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Durable Retrofitting of Concrete Structures

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ABSTRACT

This paper presents a summary of the work carried out over the last several years on reinforced concrete columns. Professor Koji Sakai being part of the research team in 1986-1987 made significant contributions to this program. Initial work was focused on new columns confined by lateral steel and the current research is mostly focused on rehabilitation of existing structures. With the use of newer composite materials such as glass and carbon FRP, a number of innovative techniques have been developed to retrofit columns, slabs and beams that are technically and economically superior to the traditional methods. Laboratory evidence and field data show that with appropriate design, a durable and sustainable rehabilitation with FRP can avoid unnecessary replacement of many structural components. This paper provides results from a few selected experimental and field studies with special emphasis to durability and sustainability as a tribute to Dr. Sakai's work.

INTRODUCTION

Aging, adverse environmental effects and stricter provisions of the new design codes have rendered numerous reinforced concrete structures in the world deficient. Their upgrade is a major engineering challenge that needs special attention. Retrofitting these structures with fibre reinforced polymer (FRP) is becoming popular as an alternative to conventional repair techniques because FRPs have high strength-to-weight ratio, excellent corrosion resistance and flexibility in size and shape. Application of FRP is less labour- and equipment- intensive. From the durability and sustainability perspective FRP provides an attractive solution for repairing the deteriorating infrastructure. In the 1980s, the test frame shown in Figure 1 was especially designed to test columns under a variety of loads including simulated earthquake loads. Professor Sakai during his tenure at Houston was involved in the design of the testing system and preparing a state of the art report on the subject [Sakai and Sheikh 1989].

Recently, a number of columns were tested in that frame to develop solutions for the durable rehabilitation of columns such as those shown in Figure2. The columns and girders in the bridge structure shown in the figure had deteriorated primarily due to the corrosion of steel reinforcement which was primarily caused by chlorides from the de-icing salts. Freeze-thaw and wet-dry cycles further intensified the rate of deterioration. It was observed that spiral steel had corroded to a point that its contribution toward load carrying capacity and deformation capability had reduced to a minimal level. The bridge was built in the 1960s and the damaged condition shown in Figure 2a is before the repair was carried out in 1996. The columns varied in sizes between 920 mm and 1010 mm in diameter.

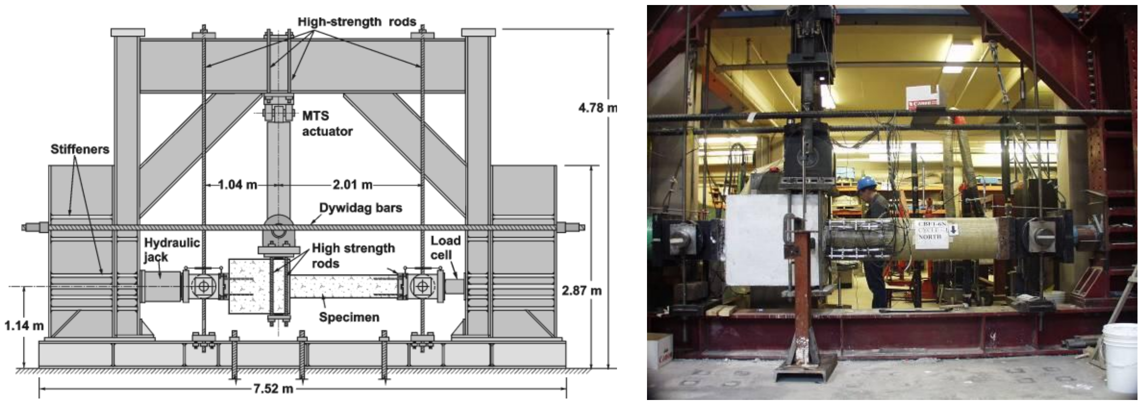

Fig. 1. Column Test Frame

a) b)

