

# Durability of Concrete Beams Prestressed with CFRP Bars

by H.C. Mertol, S. Rizkalla, P. Scott, J.M. Lees,  
and R. El-Hacha

**Synopsis:** The need for sustainable structures is the driving force for the growing international interest in using advanced materials such as fiber reinforced polymer (FRP) materials for civil infrastructure applications. To date, FRP materials have not realized their full potential within the construction industry. One of the contributing factors is the limited information regarding their long-term performance. In particular, since the service life of a civil engineering structure is typically 50 to 100 years, knowledge of the long-term durability of FRPs is of prime importance.

The research work presented in this paper is the result of a research collaboration between North Carolina State University and the University of Cambridge. The research investigates the durability of concrete beams prestressed with CFRP bars and compares the results with those of companion beams prestressed with steel wires. A total of 15 beams have been constructed and tested under different mechanical and environmental conditions. The parameters included in the program were the level of sustained stress in the bars and wires (55 and 70 percent of the ultimate bar or wire strength), the environmental exposure condition (air exposure and continuous exposure to 15 percent by mass salt water spray at 54 °C temperature), the length of time under sustained load (9 and 18 months) and the method of testing (with or without application of cyclical loading prior to static testing to failure).

The experimental program illustrates that CFRP prestressed concrete beams exhibit comparable ultimate strength and fatigue strength properties in comparison to equivalent steel prestressed concrete beams. Furthermore, test results show that the beams prestressed with steel wires did not survive the environmental exposure over 12 months whereas the beams prestressed with CFRP bars survived up to the end of the 18 month long extreme environmental exposure, indicating the excellent durability of CFRP in the marine environment. Provided that provision is made for the lack of CFRP ductility in comparison to steel, the research undertaken has indicated that CFRP prestressed concrete is a durable, appropriate option when designing structures for use in offshore environments.

**Keywords:** beam; carbon-fiber-reinforced polymer bars; cyclic loading; durability; environmental exposure; marine environment; prestressed concrete; steel wires

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ACI Member, **Halit Cenani Mertol**, obtained his B.Sc. (1999) and M.Sc. (2002) from Middle East Technical University in Turkey. He is currently a Ph. D. candidate in Civil, Construction and Environmental Engineering in North Carolina State University, Raleigh, North Carolina. His main interests are fiber reinforced polymer materials and high-strength concrete.

ACI Fellow, **Sami Rizkalla**, is a Distinguished Professor of Civil and Construction Engineering in the Department of Civil, Construction and Environmental Engineering, North Carolina State University (NCSU). He is the Director of the Constructed Facilities Laboratory and NSF Industry/University Cooperative Research Center in Repair of Structures and Bridges at NCSU. He is the current Chair of PCI committee on FRP and is the immediate Past President and the founder of the Network of Centers of Excellence on Intelligent Sensing for Innovative Structures (ISIS Canada), and Past Chairman of ACI Committee 440 "Fiber Reinforced Polymer Reinforcement". He is a fellow of ASCE, CSCE, EIC and IIFC.

**Paul Scott**, is a member of the Institute of Structural Engineers and the Institute of Mechanical Engineers, having obtained a Masters degree in Mechanical Engineering from the University of Nottingham in 2004. He is currently a Ph. D. candidate in the Structures group at the University of Cambridge, UK, investigating aspects of CFRP prestressed concrete durability in the marine environment.

ACI Member, **Janet M. Lees**, is a University Senior Lecturer in the Department of Engineering at the University of Cambridge. Her research interests include FRP prestressed concrete, the strengthening and repair of existing structures, GFRP pipelines and recycled materials in concrete.

ACI Member, **Raafat El-Hacha**, is an Assistant Professor in the Department of Civil Engineering at the University of Calgary, Canada. He is member of ACI Committee 440 "Fiber Reinforced Polymer (FRP) Reinforcement" and co-chair of Sub-Committee 440-I "FRP Prestressed Concrete". He is an associate member of joint ACI-ASCE 423 "Prestressed Concrete", ACI 408 "Bond and Development of Reinforcement", and ACI 437 "Strength Evaluation of Concrete Structures". His research interests include strengthening concrete and steel structures and reinforcing/prestressing concrete structures with fiber reinforced polymer (FRP) material, blast mitigation and repair using FRP. He is member of the CSCE, ASCE, PCI, PTI, and IIFC.

### INTRODUCTION

The need for sustainable structures is the driving force for the growing international interest in the use of fiber reinforced polymer (FRP) materials for civil infrastructure applications. To date, FRPs have not realized their full potential within the construction industry. One of the contributing factors is limited information regarding their long-term performance. In particular, since the service life of a civil engineering structure is typically 50 to 100 years, knowledge of the long-term durability of FRPs is of prime importance.

## Use of FRP for Prestressing Applications: Case Histories 3

The durability of FRP materials depends on numerous factors including the fiber properties, resin properties and manufacturing process used. In general, the durability of CFRP has been found to be very good since carbon is considered one of the most durable types of fibers when compared to glass or aramid alternatives. However, the majority of the research reported on FRP durability considers either a particular mechanical loading regime or a particular environmental exposure condition in isolation. Whilst valuable results can be obtained using such approaches (Balazs and Borosnyoi 2001, Micelli and Nanni 2004); in service, structures will be subjected to both mechanical and environmental loadings concurrently. It is therefore necessary to determine the consequences of any mechanical/environmental deterioration on the overall performance of a structure prestressed and/or reinforced with these materials. These effects must be investigated in order to build confidence within the industry and to identify both the limits and potential of FRPs in structural applications.

This research work, performed at NCSU and conducted in collaboration with the UC, investigated the durability of concrete beams prestressed with CFRP bars. Although the results of the research will apply to many types of structure, the initial objective was to build on the preliminary work carried out at the UC on systems for CFRP prestressed concrete wind turbine masts (Agyei 2002). These masts combine high-strength concrete with high-strength CFRP bars to form a strong and durable structure. A prominent issue in the design of wind turbine masts is the fatigue loading in a corrosive environment. Initially, studies were carried out at the UC on the fatigue performance of high-strength concrete beams prestressed with CFRP bars (Agyei 2002) and the results were promising. However, the research did not incorporate the implications of severe environmental conditions such as salt water spray to which offshore structures would be subjected.

The current project extended the work using the environmental chamber at NCSU. Within the chamber, durability tests were performed simultaneously as the specimens were subjected to sustained loading. Understanding the behavior of the CFRP material under these conditions is required in order to gain information on its corrosion fatigue performance as a prestressing reinforcement.

### RESEARCH SIGNIFICANCE

The research findings provide a unique evaluation of the durability and estimation of the service life of concrete structures prestressed with CFRP bars when compared with equivalent structures prestressed with steel wires. The experimental program investigated concrete beams prestressed with CFRP bars or steel wires which were subjected to sustained loading and environmental exposure prior to testing under realistic static and fatigue loading conditions. The information summarized in this paper provides a valuable contribution to the work of ACI Committee 440 which is currently in progress on the development of a comprehensive document on the durability of FRP for concrete structures.

**EXPERIMENTAL PROGRAM**

The test series considered in the experimental program consisted of fifteen concrete beams prestressed with either CFRP bars or steel wires. The beams were precracked prior to being subjected to various combinations of sustained loading and exposure conditions. The pre-cracking generated a path for moisture to propagate directly to the prestressing bars and wires, thus representing the most severe exposure conditions in the field. The precracked beams were subjected to two different sustained load levels that generated a stress level in the bars and wires of 55 or 70 percent of their ultimate tensile strength,  $f_{pu}$ . Eight of the beams subjected to sustained load were placed into the environmental chamber to simulate harsh environmental effects whereas the rest of the specimens subjected to similar sustained loads were exposed to air for comparison purposes. The beams were subjected to a combined sustained load and environmental exposure for either 9 or 18 months. After the environmental exposure and sustained load program was completed, the beams were statically loaded to failure, in certain tests after having been subjected to cyclical loading. A summary of the testing scheme is given in Table 1.

**Test Specimens**

The beam design was based on the fatigue study completed by Agyei (2002) at the UC. The beams tested at NCSU had 108x203 mm rectangular cross-section with an overall length of 2130 mm as shown in Figure 1. Each beam contained two prestressing bars or wires with a spacing of 64 mm and a clear cover of approximately 20 mm from the side of the beam. The distance from the bottom surface of the beam to the center of the bar or wire was 32 mm.

**Material Properties**

In all the test specimens, a special type of cement, SPC cement, produced by Holcim, Inc. and a single type of aggregate, Lillington sand, were used to obtain a high-strength concrete. The mixture included dry densified silica fume to increase the strength and a HRWRA to improve the workability of the mixture and enable the use of a low water/cementitious material ratio. Polypropylene fibers were also used to improve the resistance of the concrete to shrinkage and cracking.

Details of the concrete mixture design are given in Table 2. All the materials were mixed in a 0.06 m<sup>3</sup> capacity drum mixer. The moisture content of the sand was measured and the total amount of water in the mixture was adjusted accordingly. The sand, cement, silica fume and fibers were combined in the drum and mixed in a dry condition. The water was added followed by the HRWRA. All the ingredients were mixed for a few minutes in the mixer prior to casting. Further mixing was done by hand using trowels and shovels to ensure that there was no unmixed material at the corners of the pan. Due to the limited capacity of the drum mixer, multiple batches were mixed continuously to cast multiple test specimens. Ten 102x203 mm companion cylinders were cast for each test specimen to determine the compressive concrete strengths on the day of bar or wire detensioning, at the time the beam was precracked, at the commencement of the sustained

loading, when the beam was tested to failure and the tensile strength at the time of precracking. The concrete cylinders were exposed to the same environmental conditions as their companion beams.

The CFRP bars, supplied by SACAC Ltd, consisted of Tenax UTS fibers in an epoxy matrix. The CFRP bar properties were independently confirmed by an ultimate bar strength measured from anchor tests and found to be consistent with the data supplied by the company. The steel wires were supplied by VSL. A single wire from a 1200 kN seven-wire 12.7 mm low relaxation prestressing steel strand was used. The diameter was comparable to that of the CFRP bar. The breaking stress of the single wire was assumed to have the typical 1940 MPa tensile strength. Though the standard deviations of the bar and wire tensile properties were not measured, the nominal values supplied by the manufacturers are given in Table 3. The initially applied prestressing level was  $0.55 f_{pu}$  for both the CFRP bars and steel wires. The test specimens prestressed with CFRP bars were designed to fail by bar rupture and the stresses in the concrete would be fairly low. The test specimens prestressed with steel wires were designed to fail by the yielding of the wire followed by concrete crushing.

### **Specimen Preparation**

A prestressing rig was constructed using steel HSS steel members at the ends and built-up sections at the sides. A view of the prestressing rig is shown in Figure 2. The built-up sections consisted of 2 channel sections. Each end of the stressing rig consisted of two HSS steel sections separated using a 25.4 mm plate to form a slot through which the bars and wires could pass. The prestressing rig was bolted to the strong floor using threaded rods. The formwork was constructed so that four test specimens could be cast in the prestressing rig simultaneously. Two specimens were located side by side, with the other two specimens connected to the ends of the first two specimens. The formwork was constructed using 25.4 mm thick plywood to enable repeated usage. The ends of the formwork had holes through which the bars and wires could pass. The plywood at the ends was cut along the center of the holes to form two parts. The lower part of the end plywood was secured to the base plate prior to locating the bars and wires. The upper part of the end plywood was fixed after the bars and wires were tensioned. Two cross beams were used to fix the formwork to the strong floor to avoid movement of the formwork during casting and vibration.

Expansive cement anchors were used to prestress the CFRP bars to avoid possible premature bar failure due to stress concentrations within the anchorage zone. An anchorage system using steel tubes and expansive cement was employed after successive pilot tests. The CFRP bars were inserted into 254 mm long steel tubes with an internal diameter of 19 mm and a thickness of 7.7 mm. In each case, the bar was centered in the tube which was subsequently filled with expansive cement mixed with 28.5 percent water by weight. An end cap was used to seal one end of the tube and a hole was drilled in the center of the cap to center the bar. Each anchor was cast vertically at the dead end of each bar. At the other end of the bar, the live end, a 355.6 mm long steel tube with an outside diameter of 21.34 mm was used. The inner tube surface was threaded over a length of

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82.55 mm to fit a 15.88 mm threaded rod which was used for prestressing the bar. The unthreaded regions of the tube had an inside diameter of 13.21 mm. A central hole was drilled into the end of the threaded rod which was screwed into the threaded section of the tube so as to center the bar. After casting one anchor and waiting for one day, the CFRP bar was inverted and the tube at the other end was filled with expansive cement. The loads cells were located between the anchors prior to casting the second anchor. The anchors were cured for three days. The tensile tests on the anchorage system showed that the CFRP bar failed in the anchor region at a load of 26 kN which was much greater than the design prestressing force of 15 kN. The live and dead ends of the CFRP bars are shown in Figure 3.

For the stressing of the steel wires, 4 mm multiple use chucks and 4 mm reusable anchor chucks were used for the dead and live ends respectively. Initial tensile tests performed on an anchored short length of steel wire confirmed a breaking load of 25 kN. These results indicate that both the CFRP bars and the steel wires had roughly the same breaking loads. The live and dead ends of the steel wires are shown in Figure 4.

