of loading. A hairline



Fig. 18—Crack patterns (PTS-6).

crack (tension crack) for any one test slab was defined as the first crack which appeared on the top of the slab and corresponding pressure was defined as pressure at first cracking (Table 5). Once the two-way slabs cracked under approximately 2 to 4 psi (0.014 to 0.028 MPa) pressure, they exhibited reduced flexural stiffness as evidenced by the pressure-deflection relationships shown in Fig. 20, 21, and 25. The second stage of the linear behavior of cracked elastic slabs continued until approximately 4.5 to 6.5 psi (0.031 to

		Measured	$\Delta_{ ext{repaired}}$
	External water	deflection $\Delta$ ,	$\overline{\Lambda}$
Specimen	pressure, psi (MPa)	in. (mm)	non-repaired
PTS-1	5.2 (0.0359)	0.45 (11.5)	1
PTS-1CR	5.2 (0.0359)	0.40 (10.3)	0.9
PTS-2	5.2 (0.0359)	0.31 (7.9)	1
PTS-2CR	5.2 (0.0359)	0.20 (5.1)	0.64
PTS-3	5.2 (0.0359)	0.85 (21.6)	1
PTS-3CR	5.2 (0.0359)	0.6 (15.2)	0.71
PTS-4	5.2 (0.0359)	1.52 (38.6)	1
PTS-4CR	5.2 (0.0359)	1.05 (26.8)	0.69
PTS-5*	5.2 (0.0359)	2 (50.8)	1
PTS-5CR	5.2 (0.0359)	0.69 (17.5)	0.34
PTS-6	5.2 (0.0359)	0.45 (11.4)	1
PTS-6CR	5.2 (0.0359)	0.39 (9.8)	0.86

Table 4—Measured deflection at center of slab at 5.2 psi (0.0359 MPa) pressure

\*Excessive cracking.



Fig. 19—Crack patterns (PTS-3 and PTS-4). (Note: 1 ft = 305 mm; 1 in. = 25.4 mm.)



Fig. 20—Pressure-deflection relationship (PTS-1 and PTS-1CR).

Fig. 22—Pressure-deflection relationship (PTS-3 and PTS-3CR).



Fig. 21—Pressure-deflection relationship (PTS-2 and PTS-2CR).

Fig. 23—Pressure-deflection relationship (PTS-4 and PTS-4CR).

0.045 MPa) pressure was applied. As the flexural tensile cracks increased in number and width, the pressure-deflection curves started to show trilinearity and slope reduction. Subsequently, a significantly reduced stiffness was noted. This trilinear pressure-deflection behavior was similar for each two-way slab specimen. The strength of PTS-1 with a larger amount of mild steel was greater than that of PTS-2 by approximately 25%.

Similar behavior was noted for the one-way slabs. The boundary condition affected the pressure-deflection behavior. As the number of simple supports changed from 0 to 1 to 2, the stiffness and load-carrying capacity were reduced. Testing of PTS-5 with two simple supports was stopped due to excessive cracking. The curves of the PTS-3 and PTS-4 specimens also became nonlinear rather than trilinear, while PTS-5 exhibited a distinctly bilinear



Fig. 24—Pressure-deflection relationship (PTS-5 and PTS-5CR).

Fig. 25—Pressure-deflection relationship (PTS-6 and PTS-6CR).

	-		u	
Specimen	Pressure at first cracking $P$ , psi (MPa)	Ultimate pressure P . psi (MPa)	$\frac{P_u}{P_{cr}}$	$\frac{P_{u\_repaired}}{P_{u\_non-repaired}}$
	(25*(0.044))	( 5 (0.045)	1.02	NT A
P15-1	0.35 (0.044)	6.5 (0.045)	1.02	NA
PTS-2	4.8* (0.033)	5.2 (0.036)	1.08	NA
PTS-3	5.65* (0.04)	6.5 (0.045)	1.15	NA
PTS-4	4* (0.028)	5.4 (0.037)	1.35	NA
PTS-5	2* (0.021)	4.17 (0.029)	2.09	NA
PTS-6	3.5* (0.024)	5.5 (0.044)	1.57	NA
PTS-1CR	N/A	9.2 (0.063)	N/A	1.42
PTS-2CR	N/A	9.8 (0.068)	N/A	1.88
PTS-3CR	N/A	10.9 (0.075)	N/A	1.68
PTS-4CR	N/A	7.9 (0.055)	N/A	1.46
PTS-5CR	N/A	7.48+(0.052)	N/A	1.79
PTS-6CR	N/A	8.88 (0.061)	N/A	1.61