Fig. 2. Corrosion Damage in (a) Bridge Structure and (b) Lab Specimens

EXPERIMENTAL INVESTIGATION

Conventional repair techniques have been used for decades but in most cases they are cumbersome, need closing of the structure during repair and require repetition of repair within a few years. To address these drawbacks, repair with glass fibre reinforced polymer (GFRP) was chosen for application in this program. In addition to being less labour- and equipment-intensive, FRP materials are lighter, stronger and electrochemical corrosion resistant compared to steel. Two commercially available grouts, and another based on specially developed expansive cement were investigated for rebuilding the columns back to their original shape. Details on the expansive cement formulation are available elsewhere [Sheikh et al 1994].

Before the repair work was carried out in the field, an extensive research program was initiated to evaluate the expected behaviour of FRP-repaired columns and the long-term performance of FRP under severe environmental conditions. Small and large scale column specimens were tested under axial load only and also under combined axial and cyclic flexural and shear loads. A brief overview of the related experimental work is presented in the following.

Column Tests. The first series of tests included several 150 mm x 300 mm cylinders and ten 406 mm diameter and 1.370 m long columns. Each column was reinforced with 6-20M (300 mm^2) longitudinal reinforcing bars and a 10M (100 mm^2) spiral with 75 mm pitch. At the age of 34 days, eight of the columns were subjected to accelerated corrosion to produce damage similar to that observed in the field (Figure 2b). The corrosion process was activated by chloride salts and impressed current. Six columns damaged by corrosion were repaired using different retrofitting procedures. Two corroded columns were not repaired and used as control specimens along with two un-corroded columns. Details of the four columns tested under concentric compression are given in Table 1. The repair work was carried out with the aim of minimizing the costly field work. It was decided that the corroded steel and the contaminated concrete would not be removed from the damaged columns. It was also decided to investigate the use of a plastic sheet wrapped around the column as formwork for grout and a barrier between the new grout and GFRP. The barrier between GFRP and grout was considered necessary because, on the basis of simulated lab studies in which fibres alone were immersed in sodium hydroxide solution, researchers [e.g. Uomoto and Nishimura 1999] had reported that alkalis reacted adversely with glass fibres.

Table 1. Details of Test Specimens

Specimen	f_c' (MPa)	GFRP Treatment	Lateral Steel		Longitudinal Steel	
			Size @ pitch (mm)	ρ_s (%)	# of Bars-Size	ρ_g (%)
Columns under concentric compression						
Control	33.3	None	10M@75	1.5	6-20M	1.4
Damaged	33.3	None	10M@75	1.5	6-20M	1.4
Exp-repaired	33.3	2 Layers	10M@75	1.5	6-20M	1.4
Emaco-repaired	33.3	2 Layer	10M@75	1.5	6-20M	1.4
Columns under cyclic flexural and shear and constant axial loads						
S-2NT	40.1	None	US # 3@80	1.1	6-25M	3.0
S-4NT	39.2	None	US # 3@300	0.3	6-25M	3.0
ST-5NT	40.8	1 layer	US # 3@300	0.3	6-25M	3.0

One column (called Emaco-repaired) was patched with commercially available rheoplastic, shrinkage-compensated mortar called EMACO which contained propylene fibres and silica fumes. The patch was covered with a protective epoxy coating to avoid direct contact between the new mortar and GFRP. After 24 hours of curing, the column was wrapped with two layers of GFRP. Another column (called Exp-repaired) was repaired with grout based on the expansive cement [Sheikh et al 1994]. A 3 mm thick polymer sheet reinforced with polyethylene fibres was wrapped around the damaged area of the column and held in place with five hose clamps so as to act as formwork for the column repair. Next the expansive cement mortar consisting of one part cement, one part fine sand and 0.55 part water, was poured in place. Four hours after grouting, the column was wrapped with two layers of GFRP on top of the polymer sheet. The columns were stored in the lab at 23°C temperature and about 50% relative humidity for three months after repair before they were tested.