Prior to stressing, the bars and wires were located in the desired positions and a small prestress was applied to position the bars and wires in their location. Each bar and wire was stressed using a hollow hydraulic jack with a central hole supported on a jacking chair as presented in Figures 3 and 4. When the desired prestressing force, 14.7 and 15.1 kN for the CFRP bar and steel wire respectively, was reached, the bar or wire force was transferred from the jack to the end plate. Each bar or wire was stressed sequentially. After all four bars or wires had been stressed, minor adjustments were made by tightening or loosening the bearing nuts to ensure the prestressing loads in all the bars and wires were as desired.

Casting of the test beams took place within two hours of prestressing. Four batches were mixed in the drum mixer to cast four specimens, simultaneously. Due to concerns about inadvertently damaging the bars and wires, 25.4 mm vibrators were applied directly to the formwork rather than the concrete.

After casting, the beam specimens and companion cylinders were covered with a polythene sheet to reduce evaporation. Furthermore, water was sprayed on the specimens twice a day to ensure proper hydration. The beams were typically detensioned three days after casting. By this time the average compressive strength of the concrete was approximately 46 MPa. The bars and wires were detensioned initially at the live end prior to cutting the bars and wires at the dead end using a hacksaw. The formwork was demolded and the beam specimens were stored outside the laboratory.

### **Test Set-Up and Method**

All the beams were precracked under a two point simply supported loading condition as shown in Figure 5. The precracking was carried out between 28 and 30 days after casting in order to maintain a consistent concrete tensile strength in all the test beams. The test procedure consisted of two loading cycles to determine the uncracked and cracked

stiffness of each beam specimen. For the first loading cycle, each beam specimen was loaded up to a level corresponding to a bar or wire stress level of approximately 70 to 75 percent of  $f_{pu}$  and then unloaded. For the second loading cycle, the specimen was reloaded to the same load level and unloaded again.

The sustained loading was applied to the precracked beams using a two point simply supported loading configuration. Threaded rods were used to load the beams against a supporting slab as shown in Figure 6. Springs were used to compensate for any load loss due to creep. The specimens were loaded so that the stress levels in the bars or wires were 55 percent or 70 percent of  $f_{pu}$ . Seven specimens under sustained load were exposed to air as control specimens as shown in Figure 7. Eight beam specimens under sustained load were located in the environmental chamber as shown in Figure 8. The tank of the environmental chamber was filled with 15 percent by mass salt water solution at 54 °C. A heater-thermostat combination was used to keep the water temperature constant. A pump-pipe system was used to spray water on the specimens continuously. The duration of the combined sustained load and environmental exposure program was either 9 or 18 months.

After completion of the combined sustained loading and environmental exposure program, the beams were tested to failure using a two point simply supported loading condition as shown in Figure 9. This figure also shows a typical failure mode of the beams subjected to static loading. Seven of the beams were tested under static loading whilst eight of the beams were subjected to cyclic loading prior to static loading to failure. The static load was applied using a loading rate of 668 N per minute. The duration of the static tests was approximately 40 minutes. The procedure for the cyclic loading included the application of a load level corresponding to bar or wire stress ranges varying from 65 to 75 percent of  $f_{pu}$  at a frequency of 3 Hz for two million cycles. The specimens that survived the cyclic loading were tested to failure under static loading. Prior to the application of the cyclic loading, the beams were statically loaded up to their precracking load to determine their stiffnesses.

### **Instrumentation**

Before casting concrete, the two prestressing bars or wires of each beam were instrumented using four ERSGs. Two gages were mounted at the mid-point of each bar or wire; two further gages were attached to one of the bars or wires at a distance of 305 and 660 mm from the midpoint to obtain the strain profile throughout the length of the bar or wire. The bars and wires were stressed using a manually controlled 20 ton hollow hydraulic jack. The load on each prestressing bar or wire was measured using a 230 kN load cell. The ERSGs and load cells were connected to a strain indicator and the data was recorded manually throughout the prestressing, casting and detensioning processes.

The same hydraulic jack used in the prestressing operations was also used to apply the load for the precracking tests. The load was measured using a 114 kN flat load cell. The deflections at the midspan, loading points and supports were monitored using five 38.1 mm LVDTs. The ERSGs, LVDTs and load cell were monitored using a data acquisition system. Demec points with a gage length of 102 mm were bonded on the side face of the

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midspan of the beam to measure the strain at the bar or wire level (32 mm above the bottom surface) and in the compression zone (190 mm from the bottom surface). The data from the demec points were monitored at various load increments during precracking.

In the sustained loading stage, each of the threaded rods was instrumented using two ERSGs which were used as load cells on the threaded rod. Prior to the application of the sustained load using a 20 ton hollow hydraulic jack, the ERSGs on each of the threaded rods were calibrated using a 114 kN Strainsert flat load cell. The wires of the ERSGs were insulated using plastic tubes to prevent damage to the gage. Only the ERSGs applied on the bars and wires of the beams and applied on the threaded rods were monitored during the sustained loading stage using a datalogger.

The static and fatigue testing was performed using a 100 kN MTS hydraulic actuator with an internal load cell. The deflections at the midspan and loading points were monitored using three 152 mm Honeywell LPs and the deflections at the supports were monitored using two 76 mm Honeywell LPs. Three 100 mm pi gages were used to monitor the mid-span strains at the top surface, mid-height and 25.4 mm above the bottom surface. The strain gages, linear potentiometers, pi gages, load and stroke of the actuator were monitored using a data acquisition system. In order to determine the degradation due to cyclic loading, dynamic behavior was monitored at intervals of 1k, 5k, 10k, 50k, 100k, 250k, 500k, 1,000k, 1,250k, 1,500k, 1,750k and 2,000k cycles. A static reading was also taken just before the dynamic data was captured at each stage.

### TEST RESULTS AND DISCUSSIONS

Table 4 lists the concrete strengths of each specimen which were evaluated using companion concrete cylinders tested at the time of the beam testing. The concrete strengths of Beams B14 and B15 were lower than average due to partial hydration of the cement prior to casting caused by a period of time in which it was in contact with moist sand. The variation in compressive strengths of the remaining cylinders at the time that the beams were precracked can generally be attributed to the differing times after casting at which each beam was precracked, shown also in Table 4.

The compressive strength of the concrete was affected by the exposure environment. When the beams were tested to failure, companion cylinders for beams B1 to B6 subjected to the heated salt water spray had a mean compressive strength of 92.3 MPa, compared to 72.9 MPa measured for companion cylinders for the air exposed beams B7 to B10 and B13. The higher average strength of the companion cylinders for beams B1 to B6 can be attributed to the further cement hydration and concrete curing caused by the continual immersion of the beams and cylinders in the heated salt water spray.

As the beams were all designed to fail due to bar rupture or wire yielding, rather than concrete crushing, the variation in ultimate concrete strength between beams should not be particularly significant. However, there will be corresponding variations in the

modulus of the concrete, which will have corresponding effects on the stiffness of the beams.

The experimental program details are given in Table 5. The beams were precracked before they were subjected to the loading and exposure program. The beam stiffnesses prior to cracking were measured to be 14.0 - 17.5 kN per mm central deflection and were not significantly affected by the concrete compressive strengths, with the exception of beams B14 and B15, whose considerably lower concrete strengths resulted in a slight reduction in beam stiffness. Typically, either two or three cracks resulted within the constant moment region of the beams. The crack widths ranged between 0.2 and 0.3 mm. When the beams were unloaded, all the cracks closed although some residual displacements were recorded. A typical example of experimental precracked and postcracked behavior is shown in Figure 10.

The measured precracking loads for the beams ranged from 16.4 kN to 18.4 kN. The cracking loads were calculated based on the measured tensile strengths of the companion concrete cylinders, and an allowance was made for the creep and shrinkage of concrete as well as bar or wire relaxation. The calculated cracking loads were lower than the measured values which suggested that either the concrete tensile strength was greater than the values measured using concrete cylinders or the prestressing losses were less than anticipated.

The sustained loadings were applied for periods of 9 or 18 months. Due to the severe environmental conditions used in the chamber, all of the ERSGs attached to the prestressing bars and wires, and threaded rods did not function after 7 days. Although the wires of the ERSGs were insulated using plastic tubes, the damage of the gages due to environmental conditions could not be prevented. Furthermore, the gages attached to the bars and wires of the beams and threaded rods exposed to air showed unreliable results after 3 months. Therefore, the strain gage readings were not considered sufficiently reliable for comparison purposes. Beams subjected to environmental exposure for 9 months are shown in Figure 11.

All beams subjected to cyclic loading survived two million cycles except beam B7 due to some experimental problems related to the testing equipment which was corrected for the rest of the testing program.

The load versus midspan deflection behavior of all the beams tested to failure is shown in Figures 12 to 14. All the beams prestressed with CFRP bars showed a similar type of behavior except beam B1 which failed under a relatively low load. Reasons for this discrepancy are being investigated. The beams prestressed with steel wires behaved similarly to each other.

In the following, comparisons are made between the test results, and comments made as to the significance of the variables considered in the test program. However, differences in concrete strength between the beams at the time of testing, due to differing types of environmental exposure, may also play a role.

**Effect of Prestressing Material**

After precracking, the deflections of the beams prestressed with steel wires were always greater than those of the beams prestressed with CFRP bars under similar applied loads. The beams prestressed with steel wires were affected more significantly than the beams prestressed with CFRP bars under exposure to salt water spray. CFRP prestressed beams B5 and B6 were subjected to the same sustained load and salt water spray as steel prestressed beams B11 and B12. Whereas beams B5 and B6 survived the environmental exposure program for 18 months, beams B11 and B12 failed after 12 months due to the harsh environmental conditions. On inspection of the steel wires in beams B11 and B12 post failure, significant levels of corrosion were observed, which caused the wire rupture. This illustrates the benefit of using CFRP to prestress concrete in aggressive environments, in comparison with structures prestressed with steel.

In general, the measured ultimate load capacities of the beams prestressed with CFRP bars were greater than those of the beams prestressed with steel wires. The maximum deflections at failure of the beams prestressed with steel wires were much greater than those of the beams prestressed with CFRP bars; an expected result given the ductility of the steel compared with the brittle CFRP. Beams B8 and B14, prestressed with CFRP bars and steel wires respectively, sustained  $0.7 f_{pu}$  of the bar or wire rupture strength and were exposed to air. On testing, the load and deflection of beam B8 at failure were 31 percent higher and 80 percent lower respectively compared to beam B14. Beams B9 (CFRP prestressed) and B15 (steel prestressed), were subjected to the same conditions as beams B8 and B14, but were tested cyclically. The load and deflection of beam B9 at failure were 16 percent higher and 92 percent lower respectively compared to beam B15.

**Effect of Cyclic Loading**

In all of the beams, a degradation in stiffness was observed as the number of cycles increased. Cyclically loading CFRP prestressed beams slightly reduced their maximum load capacity. When comparing beams with identical loading, exposure and test periods, cyclically loaded beams B4, B6 and B9 failed at loads 3.0 percent, 6.5 percent and 5.5 percent lower than the equivalent non-cyclically loaded beams B3, B5 and B8 respectively.

The effect of cyclic loading on the steel prestressed beams was mixed. When other test parameters were identical, cyclically loaded beams B13 and B15 failed at loads 11.5 percent lower and 6.7 percent higher than non-cyclically loaded beams B10 and B14 respectively. These results indicate that beams prestressed with CFRP bars and steel wires are similarly capable of carrying cyclic loading within the vulnerable range of stresses.

**Effect of the Level of Sustained Loading and Duration of Loading**

The test results indicate that the stress range for sustained loading did not have a significant effect on the beams prestressed with CFRP bars. With other test parameters

identical, beam B4 in which the bars were subjected to a sustained stress of  $0.7 f_{pu}$ , failed at a load 2.3 percent lower than beam B2 in where the bars were subjected to a sustained stress of  $0.55 f_{pu}$ . Given that the beams prestressed with steel wires, beams B11 and B12 failed in the environmental chamber, it can be surmised that in aggressive environments, the stress range has severe effects on beams prestressed with steel wires. As the stress in steel wires increases, the crack width also increases resulting in an increased vulnerability of the steel wires to environmental effects.

The beams prestressed with steel wires and subjected to air exposure, B10 and B14, were subjected to sustained loads for 9 months and 18 months respectively. The wires in beam B10 were subjected to a stress of  $0.55 f_{pu}$  compared to  $0.7 f_{pu}$  in beam B14. The combined effects of an increase in stress level and an increase in load duration were responsible for a 14 percent reduction in beam strength.

Increasing the duration of the sustained loading on CFRP prestressed beams in an aggressive environment caused either no change or only a slight reduction in beam strength. This was evident in the behavior of two identical beams, B5 and B6 that were exposed to salt water spray for 18 months and which failed at almost identical loads to beams B3 and B4 that were in this environment for only 9 months.

### **Effect of Environmental Exposure**

The environmental exposure to 15 percent by mass salt water spray and 54 °C temperature did not reduce the strength of the beams prestressed with CFRP bars. This was evident in the behavior of beams B5 and B6 which were exposed to the salt water spray and which failed at load levels even slightly higher than beams B8 and B9 which were exposed to the air. However, the environmental exposure affected the beams prestressed with steel wires significantly resulting in the premature failure of beams B11 and B12 which failed after 12 months of environmental exposure whereas the identically loaded air exposed beams B14 and B15 survived the sustained loading for 18 months.