### Table 5—Measured values for pressure at first cracking P<sub>c</sub> and ultimate pressure P

\*Based on visual observation.

<sup>+</sup>Testing was prematurely stopped for safety reason; thus, 10% of last measured value was added. Note: N/A is not available.

pressure-deflection relationship. This indicates that the initial stiffness of unbonded PT slabs can be improved by increasing the fixity of the end supports. This point is especially important in terms of the serviceability of such slender PT members. Thus, in order to minimize floor vibrations, etc., application of restrained boundary conditions is highly recommended. After the Phase-I testing of control specimens and release of water pressure, it was noticed that the residual deflections were negligible due to the restoring force provided by the PT tendons. These measurements are also noteworthy in that unbonded PT structures possess a high elastic deformation-restoring capability even after considerable concrete damage. Therefore, there was no additional



*Fig.* 26—*Tendon load variation with increasing pressure (PTS-3).* 



Fig. 27—Reinforcing bar strain variation with increasing pressure (PTS-3).

step related to zeroing slab deflection before applying CFRP sheets. All LVDTs were removed prior to the application of CFRP sheets on the surface of specimens, and then the LVDTs were reinstalled. At this point, initial midspan deflections were reset to zero.

The deflection profiles for the strengthened slabs primarily show essentially linear or slightly nonlinear behavior without sharp turning points, whereas the loadcarrying capacity was considerably increased (by approximately 17 to 88%) (Table 5). For the two-way slabs, the stiffness was recovered up to that of the control slabs. For the repaired one-way slabs, the stiffness also became equivalent to that of the original one-way slabs or even superior to the nonrepaired slabs with simple support (PTS-3CR, PTS-4CR, and PTS-5CR). In particular, the simply supported PTS-5CR had less deflection at the ultimate load of PTS-5 even after excessive cracking. At the pressure level around the yielding point of the control specimen, the repaired slab's deflection was much smaller than the nonrepaired slab's deflection (see Fig. 24), indicating that use of CFRP sheets effectively increases the flexural resistance of unbonded PT slabs.

In terms of ductility capacity, there were no consistent trends between the nonrepaired and repaired specimens. If significantly smaller reinforcing bar amounts are present, the failure mode would have been more ductile. More studies need to be developed to achieve ductile failure mode, and/or a strength reduction factor should be applied to the brittle mode of failure.

#### Stresses in PT and nonprestressed mild steel

Unlike bonded prestressed or conventionally reinforced concrete members, the prestressing strands in the unbonded PT members never reach their ultimate strength  $f_{pu}$ . This is because the ultimate strength  $f_{vu}$  of unbonded tendons is not dependent on the localized strain at the flexural critical section but depends on the total member elongation, number of spans, span-depth ratio, and loading type (ACI 318-08; Kang and Wallace 2008). Just before loading of the slabs, the PT forces in the strands were recorded. The average effective stress (in this case before external loading) was normally kept between 65 and 70% of the ultimate strength of the strands. As the loading increased, the tendon stress increased nonlinearly. The rate of tendon stress increase was very small (approximately 0.005% of  $f_{m}$ ) before concrete cracking, but it became increasingly larger as the slab deflection increased until the ultimate load. Figure 26 shows a representative result for PT force variation against an external pressure.