The results from short term tests on four columns are presented in Figure 3 which shows that corrosion of steel and loss of concrete cover resulted in approximately 20% loss of strength in the damaged column compared with the undamaged control column. Reduction in ductility and energy dissipating capacity was even higher. The cover concrete had almost entirely spalled off or become ineffective in all the corroded columns. Figure 4 shows four columns at the end of their tests. It can be seen from the figure that corrosion rendered the spiral steel completely ineffective in the damaged columns resulting in brittle column behaviour similar to that of an unconfined concrete column.

Both the repaired columns performed better than the damaged unrepaired column. The Exp-repaired column, in which plastic sheet formwork was used, did not display as good a performance as the Emaco-repaired column in which no formwork was used. At large strains the plastic sheet of the formwork in the Exp-repaired column opened out and caused premature failure of FRP wrap and the column as shown in Figure 4. The axial load carrying capacity of the Emaco-repaired specimen was as large as the control undamaged column. Its ductility and energy dissipation capacity exceeded those of the undamaged control column.

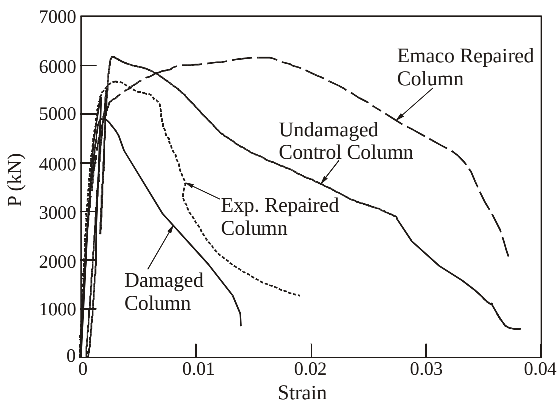

Fig. 3. Behaviour of Undamaged, Damaged and Repaired Columns

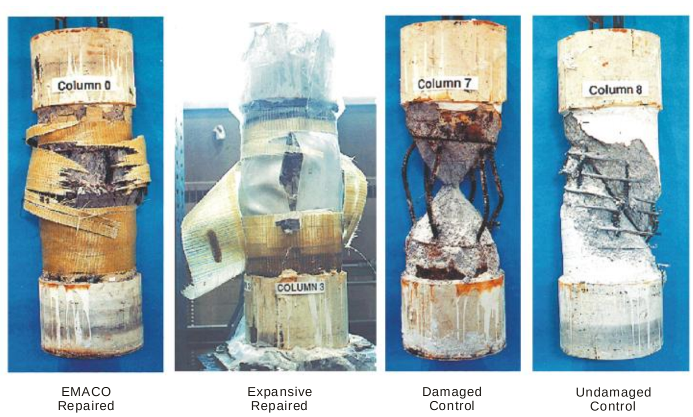

Fig. 4. Column Specimens after Testing

The specimens for the second series were designed to evaluate their performance under combined loads [Sheikh and Yau 2002]. Twelve columns were tested in the frame shown in Figure 1 under cyclic flexural and shear loads while subjected to constant axial loads. All the columns were 356 mm in diameter, 1.47 m long and cast integrally with a stub of dimensions