## **SUMMARY AND CONCLUSIONS**

Nine beams prestressed with CFRP bars were tested under various mechanical and environmental conditions. The parameters included in the program were the level of sustained stress in the bars (55 and 70 percent of  $f_{pu}$ ), the environmental exposure condition (air exposure and continuous exposure to 15 percent by mass salt water spray at 54 °C temperature within an environmental chamber), the length of time under sustained load (9 and 18 months) and the testing method (with or without application of cyclical loading prior to static testing to failure). Six similar beams prestressed with steel wires were also tested as control specimens for comparison purposes.

The experimental program illustrated that CFRP prestressed beams exhibit comparable ultimate strength and fatigue strength properties in comparison to equivalent steel prestressed concrete beams. Furthermore, test results showed that the beams prestressed with steel wires did not survive the environmental exposure over 12 months whereas the

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beams prestressed with CFRP bars survived up to the end of the 18 month long extreme environmental exposure, indicating the excellent durability of CFRP in the marine environment.

As prestressing members, the brittle failure characteristic of CFRP bars is distinct to the ductile yielding of steel wires. This discrepancy is evident in the respective load deflection curves and differences in ultimate deflection of the beams at failure. The reduced ductility observed in the CFRP prestressed beams does not however characterise CFRP as an unsuitable choice for use in construction. The beam specimens were designed to fail by bar rupture or tendon yielding rather than concrete crushing, in order that the environmental and loading characteristics of the prestressing materials rather than the concrete could be evaluated. In practice, provided that design provision is made for the lack of CFRP ductility in comparison to steel, the research undertaken has indicated that CFRP prestressed concrete is a durable option when designing structures for use in offshore environments.

Specifically, test results indicate that:

1. Generally, the CFRP prestressed beams retained greater strength compared to the steel prestressed beams across the range of tests undertaken.
2. The beams prestressed with steel wires did not survive the environmental exposure over 12 months whereas the beams prestressed with CFRP bars survived 18 months up to the end of the environmental exposure.
3. When precracked, the beams prestressed with steel bars could sustain induced stresses of  $0.7 f_{pu}$ , when exposed to air, but not when exposed to a heated salt water spray over a period of 12 months.
4. Cyclic loading had insignificant effects on beams prestressed with CFRP bars and steel wires.
5. A higher sustained bar stress in the beams prestressed with CFRP bars resulted in a slight reduction in beam strength. Beams prestressed with steel wires subjected to higher sustained stress levels become more vulnerable to environmental effects.
6. Exposing the beams prestressed with CFRP bars to 15 percent by mass salt water spray and 54 °C temperature did not affect their behavior or cause any deterioration.
7. Increasing the salt water environmental exposure period for beams prestressed with CFRP bars affected neither the overall durability nor the strength.
8. Concurrently increasing the duration and level of sustained loading in the beams with steel prestressing wires exposed to air was found to cause a reduction in strength.

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LIST OF NOTATIONS

- CFRP: Carbon fiber reinforced polymer
- ERSG: Electrical resistance strain gage
- $f'_c$ : Compressive strength of cylinder
- $f_{pu}$ : Ultimate tensile strength of the CFRP bar or steel wire
- FRP: Fiber reinforced polymer
- HRWRA: High range water reducing admixture
- HSS: Hollow square section
- LP: Linear potentiometer
- LVDT: Linear variable displacement transducer
- NCSU: North Carolina State University
- UC: University of Cambridge

Table 1 – Summary of the experimental program

| Beam No. | Reinforcing Material | Sustained Stress Level in Bars and Wires | Environmental Exposure                           | Duration of Combined Sustained Load and Exposure Program (months) | Cyclic Loading |
|----------|----------------------|------------------------------------------|--------------------------------------------------|-------------------------------------------------------------------|----------------|
| B1       | CFRP Bar             | $0.55 f_{pu}$                            | 15 Percent Salt Water Spray<br>54 °C Temperature | 9                                                                 | No             |
| B2       |                      |                                          |                                                  |                                                                   | Yes            |
| B3       |                      |                                          |                                                  |                                                                   | No             |
| B4       |                      |                                          |                                                  |                                                                   | Yes            |
| B5       |                      |                                          |                                                  |                                                                   | No             |
| B6       | Steel Wire           | $0.70 f_{pu}$                            |                                                  | 18                                                                | Yes            |
| B11      |                      |                                          |                                                  |                                                                   | No             |
| B12      |                      |                                          |                                                  |                                                                   | Yes            |
| B7       | CFRP Bar             | $0.55 f_{pu}$                            |                                                  | 9                                                                 | Yes            |
| B8       |                      |                                          |                                                  |                                                                   | No             |
| B9       |                      |                                          |                                                  |                                                                   | Yes            |
| B10      | Steel Wire           | $0.55 f_{pu}$                            | Air                                              | 9                                                                 | No             |
| B13      |                      |                                          |                                                  |                                                                   | Yes            |
| B14      |                      |                                          |                                                  |                                                                   | No             |
| B15      |                      | $0.70 f_{pu}$                            |                                                  | 18                                                                | Yes            |
|          |                      |                                          |                                                  |                                                                   |                |

Table 2 – Concrete mixture design

| Material                                                                                       | Amount |
|------------------------------------------------------------------------------------------------|--------|
| Cement (kg/m <sup>3</sup> )                                                                    | 571.3  |
| Silica Fume (kg/m <sup>3</sup> )                                                               | 49.8   |
| Sand (kg/m <sup>3</sup> )                                                                      | 1568.6 |
| HRWRA (kg/m <sup>3</sup> )                                                                     | 10.29  |
| Polypropylene Fibers (kg/m <sup>3</sup> )                                                      | 1.03   |
| Water (kg/m <sup>3</sup> )                                                                     | 205.9  |
| water/cementitious material                                                                    | 0.33   |
| Average Compressive Strength at Time of Beam<br>precracking (at Approximately One Month) (MPa) | 61.5   |

Table 3 – Material properties of the prestressing bars and wires

| Property              | CFRP Bar              | Steel Wire            |
|-----------------------|-----------------------|-----------------------|
| Nominal Diameter      | 4.0 mm                | 4.3 mm                |
| Area                  | 12.57 mm <sup>2</sup> | 14.52 mm <sup>2</sup> |
| Tensile Strength      | 2200 MPa              | 1940 MPa              |
| Modulus of Elasticity | 161000 MPa            | 200000 MPa            |
| Elongation at Rupture | 1.37%                 | 5.21%                 |

Table 4 – Test results of companion concrete cylinders

| Beam No. | $f'_c$ at Bar or Wire Detensioning (MPa) | $f'_c$ at Precracking (MPa) | $f'_c$ at Application of Sustained Load (MPa) | $f'_c$ at Loading to Failure (MPa) | Tensile Strength at Precracking (MPa) |
|----------|------------------------------------------|-----------------------------|-----------------------------------------------|------------------------------------|---------------------------------------|
| B1       | 45.2                                     | 81.1 (38 days)              | 88.0                                          | 95.2                               | 4.3                                   |
| B2       | 46.9                                     | 67.0 (28 days)              | 76.9                                          | 84.6                               | 4.6                                   |
| B3       | 47.8                                     | 66.9 (28 days)              | 76.9                                          | 93.1                               | 4.9                                   |
| B4       | 52.3                                     | 82.5 (38 days)              | 84.9                                          | 100.1                              | 5.1                                   |
| B5       | 49.0                                     | 76.2 (59 days)              | 80.0                                          | 103.9                              | 5.0                                   |
| B6       | 49.6                                     | 65.9 (39 days)              | 63.7                                          | 82.3                               | 4.7                                   |
| B7       | 50.0                                     | 63.5 (39 days)              | 65.0                                          | 66.3                               | 4.4                                   |
| B8       | 49.4                                     | 78.0 (35 days)              | 84.0                                          | 83.2                               | 5.2                                   |
| B9       | 44.5                                     | 70.3 (35 days)              | 75.0                                          | 77.1                               | 5.2                                   |
| B10      | -                                        | 63.1 (30 days)              | 66.1                                          | 70.3                               | 4.2                                   |
| B11      | -                                        | 62.9 (30 days)              | 66.1                                          | -                                  | 4.3                                   |
| B12      | -                                        | 70.8 (30 days)              | 73.6                                          | -                                  | 4.3                                   |
| B13      | -                                        | 65.8 (30 days)              | 69.4                                          | 67.5                               | 3.9                                   |
| B14      | 36.8                                     | 47.0 (28 days)              | 51.6                                          | 55.2                               | 3.9                                   |
| B15      | 36.6                                     | 46.2 (28 days)              | 52.3                                          | 52.9                               | 4.2                                   |

- No specimens were tested

Table 5 – Experimental program details

| Beam No | Precrack. Loads (kN) | Sustained Loading |                                  | Cyclic Loading                      |                                  | At Failure     |                 |
|---------|----------------------|-------------------|----------------------------------|-------------------------------------|----------------------------------|----------------|-----------------|
|         |                      | Applied Load (kN) | Tendon Stress Level ( $f_{pu}$ ) | Load Range (kN)                     | Tendon Stress Range ( $f_{pu}$ ) | Max. Load (kN) | Max. Defl. (mm) |
| B1      | 16.5                 | 13.3              | 0.55                             | Static Testing Only                 |                                  | 23.6           | 10.7            |
| B2      | 16.4                 | 13.3              | 0.55                             | 16.0-18.7                           | 0.65-0.75                        | 30.8           | 12.0            |
| B3      | 18.6                 | 17.1              | 0.70                             | Static Testing Only                 |                                  | 31.0           | 10.9            |
| B4      | 16.4                 | 17.1              | 0.70                             | 16.0-18.7                           | 0.65-0.75                        | 30.1           | 12.2            |
| B5      | 16.4                 | 17.1              | 0.70                             | Static Testing Only                 |                                  | 31.0           | 11.2            |
| B6      | 17.7                 | 17.1              | 0.70                             | 16.0-18.7                           | 0.65-0.75                        | 29.0           | 10.0            |
| B7      | 18.4                 | 13.3              | 0.55                             | 16.0-18.7                           | 0.65-0.75                        | *              | 8.8             |
| B8      | 18.3                 | 17.1              | 0.70                             | Static Testing Only                 |                                  | 29.3           |                 |
| B9      | 18.0                 | 17.1              | 0.70                             | 16.0-18.7                           | 0.65-0.75                        | 27.7           | 4.0             |
| B10     | 17.4                 | 13.3              | 0.55                             | Static Testing Only                 |                                  | 26.1           | 42.8            |
| B11     | 17.4                 | 17.1              | 0.70                             | Failed under Environmental Exposure |                                  |                |                 |
| B12     | 16.9                 | 17.1              | 0.70                             |                                     |                                  |                |                 |
| B13     | 17.7                 | 13.3              | 0.55                             | 16.0-18.7                           | 0.65-0.75                        | 23.1           | 33.4            |
| B14     | 17.3                 | 17.1              | 0.70                             | Static Testing Only                 |                                  | 22.4           | 52.1            |
| B15     | 17.6                 | 17.1              | 0.70                             | 16.0-18.7                           | 0.65-0.75                        | 23.9           | 46.1            |

\* Failed due to short of electric circuit of the actuator after 300000 cycles

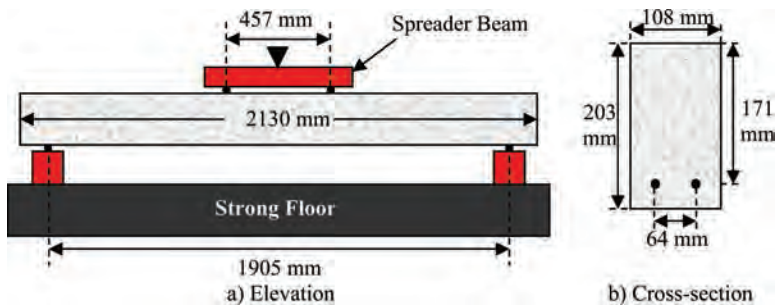


Figure 1—General view of the test specimens.



Figure 2—Prestressing rig.

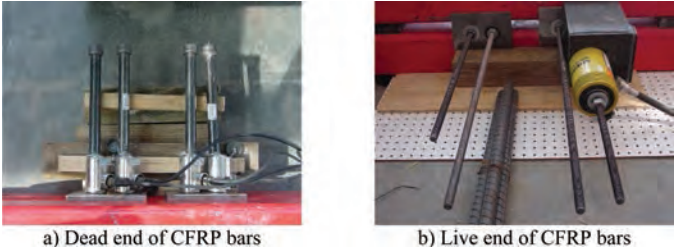


Figure 3—Preparation details for specimens prestressed with CFRP.

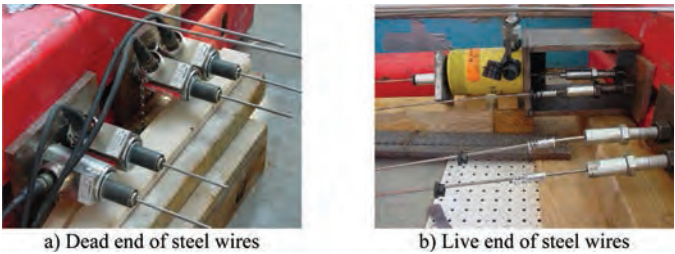


Figure 4—Preparation details for specimens prestressed with steel.



Figure 5—Test set-up for precracking.