On the other hand, as the slabs were loaded and started to crack heavily, the stress in the mild steel started to increase and became close to yield stresses (Fig. 27); however, no significant yielding was observed from the strain gauge data (Fig. 27). For example, strain in the wire mesh of PTS-1

		Measured average PT forces, lb (kN)			$\Delta F$
	Applied pressure,	0		$\Delta F = F_u - F_e,$	
Specimen	psi (MPa)	F <sub>e</sub>	$F_{u}$	lb (kN)	$F_e$ ,%
PTS-1	6.5 (0.045)	6900 (30.8)	7101 (31.7)	201 (0.897)	2.9
PTS-1CR	9.2 (0.063)	6920 (30.89)	7001 (31.25)	81 (0.362)	1.2
PTS-2	5.2 (0.036)	6931 (30.94)	7128 (31.82)	197 (1.027)	2.8
PTS-2CR	9.8 (0.068)	6910 (30.85)	7013 (31.3)	103 (0.879)	1.5
PTS-3	6.5 (0.045)	6892 (30.77)	7122 (31.8)	230 (1.46)	3.3
PTS-3CR	10.9 (0.075)	6920 (30.89)	7068 (31.55)	148 (0.661)	2.1
PTS-4	5.4 (0.037)	6588 (29.41)	7033 (31.4)	445 (1.987)	6.8
PTS-4CR	7.9 (0.054)	6546 (29.22)	6601 (29.47)	55 (0.246)	0.8

#### Table 6—Effective tendon forces and tendon forces at ultimate loads for selected specimens

Notes:  $F_e$  is effective tendon force;  $F_u$  is tendon force at ultimate load.

started to increase as the external loading was increased and reached a maximum strain of approximately 0.002 at an external pressure on the slab of 6.5 psi (0.045 MPa). The stresses of the wire mesh in the two-way slabs nearly reached the yield stress near the ultimate load, but it cannot be said that the nonlinear pressure-deflection behavior is attributed to the yielding of the mild steel. Rather, the nonlinear behavior was related to the significant concrete cracking. Note that the nonlinear behavior of the tendons was modest, as it was kept within the elastic range.

The stresses of the reinforcing bars in the one-way slabs were greater than those of the wire mesh in the two-way slabs. For instance, the average stress at the ultimate load was 52.5 ksi (362 MPa) and 55.6 ksi (383 MPa) for PTS-1 and PTS-2, respectively, whereas the average was 56 ksi (386 MPa) and 61 ksi (421 MPa) for PTS-3 and PTS-4, respectively. This may be due in part to the larger width of the damaged region that formed in the one-way slab than in the two-way slab; however, the degree of bonded steel yielding was limited for the specimens.

#### Behavior of slabs repaired with CFRP sheets

As noted, the cracked slabs were repaired with CFRP sheets and tested again. The ultimate loads of the test specimens strengthened with CFRP sheets were always higher than the ultimate loads of the non-repaired specimens (Table 5). The flexural strength of the slabs strengthened with CFRP composite materials increased by 42%, 88%, and 61% for PTS-1CR, PTS-2CR, and PTS6CR (two-way slabs), respectively, and by 68%, 46% and 79% for PTS-3CR, PTS-4CR, and PTS-5CR (one-way slabs), respectively. Interestingly, however, the stress increases in PT tendons in the repaired specimens at ultimate loading stage were always lower in comparison to those in the control specimens by approximately 10 to 65% (Table 6). This was the case even though the maximum deflections of the repaired specimens were larger than those of the control specimens. Note that the testing of the non-repaired control specimens was stopped when the tendon stress reached the criteria of  $0.75 f_{nu}$  to  $0.8 f_{nu}$  or other criteria were reached. This means that the total elongation of the tendon was larger when the concrete cracked heavily such that the plastic concrete deformation (that is, opening of cracks) at the level of the tendons was substantial. On the other hand, the cracks were stitched by the CFRP reinforcement externally bonded to the concrete top surface. After the repair, the slab concrete behaved like an elastic solid and, in this case, the tendon stress increase in the CFRP-repaired slab was not as much as that in the slab without CFRP at the same given deflection. As a result, the components of tension were produced primarily by the CFRP composite materials under bending. This experimental finding is of value and demonstrates another benefit of using CFRP for unbonded PT structures, as the CFRP strengthening not only provides the additional strength but also leads to the decrease in the tendon stress increase.