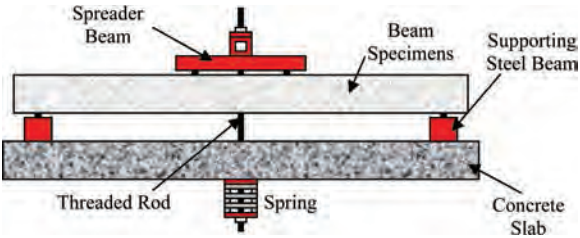


Figure 6—Test set-up for sustained loading.



Figure 7—Test set-up for sustained loading (air exposure).



Figure 8—Test set-up for sustained loading (environmental exposure).

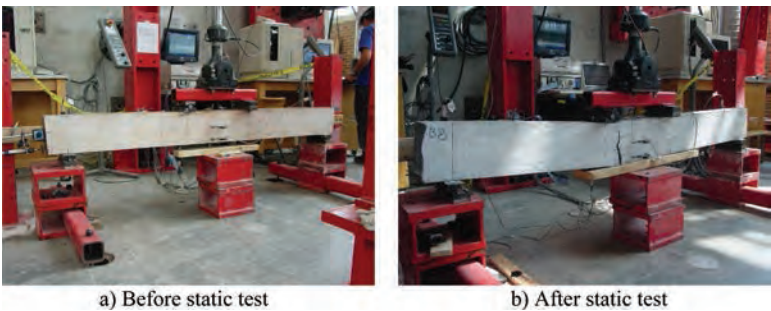


Figure 9—Test set-up for static and fatigue loading.

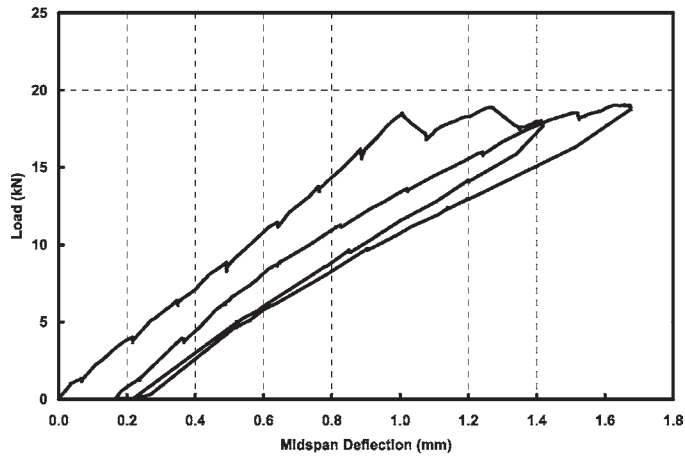


Figure 10—Typical behavior at the precracking stage (Beam B7).



Figure 11—Specimens exposed to environmental conditions for 9 months.

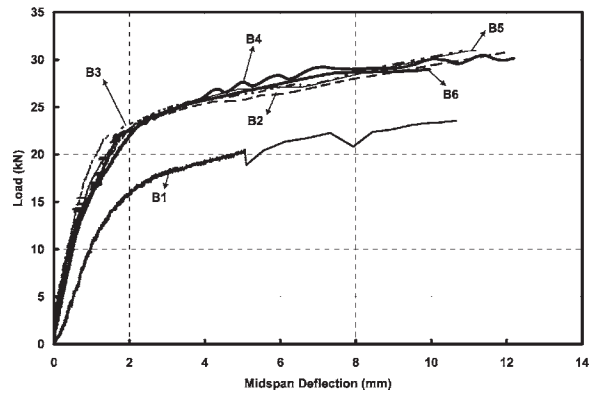


Figure 12—Load versus midspan deflection of beams prestressed with CFRP bars (exposed to environmental conditions).

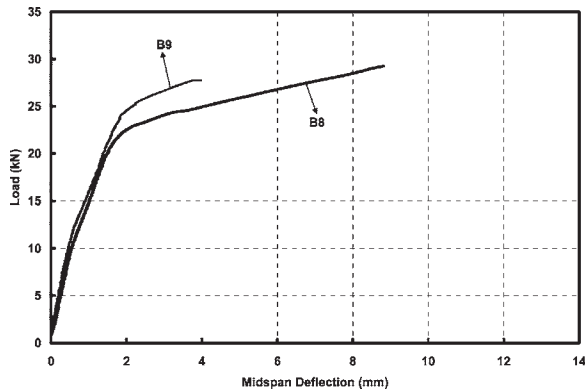


Figure 13—Load versus midspan deflection of beams prestressed with CFRP bars (exposed to air).

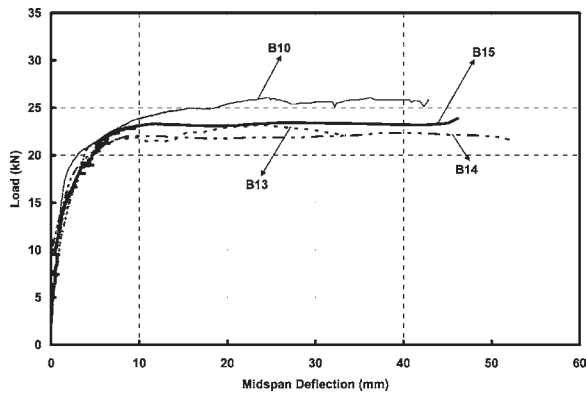


Figure 14—Load versus midspan deflection of beams prestressed with steel wires (exposed to air).



# Analysis of Long-Term Effects in Concrete Columns Prestressed with FRP Tendons

by C.C. Choo, I.E. Harik, and H. Gesund

**Synopsis:** An analytical investigation was conducted to study the long term effects in the strength interaction ( $P_n$ - $M_n$ ) behavior of non-slender concrete columns prestressed with fiber reinforced polymer (FRP) tendons. Two long term concrete stress-strain behaviors were used and compared to the instantaneous or short term load. The expressions for the strength interaction of FRP prestressed concrete columns are derived from the strain compatibility and equilibrium conditions and are presented herein. Based on this preliminary investigation, it was concluded that the long term loadings and the tensile creep rupture of FRP tendons considerably influence the column strength. The results of the column cross sections considered herein show that disregarding long term effects may be conservative provided that rupture of FRP tendons does not occur in the columns.

**Keywords:** concrete columns; FRP tendons; long term loads; non-slender; prestressing; tensile rupture; ultimate load

**Ching Chiaw Choo**, Ph.D., is currently a research professor at the Kentucky Transportation Center and the Department of Civil Engineering, University of Kentucky, Lexington, KY. His research interests include design and strengthening of concrete structures, fiber-reinforced polymer composites, and seismic design and analysis.

ACI Member, **Issam E. Harik**, is the Raymond-Blythe Professor of Civil Engineering, University of Kentucky, Lexington, KY. He is a member of ACI Committee 440, Fiber Reinforced Polymer Reinforcement. His research interests include structural, seismic, hazard mitigation, and bridge analysis and design, fiber-reinforced polymer composites, reinforced and prestressed concrete, and plates and shells.

ACI Fellow, **Hans Gesund**, is professor of Structural Engineering, University of Kentucky, Lexington, KY. His research interests include structural engineering, reinforced concrete, limit design of reinforced concrete slabs and frames, practical design applications.

### INTRODUCTION

The use of prestressing in a concrete column may be counter-intuitive at first glance. However, as pointed out by Naaman (1982), prestressing with steel tendons reduces column resistance to short term compression loadings, but improves column resistance to bending. Additionally, prestressing increases column resistance to cracking; which in turn improves the flexural performance in the service load range. The use of fiber reinforced polymer (FRP) tendons to replace steel tendons in prestressing applications is promising as FRP tendons are non-conductive and non-corrosive. In addition, they have or can be produced with lower elastic moduli than steel prestressing reinforcement, which helps to maintain prestress and reduce prestress losses due to the effects of creep and shrinkage in concrete.

ACI Committee 440 (2004) provides an overview on the history and use of FRP for prestressing applications and a description of the material properties of FRP. The document presents the current state of research, development, and applications that used FRP as prestressed reinforcement. In regard to axial prestressed compression members, studies on prestressed concrete piles (Iyer 1995; Iyer et al. 1996; Arockiasamy and Amer 1998; and Schiebel and Nanni 2000) concluded that, in general, FRP prestressed piles performed satisfactorily (i.e. no loss of effective prestress under impact/driving). Other than the aforementioned research work, studies on concrete columns prestressed with FRP tendons are scarce.

### RESEARCH OBJECTIVE AND SIGNIFICANCE

This paper presents a preliminary analytical investigation of the behavior of non-slender concrete columns prestressed with FRP tendons with emphasis on long term loading effects on the strength interactions ( $P_n$ - $M_n$ ) of the columns. The examination of prestressed concrete elements under long term or sustained loading conditions is warranted since effective prestress is assumed to be sustained throughout the member's

service life. Therefore, considerations of creep and shrinkage of concrete and creep rupture of FRP tendons are essential.

## CONCRETE COLUMNS PRESTRESSED WITH FRP TENDONS

### Derivation of strength interaction relationships

The general expressions for a set of axial load ( $P_n$ ) and moment ( $M_n$ ) of a rectangular column cross section are based on linear strain distribution (as shown in Figure 1) and equilibrium conditions (Choo 2005) (see Notations)

$$P_n = \sum_{i=1}^N C_{ci} + \sum_{j=1}^n F_{pjf} - \sum_{k=1}^m C_{cfk} \quad (1)$$

$$M_n = \sum_{i=1}^N C_{ci} \left( \frac{h}{2} - d_{ci} \right) + \sum_{j=1}^n F_{pjf} \left( \frac{h}{2} - d_{pjf} \right) - \sum_{k=1}^m C_{cfk} \left( \frac{h}{2} - d_{pjf} \right) \quad (2)$$

In equation (1), the stresses in concrete in the compression region are determined for finite strips of concrete (Dunham and Gesund 1960), analogous to having multiple layers of reinforcement. In this case, the average stress of concrete in strip  $i$  ( $f_{ci}$ ) can be determined from the concrete strain at the centroid of strip  $i$  ( $\varepsilon_{ci}$ ), using a chosen concrete stress-strain relation. In this study, in lieu of the rectangular stress block assumption of ACI 318-05, the concrete stress-strain curve shown in Figure 2(a) is used. The resulting concrete compression force ( $C_{ci}$ ) in strip  $i$  can be expressed as

$$C_{ci} = f_{ci} A_{ci} \quad (3)$$

$A_{ci}$  is the area of concrete strip  $i$ . The distance measured from the extreme concrete compression fiber to the centroid of concrete strip  $i$  is defined as  $d_{ci}$ .

When determining either  $P_n$  [Eq. (1)] or  $M_n$  [Eq. (2)], the corresponding concrete force ( $C_{cfk}$ ) at the reinforcement location ( $d_{pjf}$ ), measured from the extreme concrete compression fiber to the centroid of the reinforcement, can be subtracted knowing the concrete stress ( $f_{cfj}$ ) at the reinforcement location  $j$  and the prestressing compression reinforcement area ( $A_{pjf}$ ):

$$C_{cfk} = A_{pjf} f_{cfj} \quad (4)$$

The force ( $F_{pjf}$ ) in Eq. (1) produced by the FRP prestressing tendon at location  $j$  can be expressed as follows

$$F_{pjf} = A_{pjf} E_{pf} \varepsilon_{pjf} \quad (5)$$

where  $E_{pf}$  is the Young's modulus of the FRP prestressing tendons.  $\varepsilon_{pjf}$  is the strain in the prestressing tendon at level  $j$ , and is dependent upon two factors: (a) the effective strain

( $\varepsilon_{pfe}$ ) in the prestressing tendon due the effective prestressing stress, and (b)  $\Delta\varepsilon_{pi}$ . In Figure 1, a rectangular cross section with two layers (i.e., layer 1 and layer 2) of reinforcements is shown for illustrative purposes.

$$\varepsilon_{pfe} = f_{fe}/E_{pf} \quad (6)$$

$f_{fe}$  in Eq. (6) is the effective prestress of the tendons after accounting for all losses such as the elastic shortening of concrete, relaxation of stressed tendons, creep and shrinkage of concrete, etc.

The general expression of  $\Delta\varepsilon_{pi}$  can be determined from the strain diagram

$$\Delta\varepsilon_{pi} = \varepsilon_{cu} \left( \frac{kd - d_{pfi}}{kd} \right) + \varepsilon_{ce} \quad (7)$$

The distance measured from the extreme concrete compression fiber to the location of neutral axis is defined as  $kd$ . The maximum usable short term concrete compression strain as defined in ACI 318-05  $\varepsilon_{cu} = 0.003$  is adopted in this study. The initial concrete strain ( $\varepsilon_{ce}$ ) is a function of the effective prestressing, and for a column, it is assumed to be uniform across the column section [i.e. prestressing tendons are assumed to be uniformly stressed and distributed symmetrically in a concrete column.  $\varepsilon_{ce}$  can therefore be estimated as

$$\varepsilon_{ce} = \frac{A_{pf} f_{fe}}{A_{ct} E_c} \quad (8)$$

$A_{pf}$  in Eq. (8) is defined as the total area provided by the prestressing tendons ( $\Sigma A_{pfi}$ ).  $A_{ct}$  is the concrete area; which for post-tensioning application would be the gross cross section area minus  $A_{pf}$ . For pre-tensioning, the transformed area would be used.  $E_c$  is the Young's modulus of concrete.

The combination of Eqs. (6) and (7) is the strain in prestressing tendon ( $\varepsilon_{pfi} = \varepsilon_{pfe} + \Delta\varepsilon_{pi}$ ) at location  $i$ . The complete strength interaction ( $P_n$ - $M_n$ ) relations can be derived using Eqs. 1 to 8, repeated for a series of assumed locations of the neutral axis.

The column cross section shown in Figure 3 will be used for illustrative purposes in this paper. The tensile properties, such as the modulus of elasticity ( $E_f$ ) and the ultimate strain ( $\varepsilon_{fult}$ ), assumed for aramid fiber-reinforced polymer (AFRP) and carbon fiber-reinforced polymer (CFRP) prestressing tendons in Figure 2(b) are consistent with the average of values reported in the literature. Glass fiber reinforced polymer (GFRP) tendons are excluded from the example due to the fact that the material is more susceptible to tensile creep rupture when subjected to high tensile stress level (Yamaguchi 1997).

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The modulus of elasticity in compression is assumed to be the same as that in tension. While, not routinely reported, the ultimate compressive strain of FRPs is generally expected to be smaller than the ultimate tensile strain (Deitz et. al. 2003). In this study, the ultimate compressive strain ( $\epsilon_{fuc}$ ) is assumed to be 80% that of the ultimate tensile strain ( $\epsilon_{fut}$ ) of FRP tendons (i.e. for the illustrative examples shown in Figures 5, 7 and 8).

The instantaneous or short term strength interactions of non-slender concrete columns prestressed with 270-ksi steel (Figure 4), and AFRP and CFRP tendons (Figure 5) are derived based on the maximum usable concrete compressive strain of 0.003 per ACI 318-05. In lieu of the ACI rectangular stress block, the instantaneous or short-term concrete stress-strain relationship used is shown in Figure 2(a). The frequently used relationship is described by a parabolic expression for the ascending branch, and a linear expression as the descending branch (Ford et al. 1981):

$$f_c = 0.85 f'_c \left[ \frac{2\epsilon_c}{\epsilon_o} - \left( \frac{\epsilon_c}{\epsilon_o} \right)^2 \right], 0 \leq \epsilon_c < \epsilon_o \quad (9)$$

$$f_c = 0.85 f'_c [1 - 20(\epsilon_c - \epsilon_o)], \epsilon_o \leq \epsilon_c \leq \epsilon_{cu} \quad (10)$$

$f_c$  is the concrete stress in compression (ordinate) shown in Figure 2(a).  $\epsilon_c$  is the instantaneous or short-term concrete strain in compression (abscissa).  $\epsilon_{cu}$  is the ultimate concrete compression strain and for short term loading it is the ACI maximum usable strain of 0.003.  $\epsilon_o$  is the concrete strain corresponding to the maximum concrete compression stress,  $f'_c$  and is expressed as

$$\epsilon_o = \frac{1.7 f'_c}{E_c} \quad (11)$$

where  $E_c = 33 w_c^{1.5} \sqrt{f'_c}$  (ACI 318-05), and  $w_c$  = unit weight of concrete in lbs/ft<sup>3</sup>.

The stress-strain relationships for AFRP and CFRP tendons used as illustrative examples are shown in Figure 2(b). In general, they behave linear-elastically up to rupture.

Figure 4 illustrates the strength interactions of concrete columns prestressed with 270-ksi steel tendons. The material properties of the 270-ksi steel can be found in PCI design handbook (2004). The columns in Figure 4 are pre-tensioned to concrete compressive stresses ( $f_{pc}$ ) of 225 psi ( $\approx 1.6$  MPa), 550 psi ( $\approx 3.8$  MPa), and 1,100 psi ( $\approx 7.6$  MPa), respectively. It should be noted that the 225 psi ( $\approx 1.6$  MPa) is the limiting stress specified in ACI 318-05 for columns to be classified as prestressed axial compression members and therefore not subjected to the reinforcement ratio limits (ACI 318-05 Section 10.9) for non-prestressed axial compression members.

The concrete compressive stress ( $f_{pc}$ ) is a function of the effective prestress force ( $P_{fe} = f_{fe} A_{pf}$ ), and is

$$f_{pc} = \frac{f_{fe} A_{pf}}{A_g} \quad (12)$$

Figure 5 shows examples of strength interactions of non-slender concrete columns prestressed with AFRP and CFRP tendons. In addition to the aforementioned three concrete compressive stress levels, strength interactions with  $f_{pc} = 0$  psi (0 MPa) are also included to illustrate the effect of prestressing.  $f_{pc} = 0$  psi (0 MPa) implies that the columns are essentially *reinforced* and do not abide by the limit for *prestressed* concrete per ACI 318-05, and therefore only serve as an illustrative example. Overall the concrete columns prestressed with FRP tendons show a reduction of axial load and a slight gain in bending resistance as prestress increases; especially at the lower axial load levels. The gains are more pronounced in concrete columns prestressed with FRP tendons of lower modulus of elasticity such as AFRP in this case. The gain is beneficial for columns subjected to large bending moments and low axial compression. It should be noted that neither the AFRP nor the CFRP tendons experience tensile rupture under the short-term loading condition.

### **Long term concrete loadings effect on strength interaction diagrams**

Long term concrete loads – In order to develop more accurate strength interaction diagrams of prestressed concrete columns, the time-dependent behavior of the materials must be considered, especially when dealing with concrete columns prestressed with FRPs. Transfer of load from concrete to reinforcement occurs under long term sustained loading as a result of stiffness reduction in concrete due to creep. Hence, the mechanism increases the tensile strain carried by the reinforcement. Therefore, the increase in tensile strain could be significant for concrete columns prestressed with FRP to a point of tensile rupture and subsequent failure of the column. Additionally tensile creep rupture of FRP tendons, reported in Seki et al. 1997; Yamaguchi et al. 1997; and Zou 2003, may compound such effect, and hence must be considered.

Two long term concrete stress-strain relationships (Choo et al. 2003) are used to examine their effects on ultimate strength interactions of concrete columns prestressed with FRP tendons. Creep rupture of FRP tendons is accounted for by limiting the ultimate tensile strain ( $\epsilon_{fut}$ ) of the respective FRP tendons to various percentages as illustrated later in the paper. The two long term relationships, along with instantaneous concrete stress-strain relationship, are presented in Figure 6.

Shrinkage in concrete is accounted for in the two long term concrete relationships by assuming a shrinkage strain,  $\epsilon_{sh}$ , (i.e.  $2 \times 10^{-4}$  to  $12 \times 10^{-4}$ ), causing the shift of the long term curves to the right in Figure 6. The *typical* long term (TLT) curve in Figure 6 is obtained by adding creep strain,  $\epsilon_{cr}$ , to the instantaneous concrete strain,  $\epsilon_c$ . The consequence is a reduction of stiffness; this process is analogous to adjusting the short term modulus of elasticity of concrete. The derivation of  $\epsilon_{cr}$  can be found in Choo et al. (2003).

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The *realistic* long term (RLT) concrete stress-strain curve was derived (Choo et. al. 2003) to represent a more realistic load path for concrete in compression. Generally, creep and shrinkage will occur in concrete columns under long-term service load ( $\alpha_{cs}f_c'$ ; where  $\alpha_{cs}$  is a service load factor typically ranging from 0.3 to 0.6). This part of the RLT curve, therefore, resembles the initial part of the TLT curve up to a prescribed service load. To obtain the ultimate load, the load would then be applied at a relatively quick rate to failure; hence the latter part resembles the ST curve. For complete details of TLT and RLT refer to Choo et al. (2003).

Only the *ultimate* strength interactions of prestressed concrete columns with FRP tendons are examined herein (i.e. time-step behaviors are excluded). At ultimate conditions, in the examples in Figures 7 and 8, creep and shrinkage strains lead to increase in ultimate concrete strains. They are taken as 0.0120 and 0.0052 for TLT and RLT loadings, respectively.

Impact of long term loads on strength interaction diagrams – Figures 7(a) and 7(b) show the *ultimate* strength interaction diagrams of non-slender concrete columns prestressed with AFRP and CFRP tendons under the three different concrete loadings for the three concrete stress-strain relationships: (a) instantaneous or short term (ST), (b) *typical* long term (TLT), and (c) *realistic* long term (RLT) loading relationships. These columns (Figure 7) are pre-tensioned with a minimum effective prestress ( $f_{pe}$ ) in concrete equaled 225 psi ( $\approx 1.6$  MPa) per ACI 318-05.

Comparing the ultimate strength interactions generated for the instantaneous (ST) and *realistic* long term (RLT) concrete loading in Figure 7, it can be seen that they behave somewhat similarly (i.e. no particular discernible gain or loss in strength). This is due to the fact that the *softening* (or reduction of stiffness) of concrete is compensated almost fully by the increase in strain and stress in the FRP tendons. At this particular prestress level, increase in tensile strain of FRP tendons due to an increase in the ultimate concrete compression strain did not lead to FRP rupture. Therefore, the failure of such columns is defined by concrete crushing, as the tensile strains in FRP tendons are smaller than their ultimate tensile strains ( $\epsilon_{fut}$ ).

In the same Figure 7, comparing the ultimate strength interactions of the instantaneous (ST) and *typical* long term (TLT) loading, there is a noticeable gain in overall strength. In this case, while concrete softens quite a bit under the long term loading, the column strength is well-compensated by the FRP tendons due to a tremendous increase in strain (i.e. note that the ultimate concrete strain increases from 0.003 to 0.0120 in TLT loading). This allows a proportional gain in tensile stress in the FRP tendons. However, it can also be observed at a certain axial load level FRP tendons in the columns ruptured. Choo et al. (2006) described this particular failure type as *brittle-tension* failure; where FRP bars (or tendons) rupture prior to concrete reaching its ultimate strain. This failure type can be sudden.

Impact of creep rupture on strength interactions – In Figure 8 creep rupture of FRP tendons is considered in conjunction with *typical* long term (TLT) concrete loading. In

the example of Figure 8, the ultimate tensile strain ( $\epsilon_{fu}$ ) of FRP tendons is reduced to various percentages: 80, 60, 40, and 20%, respectively, in anticipation of tensile creep rupture failure of the tendons.

With the inclusion of the creep rupture, it appears that brittle-tension failure of concrete columns prestressed with FRP tendons is accelerated as the percentage decreases. Although not illustrated herein, brittle-tension failure could have occurred when realistic long term concrete loading is considered. Creep rupture therefore is an important consideration in the analysis and design of concrete columns prestressed with FRP tendons.

## SUMMARY AND CONCLUSION

This paper presents a preliminary examination of the strength interactions of non-slender concrete columns prestressed with FRP tendons. Of particular interest are the strength interactions of the columns under long term concrete loadings, and the possibility of creep rupture of FRP tendons. The expressions for determining the strength interactions of concrete columns prestressed with FRP tendons are presented. The derivation is consistent with ones used for concrete columns prestressed with steel tendons. Two long term concrete stress-strain relationships, along with an instantaneous or short term concrete relationship, were used in this investigation. At ultimate loading conditions, the long term phenomena increase the maximum usable concrete compression strain. The analytical investigations also accounted for the possibility of creep rupture failure of FRP tendons under sustained loading.

Based on the analytical investigation, the followings can be concluded:

- The short term strength interactions of concrete columns prestressed with FRP tendons are consistent to those prestressed with steel tendons.
- There is a gain in bending resistance at low axial load levels as effective prestress increases.
- The investigations considering long term stress-strain relationships show potential gain in overall strength interactions.
- However, the results also show the potential of tensile rupture of FRP tendons when long term loadings are considered. The *brittle-tension* failure, is due to the tensile strain ( $\epsilon_{ft}$ ) of FRP tendons exceeding their ultimate tensile strain ( $\epsilon_{fu}$ ). The failure is particularly pronounced under *typical* long term (TLT) loading since the ultimate concrete compression strain is increased multifold.
- When creep rupture of FRP tendons is considered, the investigation shows that brittle-tension failure in concrete columns prestressed with FRP tendons can either be initiated or accelerated. It is therefore advisable that creep rupture be considered in this type of analysis.
- In summary, for the prestressed concrete columns presented in this study, the long term effects lead to a gain in strength and can be conservatively disregarded provided that *brittle tension* failure of FRP tendons can be prevented.

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This analytical investigation presents an initial assessment of concrete columns prestressed with FRP tendons subjected to long term loadings. The investigation should be complemented by experimental studies, and should be extended to slender columns prestressed with FRP.