The increased strength with respect to the nonrepaired strength varied from 42 to 88% for two-way slabs and from 46 to 79% for one-way slabs. The larger increase in strength was attributed to the larger amount of CFRP used for strengthening (refer to Tables 2 and 5). The critical yield line pattern developed in the control specimens could not propagate further because of the presence of the CFRP across the crack lines. Again, at the time the testing was completed, the CFRP-repaired slabs remained in an essentially elastic condition. This is due to the perfectly linear strain-stress behavior of the carbon fibers, which had not

been ruptured throughout the testing. Also, rigid plate movement along the crack line did not happen. Concrete did not crush at the bottom surface along the yield lines. Overall, it was verified that effective placement of CFRP increased the load-carrying capacity of unbonded PT slabs. The quantity of the CFRP sheets used for the specimens was sufficient and adequate for increasing the flexural strength by about 42 to 88% without concrete crushing. Furthermore, it was concluded that the CFRP placement patterns of both the diagonal and orthogonal schemes were effective for two-way unbonded PT slabs.

#### SUMMARY AND CONCLUSIONS

1. Nonlinear behavior was observed from pressuredeflection relationships of unbonded PT one- and two-way slabs under uniformly distributed pressure or area loads. This was due to the considerable tensile cracks that occurred at the high moment region. However, as anticipated, the tendon stress increase was only approximately 0.8 to 6.8% of the effective stress.

2. While the deflection was much higher for CFRPrepaired slabs, the unbonded tendon stress increases were lower than those in the control specimens by approximately 10 to 65%. This indicates that a total elongation of the tendons is much higher when large crack opening occurs, rather than when a large deflection occurs.

3. The PT concrete slabs repaired with CFRP fabric and bonded to the tension surfaces gained considerable strength. Flexural capacity of the slabs strengthened with CFRP composite materials increased by approximately 40 to 90% for two-way slabs (PTS-1CR, PTS-2CR, and PTS6CR) and approximately 50 to 80% for one-way slabs (PTS-3CR, PTS-4CR, and PTS-5CR).

4. As such, the slabs repaired with properly designed CFRP schemes showed sufficiently larger load-carrying capacities than the nonrepaired slabs. Both orthogonal and diagonal two-layer placement schemes used in this study were effective, as the CFRP fibers were perpendicular to the crack lines.

5. The measured pressure-deflection relationships between the control PT slabs and repaired slabs also indicate better serviceability conditions (for example, stiffness and crack restraint) for the repaired slabs even after substantial damage. The behavior of the CFRP-repaired slabs was essentially linear or slightly nonlinear. No fiber tensile failure, debonding, or concrete crushing was observed.

6. The quantity of used CFRP sheets was sufficient and adequate for increasing the flexural strength by approximately 42 to 88% without concrete crushing.

7. The results from this study indicate that different end supports of one-way slabs caused large variations in performance. The fixed-fixed condition (PTS-3 and PTS-3CR) shows a 68% increase in ultimate strength due to CFRP repair, whereas the fixed-pin condition (PTS-4 and PTS-4CR) shows a 46% increase. As the degree of fixity at the ends decreased, the stiffness and load-carrying capacity (ultimate strength) increased by the CFRP sheets were reduced.

An alternate CFRP retrofitting system that can be employed is to use CFRP laminated strips or CFRP prestressed strips. These retrofitting methods for prestressed or PT concrete structures should also be considered as future studies. Although promising outcomes have been reported by this study, the CFRP systems applied to unbonded PT slabs may not be considered as a generally applicable repair system until further verifications are undertaken on the ductility of CFRP-repaired PT slabs with overstressed or ruptured steel reinforcement.

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