### NOTATION

$A_{ci}$  = area of concrete in strip  $i$

$A_{ct}$  = Net concrete area or transformed concrete area

$A_g$  = cross sectional area of column

$A_{pf}$  = total area of prestress reinforcement

$A_{pj}$  = area of prestress reinforcement at location  $j$

$b$  = width of column (dimension parallel to the bending axis)

$C_{cj}$  = concrete force at the reinforcement layer  $j$

$C_{ci}$  = average concrete force in strip  $i$

$d_{ci}$  = distance measured from the extreme concrete compression fiber to the centroid of concrete strip  $i$

$d_{pj}$  = distance measured from the extreme concrete compression fiber to the centroid of reinforcement  $j$

$E_c$  = Young's modulus of concrete

$E_{fc}$  = Young's modulus of FRP reinforcement in compression

$E_{ft}$  = Young's modulus of FRP reinforcement in tension

$E_{pf}$  = Young's modulus of FRP reinforcement

$f_c$  = concrete stress

$f_{fe}$  = effective prestress

$f_{pc}$  = concrete stress due to effective prestress

$F_{pj}$  = FRP tendon force in layer  $j$

$h$  = height of column (dimension perpendicular to the bending axis)

$i$  (or  $j$ ) = layer designated as  $i$  (or  $j$ )

$kd$  = distance measured from the extreme concrete compression fiber to the location of the neutral axis

$M_n$  = nominal bending moment

$M^*$  = normalized moment =  $M_n/bh^2$

$P_{fe}$  = effective prestress force

$P_n$  = nominal axial load

$P^*$  = normalized axial load =  $P_n/bh$

RLT = realistic long term concrete loading stress-strain curve

TLT = typical long term concrete loading stress-strain curve

$\epsilon_{ce}$  = uniform concrete strain due to effective prestress

$\epsilon_{cu}$  = maximum usable concrete compression strain (e.g. 0.003 per ACI 318-05)

$\epsilon_{fut}$  = ultimate strain of FRP reinforcement in tension

$\epsilon_o$  = concrete strain corresponding to the maximum concrete compression stress,  $f'_c$

$\epsilon_{pj}$  = strain in the prestressing tendon at location  $j$

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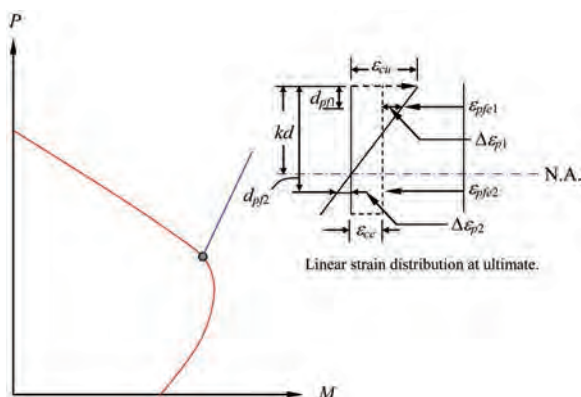
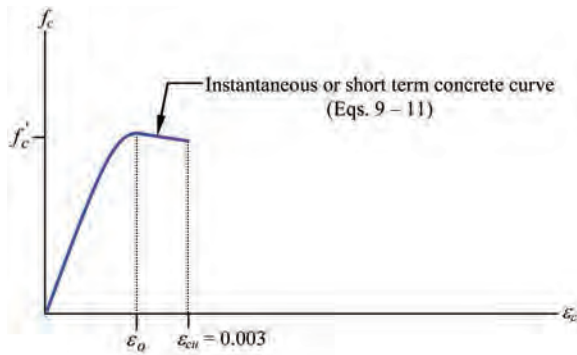
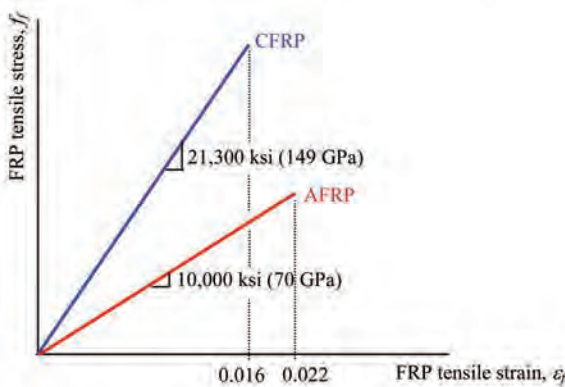


Figure 1—Strength interaction diagram of a prestressed concrete column.



(a) Short term concrete stress-strain curve



(b) FRP tensile stress-strain curves

Figure 2—Material stress-strain properties used to generate Figures 4, 5, 7, and 8.

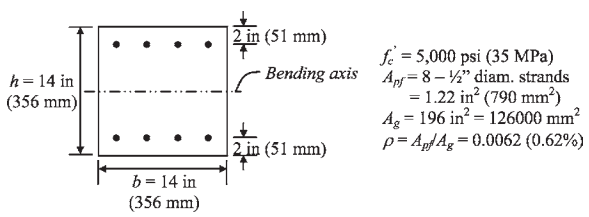


Figure 3—Column cross section properties used to generate Figures 4, 5, 7, and 8.

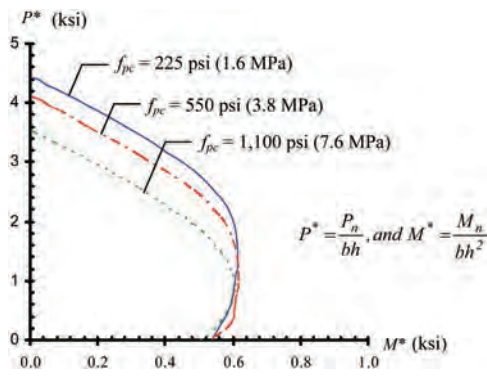


Figure 4—Strength interactions of nonslender concrete columns prestressed with 270-ksi steel tendons. (1 ksi = 6.895 MPa)

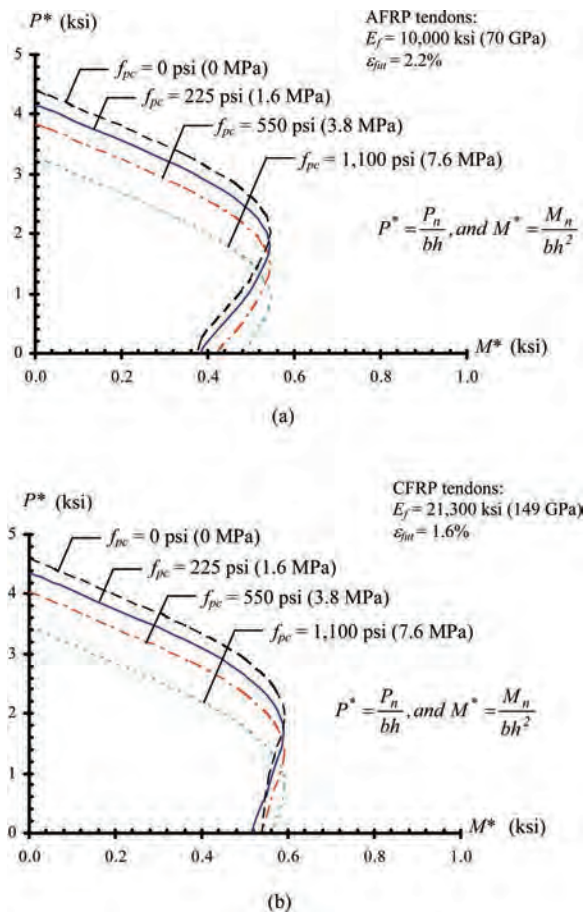


Figure 5—Strength interactions of nonslender concrete columns prestressed with: (a) AFRP tendons and (b) CFRP tendons. (1 ksi = 6.895 MPa)

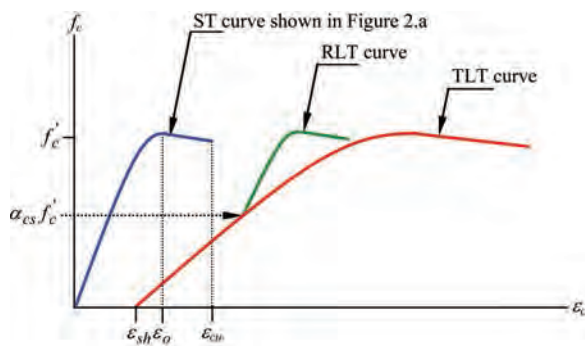


Figure 6—Concrete stress-strain curves (Choo et. al. 2003).

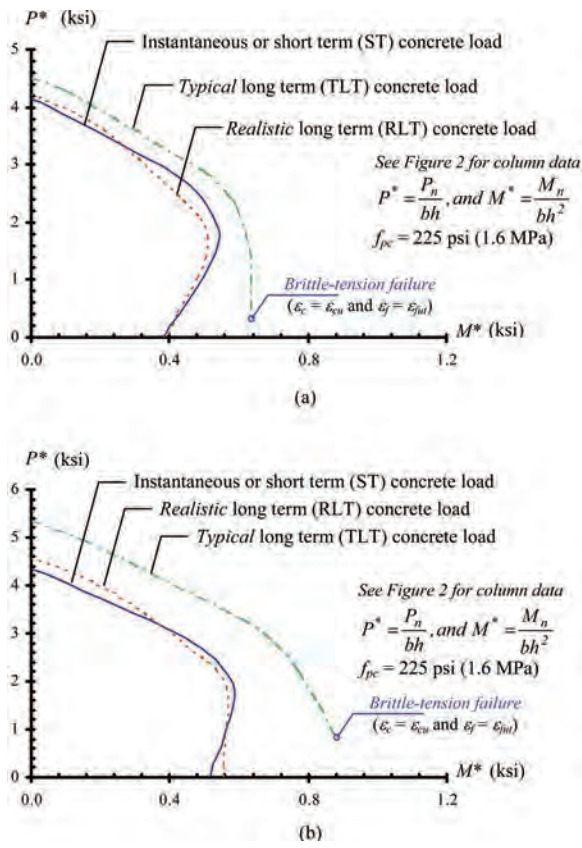
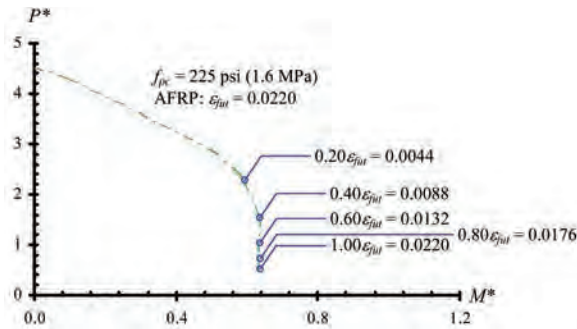
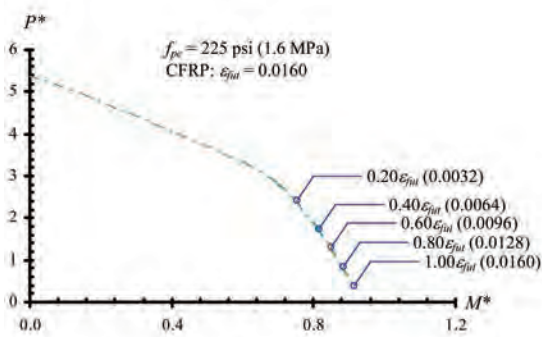


Figure 7—The effects of long term concrete loadings on strength interactions of non-slender prestressed concrete columns (refer to cross section in Fig. 3) with: (a) AFRP tendons; and (b) CFRP tendons. (1 ksi = 6.895 MPa)



(a)



(b)

Figure 8—The effects of creep rupture on typical long term strength interactions of non-slender prestressed concrete columns (refer to cross section in Fig. 3) with:  
(a) AFRP tendons; and (b) CFRP tendons. (1 ksi = 6.895 MPa)

# Modeling Time-Dependent Deformations of a Masonry Wall Post-Tensioned with CFRP

by **M.M. Reda Taha, K.-K. Choi, M. Tait, and S.L. Lissel**

**Synopsis:** This paper summarizes the experimental and analytical research work for simulating a recently built retaining masonry wall post-tensioned with carbon fiber reinforced polymer (CFRP) tendons. A group of creep and shrinkage experiments on masonry prisms similar to that used in the masonry diaphragm wall is discussed. A theoretical investigation using numerical models to describe the time-dependent deformations of the masonry diaphragm wall was performed. Creep coefficients based on the experiments are compared to common creep values based on other research investigations and on well established code models. A finite element model of the masonry diaphragm wall was developed incorporating the non-linear time-dependent properties of masonry to investigate the long-term deformations and stress redistribution in the walls over time. Deformations of the walls are compared to observations from a continuous monitoring system observing the lateral deformations and strains and stresses of the wall.

**Keywords:** creep; FE analysis; fiber-reinforced polymers (FRP); masonry; masonry post-tensioning

ACI Member **Mahmoud M. Reda Taha**, is an Assistant Professor in the Department of Civil Engineering, the University of New Mexico. He received BSc and MSc from Ain Shams University, Cairo, Egypt and PhD from the University of Calgary, Canada. His research interests include structural health monitoring, and using FRP.

ACI Member **Kyoung-Kyu Choi**, is a Research Assistant Professor in the Department of Civil Engineering, the Univ. of New Mexico. He received his BSc, MSc and PhD in architecture from Seoul National Univ. His research interests include shear strength and seismic design of RC structures

**Matthew Tait**, is an Assistant Professor, in the Department of Geomatics Engineering, The University of Calgary. He received his BEng from John Moore's University and his PhD from the University of Leeds. His research interest include vision metrology, laser scanning, and engineering deformation

**Shelley L. Lissel**, is an Assistant Professor in the Department of Civil Engineering, The University of Calgary, Canada. She received her BSc and PhD from the University of Calgary. Her research interests include masonry structures and FRP.

### INTRODUCTION

In 2002, a new masonry diaphragm retaining wall post-tensioned using Carbon Fiber Reinforced Polymer (CFRP) tendons was constructed on campus at the University of Calgary, Calgary, Canada. The retaining wall is convex to the soil as shown in Figure 1 (a) and (b). Earth pressure loading due to retaining soil is present on the lowest 1.5 m of the wall only. The wall was designed as a cantilever retaining wall constructed on concrete footings (Lissel et al. 2005). The design loads for the wall included lateral earth pressure, ramp surcharge, and handrail loading. The cross section of the wall is shown in Figure 1 (c).

While the post-tensioning was an inherent part of the design and construction process, the flexural capacity of the wall did not govern the design. Shear requirements governed in determining the required depth of the masonry diaphragm wall. The wall was post-tensioned at the age of 14 days and back-filled at the age of 19 days. The detailed description of the post-tensioning process and the challenges of post-tensioning the masonry diaphragm walls using CFRP Leadline<sup>®</sup> tendons with a new anchoring system may be found elsewhere (Lissel and Shrive 2003; Lissel et al. 2005).

This paper reports on a new research investigation to provide realistic modeling of the post-tensioned masonry diaphragm wall while considering long term time-dependent effects. Preliminary investigations for modeling this post-tensioned diaphragm wall showed the importance of creep modeling on the prediction of masonry wall deformations (Lissel et al. 2004). The fact that masonry has significant creep behaviour and that the wall is post-tensioned magnified the importance of the time-dependent effects on the structural behaviour. Therefore, an experimental program examining creep in masonry materials similar to that used in the post-tensioned masonry retaining wall

was performed. The experimental program aimed at establishing a realistic prediction of masonry creep. The creep properties derived from these experiments are then used to represent the time-dependent behaviour of the masonry materials in the finite element (FE) modeling.

The wall was constructed in early July 2002 and its deformation monitoring started after backfilling at 19 days of age. Early deformation monitoring was performed using a total station measuring reflective targets placed on the wall's surface. In April 2004 (660 days of age) and in May 2006 (1430 days of age) close-range photogrammetry (CPR) was implemented for determining deformations. Moreover, strain and humidity monitoring are being performed continuously since the wall was constructed. Finally, the lateral time-dependent deformations predicted by the FE model are compared to the deformations monitored using CRP. The FE modeling prediction was estimated at 4 specific ages being 19 days of age, 660 days of age, 1000 days of age and 1430 days of age respectively. The primary objectives of this paper are: (1) to present the effect of creep and shrinkage on long-term deformations of masonry retaining walls, (2) to demonstrate the significance between long-term deformation predictions using FE models incorporating results of experimentally observed creep and shrinkage and long-term deformation predictions based on current design codes and finally (3) to report on the comparison between the predicted and monitored deformations of the wall.

### CONSTRUCTION OF RETAINING WALL

The construction of the wall took place in July 2002. The length and height of the retaining wall are 7.5 m (24 ft 7.3 in) and 2.6 m (8 ft 6.4 in), respectively, and the radius of curvature is 7.5 m. The cross-section of the wall is presented in Fig. 1(c). The compressive strength and the modulus of elasticity of the masonry were determined experimentally to be 20.1 MPa (2.92 ksi) and 8570 MPa (1244 ksi), respectively (Lissel et al 2005). Post-tensioning was performed using 8 mm (0.31 in) CFRP Leadline<sup>®</sup> tendons at 14 days of age.

### METHODS

#### Experimental investigation of masonry creep and shrinkage

The creep behaviour of the masonry was experimentally investigated. The creep tests were performed on masonry prisms subjected to three different stress levels (0.7 MPa, 1.8 MPa and 3.61 MPa) representing approximately 3.5%, 9% and 18% of the ultimate compressive strength of the masonry prisms (20.1 MPa), where 1 MPa = 0.145ksi. The experiments included examining loaded and unloaded prisms. The unloaded control prisms were subjected to the same environmental conditions as their counterpart-loaded prisms to account for shrinkage and thermal changes. The masonry prisms for the creep experiments were produced using standard fired clay brick and Standard type S mortar. The masonry units were from the original supply of brick and the mortar was from the same plant as the materials used in constructing the post-tensioned diaphragm wall. Each creep testing frame consisted of three masonry prisms as shown in Figure 2. Each prism

consisted of four masonry units and three mortar joints. The three prisms were loaded together. The prisms were separated with a relatively thick steel plate (12.7 mm (0.5 in) thickness) to ensure appropriate stiffness for loading the masonry prisms. The stress was kept constant over the prisms using an automatic hydraulic loading system. The adopted creep experimental set-up described here is a classical set-up recommended and used by other researchers (Shrive et al. 1997). Metal Demec<sup>TM</sup> points were glued on the prism surface to allow monitoring masonry displacement with time due to creep. The test prisms were stored under two environmental conditions; the first group in a room with ambient humidity (RH~25%), the second group in a room with controlled humidity (RH=50%). The specimens were loaded at three different ages 7, 14, and 28 days of age. All specimens were kept in the laboratory at a temperature of  $20 \pm 2$  °C. The creep testing parameters resulted in 13 experimental groups.

### **Modeling creep of masonry**

It is well established that for a given sustained stress  $\sigma$  applied at time  $t_0$ , a specific creep  $c_{sp}(t, t_0)$  evaluated at time  $t$  can be defined as in Equation (1)

$$c_{sp}(t, t_0) = \frac{\varepsilon(t, t_0)}{\sigma} \quad (1)$$

where  $\varepsilon(t, t_0)$  is the creep strain at time  $t$  due to application of the stress  $\sigma$  at time  $t_0$ . The ratio between the specific creep ( $c_{sp}$ ) and the specific elastic strain ( $\varepsilon_{sp}$ ) defined as  $\varepsilon(t_0) / \sigma$  is equal to the ratio of the creep strain and the elastic strain. This ratio between the time dependent creep strain and the instantaneous elastic strain is widely used in structural design and is known as the creep coefficient  $\phi(t, t_0)$  and can be computed as:

$$\phi(t, t_0) = \frac{c_{sp}(t, t_0)}{\varepsilon_{sp}} \quad (2)$$

Moreover, the creep compliance  $J(t, t_0)$  can be defined as

$$J(t, t_0) = \frac{1}{E(t_0)} + c_{sp}(t, t_0) \quad (3)$$

While researchers prefer using creep compliance  $J(t, t_0)$  defined in equation (3) for its recognized physical meaning (Van Zijl 1999 and Gardner and Lockman 2001), most design codes still use the creep coefficient  $\phi(t, t_0)$  for its simple implementation in the structural analysis process. Here, the creep representation was limited to the creep coefficient  $\phi(t, t_0)$ . To evaluate the creep coefficient  $\phi(t, t_0)$  of the masonry materials used in the post-tensioned diaphragm wall, the experimental observations performed on masonry prisms were used with equations (1) to (2). Typical experimental observations

of creep of the masonry assemblage with loading age ( $t_0 = 14$  days) at two different stress levels are shown in Figure 3.

A number of numerical and rheological creep models (Schapery 1969; Findley et al. 1976; Neville et al. 1983, Shrive et al 1997; Nordin and Varna 2006) were examined to represent creep of masonry. It was found that the creep experimental data presented here can be well represented by a generalized exponential model (Neville et al. 1983) as represented by Equation (4)

$$\phi(t, t_0) = \phi_u(t_0) [1 - e^{(-t/\tau)}] \quad (4)$$

where  $\phi_u(t_0)$  = creep coefficient at infinite time, and  $\tau$  = time constant denoting the time when 63 % of the creep has occurred. Figure 3 also shows typical creep results using the theoretical prediction presented in equation (4) versus the experimental creep observations in the masonry prisms. The creep prediction parameters  $\phi_u(t_0)$  and  $\tau$  were determined using principles of curve fitting to minimize the root mean square error. Further details about the creep experiment and the modelling investigation will be presented elsewhere.

### Modeling shrinkage of masonry

The experimental investigation of creep included identifying shrinkage of masonry by observing the time-dependent deformations of non-loaded prisms (Fig. 2(b)). Using these observations basic shrinkage (stress-independent shrinkage) of masonry was evaluated. The shrinkage of masonry was theoretically predicted using a generalized exponential model similar to that used to predict creep. Therefore, shrinkage of masonry can be predicted as:

$$\varepsilon_{sh}(t, t_0) = \varepsilon_{sh}(t_0) [1 - e^{(-t/\tau_s)}] \quad (5)$$

where  $\varepsilon_{sh}(t_0)$  = shrinkage of masonry at infinite time, and  $\tau_s$  = time constant denoting the time when 63 % of the shrinkage has occurred. Figure 4 shows the shrinkage strain evaluated by the theoretical prediction using Eq. (5) in comparison with measured masonry shrinkage.

### FE simulation of the masonry diaphragm wall

In the present study, a three dimensional time-dependent numerical analysis using ANSYS® 7.11 was performed to investigate the long-term behavior (creep and shrinkage) of the post-tensioned masonry diaphragm wall. The model targeted determining the long-term deflection and stress distributions of the wall. In FE modeling, a continuum approach assuming anisotropic homogenized composite of joints and bricks was used. It is well established that a meso-scale model of heterogeneous materials like masonry that considers the brick units and the mortar joints separately can yield more accurate results

than a continuum FE model (Ghosh et al. 2001). However, continuum FE modeling of masonry has also been proposed by other researchers for numerical analysis of masonry structures (e.g. Lourenço 1996, van Zijl 1999, and Berto et al. 2001). Moreover, the creep and shrinkage models presented here are based on continuum measurement of masonry prisms including both the brick units and the mortar joints. Therefore, separate creep and shrinkage characteristics of the masonry units and the mortar joints were not available. This necessitates implementing a continuum FE modeling approach for modeling the time-dependent effects in the masonry wall.

The post-tensioned diaphragm wall was modeled as three shell elements representing the three main segments of the wall (inner flange, outer flange, and web elements). The constitutive model for such shell elements was described in detail by Weaver and Johnston (1984) based on Mindlin theory. The shell elements allow modeling of the nonlinear behaviour of masonry including creep. This element has six degrees of freedom at each node: translations in the nodal x, y, and z directions and rotations about the nodal x, y, and z-axes, which is applicable to moderately-thick shell structures like the masonry wall investigated here. The finite element model of the diaphragm wall is shown in Figure 5.

Considering symmetry conditions in both geometry and loading of the wall, only half of the wall was modeled and symmetric boundary conditions were applied at the center of the FE model. Therefore, a total of 648 shell elements were used to establish the FE model. Given the size and depth of the foundation and the design guidelines presented by Lissel et al. (2005), the wall was modelled to be fully restrained at its base. The existence of the post-tensioning also negates possible cracking at the base which makes such restrained end conditions very possible. The finite element model assumed an initial elastic modulus of masonry of 8312 MPa determined experimentally for similar materials (Lissel et al. 2004). The finite element model incorporated the creep and shrinkage theoretical prediction equations presented in Equations (4) and (5) with  $\phi_u(t_0) = 1.04$ ,  $\tau = 12.2$  days,  $\varepsilon_{sh}(t_0) = 8.5 \times 10^{-5}$ , and  $\tau_s = 1.2$  days (see Figures 3 and 4). The numerical values for these coefficients were derived from the creep and shrinkage experimental data to represent the age of loading, the stress level and the relative humidity conditions of the wall. The significantly small variation of these three parameters along the wall allowed using a single equation to predict creep behaviour.

Finally, it is evident that in addition to creep and shrinkage effects, temperature and humidity cycles result in stress fluctuation (Gayevo 2006). However, it needs to be noted that given the cyclic nature of humidity and temperature effects, the realization of temperature and humidity effects requires modeling of the wall within relatively short time intervals (30 to 60 days long). When relatively long time interval are considered, such as the one here (630 and 1430 days long), the effect of temperature and humidity variations will be relatively small and can be neglected when compared with creep and shrinkage effects. Therefore, temperature and humidity effects were neglected here. Observations and analysis of the temperature and humidity effects within short time intervals can be found elsewhere (Gayevo 2006).

## Estimating prestress losses

While creep can significantly increase the wall deformation, creep can also affect the axial deformation of the wall resulting in prestressing losses. As the unbonded post-tensioning strands were not included as part of the FE model, but considered as an axially applied stress acting on the masonry it was necessary to examine the effect of creep and shrinkage and other possible effects on the post-tensioning loads. Therefore, prestressing losses due to creep and other factors such as elastic shortening, shrinkage and stress relaxation of the CFRP tendons were estimated.

Prestress losses due to elastic shortening of the masonry wall—The initial post-tensioning (PT) stress applied to the wall results in instantaneous elastic shortening of the wall and an immediate drop in the PT forces by the amount of  $\Delta P_{ES}$  as in Equation (6). The general formula for losses due to elastic shortening has been reported by many researchers examining concrete and masonry post-tensioned structures (Sayed-Ahmed et al. 1999):

$$\Delta P_{ES} = P_j \left( \frac{E_p}{E_m(t_0)} \right) \left( \frac{A_p}{A_m} \right) \quad (6)$$

where  $P_j$  is the jacking force,  $E_p$  is the modulus of elasticity for the CFRP tendons reported to be 142 GPa,  $A_p$  is the cross-sectional area of the CFRP tendons, and  $A_m$  is the cross-sectional area of the masonry wall.

Prestress losses due to creep of the masonry wall— It is well established that creep results in significant time-dependent deformations of structural masonry (Lenczner 1974; Binda et al. 1992). The losses in the PT force due to creep were estimated using equation (7) (Taneja et al. 1986, Sayed-Ahmed et al. 1999; Curtin et al. 2004):

$$\Delta P_{CR} = P_j \left( \frac{E_p}{E_m(t_0)} \right) \left( \frac{A_p}{A_m} \right) \phi(t, t_0) \quad (7)$$

Here  $\phi(t, t_0)$  is the creep coefficient of the masonry wall and  $E_m(t_0)$  is the elastic modulus of masonry at the time of loading (14 days) determined experimentally.

Prestress losses due to shrinkage of masonry wall— PT losses due to shrinkage were evaluated using equation (8) considering the shrinkage strains measured in the creep experiments on masonry prisms at 50% relative humidity

$$\Delta P_{SH} = \varepsilon_{sh}(t_o) E_p A_p \quad (8)$$

where  $\varepsilon_{sh}(t_o)$  is the shrinkage strain at infinite time. Based on the experimental investigation of masonry shrinkage,  $\varepsilon_{sh}(t_o) = 85$  microstrain was used. This shrinkage strain was used in evaluating the PT losses due to shrinkage in the masonry wall.

Prestress losses due to stress relaxation of CFRP tendons— While prestressing stress in steel strands is known to decrease due to stress relaxation, researchers have shown that stress relaxation of CFRP strands is negligible (Gayevoiy et al. 2003). Here, to address the expression recommended by other researchers (Sayed-Ahmed et al. 1999) for a conservative estimate of the prestress losses due to stress relaxation of CFRP, equation (9) was used.

$$\Delta P_{RE} = 0.035 P_j \quad (9)$$

It is important to note that such losses are a function of the ratio between the stress in the tendon to its ultimate strength (Sayed-Ahmed et al. 1999).

## MONITORING OF THE MASONRY WALL DEFORMATION

### Deformation monitoring using close-range photogrammetry

Photogrammetry is an efficient measuring technique that has been utilized many times in engineering (Fraser and Shortis, 1995), and has been applied in many areas of civil engineering (Tait and Uren, 1996). The main advantages of CRP in monitoring deformations are normally that 1) It is a full-field method that can capture the state of all the targets at the same instant (shutter speed), 2) That the network can be designed to improve on the accuracies obtained from theodolite observations, and 3) That a high redundancy of observation can be obtained quickly compared to other methods. Its advantages also include speed of capture and motion-freezing, semantic information capture, and non-contact measurement, although some of these advantages have been superseded by laser scanners. Moreover, the redundancy of information is one of the unique advantages of photogrammetry for deformation monitoring. Therefore, close-range photogrammetry (CRP) techniques were used in monitoring the deformations of the post-tensioned masonry diaphragm wall as explained below.

**Observations** — For monitoring the deformation of the post-tensioned masonry wall, a Rollei D7Metric 5 mega-pixel photogrammetric camera was used. The camera was pre-calibrated in the laboratory using self-calibrating bundle adjustment (Shortis et al., 2002) before each observation epoch. Observations of the wall started in April 2004 and the last observation set reported here was recorded in May 2006. Targets of the retro-reflective survey type had already been placed, with the original intention to use total station radial observations of the targets to determine movement. Manual measurement of the target images was required due to the nature of the targets, with the highest accuracy of image observation for this target type being only 1/2 or 2/3 of a pixel, giving resultant observation accuracies on the image of 2-3 microns.

**Network geometry** —The masonry wall had 13 targets placed and two sets of the generic network configuration taken to give very substantial overlap (>60%) between sets. Angles between camera axes are ideally 80-90 degrees, the preferred configuration for triangulation methods. In practice this was not possible to achieve due to surrounding walls and the ground plane; therefore actual convergent angles were approximately 60°.

**Analysis** — The Vision Measurement System (Shortis et al., 2002) was used to manually measure the target image locations on the image sets, before a bundle adjustment simultaneously computed the positions and orientations of the cameras and the target locations for each observation epoch. The deformation of the retaining wall was monitored at two specific epochs:  $t_1$  = April 2004, and  $t_2$  = May 2006. In this study, the standard deviations of the observations in deformation between  $t_1$  and  $t_2$  are calculated as

$$SD = (SD_{tx1}^2 + SD_{tx2}^2)^{0.5} \quad (10)$$

Significance of deformation is defined as the ratio between the change detected in the deformation and standard deviation of combined observations in the deformation for the epoch. The observation accuracy has increased over the three combined epochs due to the use of different combinations of observation method: in epoch 1 (between  $t_0$  and  $t_1$ ) and epoch 2 (between  $t_1$  and  $t_2$ ), transit plus CRP, and CRP plus CRP were used, respectively. As the observation accuracy increases, the significance does not increase. A significant result would give significance greater than 1 as the movement escapes the combined error envelope. Therefore, all deformations are well within the combined error envelopes. The displacement in epoch 1 ( $\Delta t_{0t_1}$ ) and that in epoch 2 ( $\Delta t_{1t_2}$ ) are less than 3.5 mm and 0.4 mm, respectively.

### **Environmental conditions and strain monitoring**

In addition to the deformation monitoring using close-range photogrammetry the monitoring programme also includes monitoring the temperature, the humidity and the strains of the post-tensioned masonry wall and CFRP tendons. Thermocouples were located at every strain gauge location and elsewhere to monitor temperatures in the walls. Control masonry prisms instrumented with thermocouples and strain gauges were placed in the cavities of the walls to obtain readings from unloaded masonry. Strains in the tendons and masonry are measured by strain gauges mounted on the surface of the material. Although measurements were recorded prior to post-tensioning, prior to backfilling, and after backfilling, no significant effects from any loading were noticed in any of the strain gauges due to changes in landscaping such that the actual loading is far less than the design loading. Once the wall was at service conditions it was not expected to see significant changes in the readings in the short term. PT losses due to creep are expected to be observed over the long term. Further details of the environmental conditions and strain monitoring programme can be found elsewhere (Gayevoiy 2006).

## **RESULTS AND DISCUSSIONS**

The experimental investigation of masonry creep and the numerical prediction using the theoretical creep and shrinkage prediction equations based on the experimental observations showed the creep coefficient  $\phi(t, t_0)$  to be significantly affected by stress levels. These results, while agreeing with observations by other researchers (Shrive et al. 1997), are unlike expectations of most existing masonry design codes. The creep coefficient  $\phi(t, t_0)$  for application in the finite element model ranged between 0.33 at 14 days of age and 1.04 at 1430 days of age respectively. Table 1 presents a comparison of

the creep coefficients predicted by the creep equation based on experiments and three masonry codes being the Canadian code (CSA 2004), the European code (Eurocode 2000) and the Australian code (Standards Australia 1998). The comparison is performed at three time instances being 19 days of age, 660 days of age, and 1430 days of age, respectively.

It is evident that the creep coefficient  $\phi(t, t_0)$  for the masonry wall based on the creep experiments and the stress level along the wall is far from those suggested by the Canadian code (CSA 2004). However, the creep coefficient is reasonably close to the mean value proposed by the European code (Eurocode 2000) and not significantly far from that proposed by the Australian code (Standards Australia 1998). The relatively low creep coefficient might be related to the relatively low stress level acting on the wall as a result of the conservative design. It is obvious that the significance of many factors affecting masonry creep and the required complexity of modeling proves the need for further research on creep modeling techniques. It is evident from Table 1 that the creep coefficients provided by most design codes worldwide might be useful for design purposes but cannot be used to predict creep deformations. This is because most of the values are not dependent on the stress level or the relative humidity which have long been recognized to significantly affect the creep deformations (Lenczner et al. 1974, Neville et al. 1983).

The predicted prestress losses in the CFRP tendons, by using Eqs. (6) to (9), due to the various factors including masonry creep and shrinkage, elastic shortening and CFRP tendon relaxation are summarized in Table 2. The maximum post-tensioning losses expected at 660 days did not exceed 6.35 % of the total jacking stress after 1430 days which proves the fact that PT losses in the monitored masonry wall are very insignificant. This might be attributed to the small shrinkage strain and the very negligible stress relaxation of CFRP tendons. Moreover, Figure 6 presents the variation of axial stress distribution in the masonry wall as time passes due to the combined loading of post-tensioning and earth pressure. At initial loading, a maximum stress of 0.48 MPa (0.07 ksi) is developed at the exterior surface of the wall while a minimum stress of 0.12 MPa (0.017 ksi) is developed at the interior surface of the wall where the bending moment from the soil pressure causes tensile stresses. In this Figure, it is noted that as time passes the axial stress developed in the base of retaining wall increases by 10 percent. Furthermore, the axial deformation increased to reach twice the initial deformation. Therefore, continuous simulation and monitoring of the tension stresses at the wall base might be important to ensure the limited probability of cracking. Figure 7 shows the deformed shape of the retaining masonry wall due to the combined effect of post-tensioning and earth pressure. Table 3 presents the statistics for the differential movements of the targets on the masonry wall, together with the accuracy of observation derived from the two measurement periods. It can be observed from Table 3 that the displacements of the points have a mean value very close to zero and similar standard deviations to the observations which were assumed to be normally distributed. These results therefore reflect the noise of the measurement process, and the fact that inference of movement over the observation period would have a very low confidence.

In Table 4, the long-term time-dependent deformations of the masonry wall as obtained from the FE analysis are compared in the z-direction to the deformations recorded using the CRP method at the specific epochs. Moreover, Figure 8 shows the long-term time-dependent deformations of the masonry wall, which are obtained from the FE analysis in comparison with the experimental data and predictions using current design code models including Australian code (1998), Eurocode 6 (2000), and CSA (2004). As shown in the predicted long-term deflection curves, a significant proportion of the creep deflection developed before 100 days. It can be observed that the Eurocode 6 (2000) and the Australian code (1998) underestimated the long-term time-dependent effect compared with the Canadian code (CSA 2004) and the FE model. However, it is important to note that there is a considerable difference between the long-term deflection predicted by the FE model and the monitored deflection. The predicted values are smaller than those observed in the monitoring program measured using CRP. In the case of the masonry walls, the CRP method captured the targets at an accuracy at the same level as multiple-observation theodolite intersection, but provided high redundancy of observation (8-10 observations per point) and is therefore reliable in its estimates of position. The difference between CRP measured and theoretically predicted deformations and the reason higher accuracy was not obtained can be attributed to the use of targets that are not photogrammetric targets (targets selected for use with total station originally). Moreover, three factors affect the limited deformations of the wall: First, the stresses applied to the wall are relatively small with the maximum stress in the wall due to post-tensioning representing only 2% of the compressive strength and thus creep deformations are very limited. Second, the use of CFRP strands eliminates a large part of the post-tensioning losses that usually occur due to stress relaxation (Lissel et al. 2005).

Finally, in the present analysis, only basic shrinkage (stress-independent shrinkage) was considered while stress-dependent shrinkage, commonly known as the Pickett effect (Pickett 1942, and Van Zijl 1999), was not considered. To date, a generally acceptable methodology to quantify stress-dependent shrinkage does not exist. For obtaining reliable long-term deformation data of the retaining wall, further monitoring and investigation using more improved monitoring techniques with better accuracy in the measurement and an improved numerical investigation addressing the limitations discussed above are planned.

### CONCLUSIONS

A finite element model to simulate the time-dependent deformations in a post-tensioned retaining masonry diaphragm wall was developed. An experimental creep programme investigating creep of masonry prisms constructed of materials similar to that used in the masonry diaphragm wall was performed. Theoretical predictions of creep and shrinkage of masonry based on the experimental results were performed. The creep and shrinkage prediction equations were used in the FE analysis to estimate the time-dependent deformation of the wall. While the creep model showed good accuracy in modeling the creep experimental data it is still far from most existing masonry codes. It is evident that the post-tensioning losses of the wall are insignificant due to the very negligible stress-relaxation of CFRP tendons. From the FE simulation, it is demonstrated that most long-

term deformation is developed within 100 days after loading, and significant stress increase due to creep of masonry occurs. Continuous simulation of monitoring of these stresses might be useful to ensure limited cracking probability. Significant difference exists between the wall deformations predicted using the non-linear finite element model and the monitored deformations. This difference might be attributed to the combined effects of neglecting stress-dependent shrinkage and to the limitation of monitoring accuracy, which is related to the relatively small deformations of the structure compared to its size and the use of small reflective targets.

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Table 1— Creep coefficients predicted by masonry codes and theoretical analysis

| Time            | CSA (2004) | Standard<br>Australia 1998 | Eurocode 6<br>(2000) | Theoretical<br>analysis* |
|-----------------|------------|----------------------------|----------------------|--------------------------|
| 14 days of age  | 2.0-4.0    | 0.7                        | 0.5-1.5              | 0.33                     |
| 660 days of age |            |                            |                      | 1.04                     |
| 1000 days       |            |                            |                      | 1.04                     |
| 1430 days       |            |                            |                      | 1.04                     |

\* proposed model using Equation 4 with  $\Phi_u(t_0)=1.04$  and  $\tau=12.2$  days.

Table 2— Prestress losses estimated in the CFRP strands (kN)

| Time               | 14 days | 660 days | 1000 days | 1430 days |
|--------------------|---------|----------|-----------|-----------|
| $\Delta P_{ES}$    | 0.59    | 0.59     | 0.59      | 0.59      |
| $\Delta P_{cr}$    | 0.21    | 0.61     | 0.61      | 0.61      |
| $\Delta P_{sh}$    | 0.04    | 0.60     | 0.91      | 1.30      |
| $\Delta P_{RE}$    | 2.87    | 2.87     | 2.87      | 2.87      |
| $\Delta P_{total}$ | 3.71    | 4.67     | 4.98      | 5.37      |
| Losses %*          | 4.52    | 5.70     | 6.70      | 6.55      |

1 kN = 0.2248 kip

\* Losses % is computed with respect to the jacking load of 82kN (18.4 kip).