510 × 760 × 810 mm. Figure 5 shows the moment curvature responses of the critical sections in three specimens designated (Table 1). All the columns were tested under an axial load of 0.27 P_o where P_o represents the theoretical axial load carrying capacity of the column. Specimen S-2NT contained spiral steel at 75 mm spacing thus representing a healthy column designed in accordance with the design codes. Specimen S-4NT contained spiral steel at 300 mm spacing that represented a column with deficiency in spiral steel due either to corrosion or design. The presence of sound concrete cover in Column S-4NT would overestimate the response of a column that is damaged by corrosion and has lost the concrete cover. Column ST-5NT with spiral at 300 mm spacing was retrofitted with one layer of GFRP wrap thus representing a repaired column in which the spiral steel was not replaced after damage. A comparison of the moment-curvature responses of Specimens S-2NT and S-4NT underlines the importance of the spiral steel and its spacing. Strength, ductility, energy dissipation capacity and the number of cycles of inelastic excursions that a column can sustain depend on the effectiveness of the spiral steel and the confinement it provides. The deficiency of the column due to the reduced spiral steel can be easily overcome by the addition of only one layer of GFRP wrap as shown by the response of ST-5NT. Strength, ductility and energy dissipation capacity of the GFRP-retrofitted column ST-5NT are similar or superior to those of the control column S-4NT that meets the current code provisions.

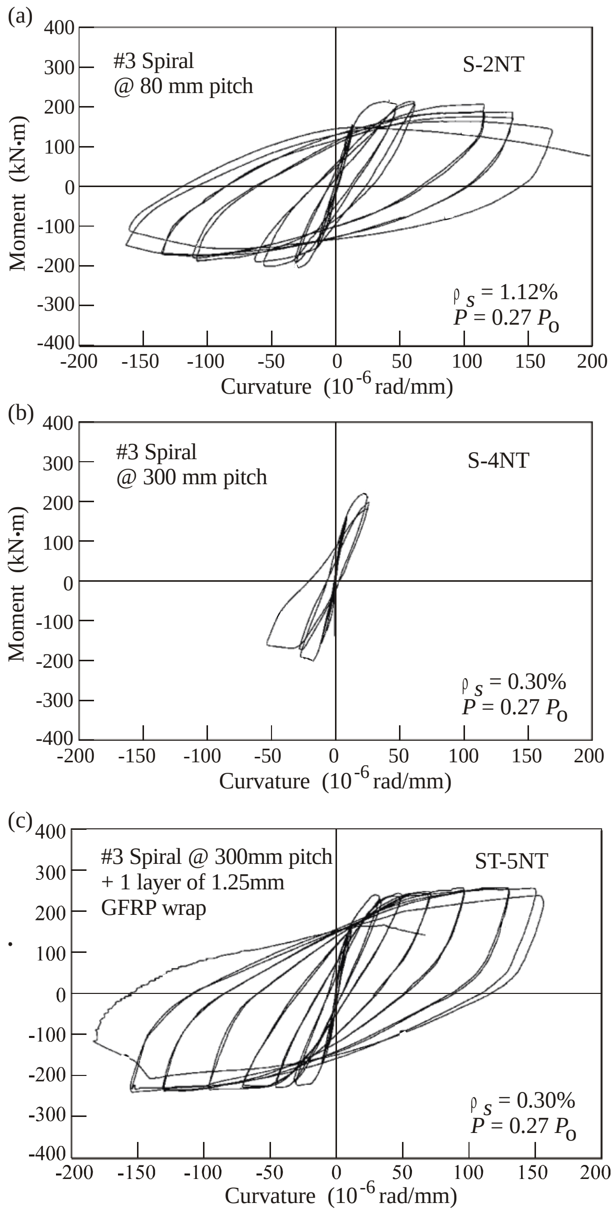

Fig. 5. Moment-Curvature Behaviour of Columns

REPAIR OF BRIDGE COLUMNS

The laboratory experiments had indicated that the level of damage in the bridge columns (Figure 2) was such that spiral steel had completely lost its effectiveness due to corrosion. It was decided not to add a new steel spiral but to compensate for it by providing a GFRP wrap. Before the repair was carried out, only the loose concrete on the surface of the bridge columns was removed with the help of a steel brush. The corroded steel and the contaminated concrete were not removed from any of the columns. The plastic sheet formwork was not used in any of the columns considering the premature failure observed in the lab specimen. Although several columns were repaired, three of the columns that used three different repair schemes were treated as field test specimens and monitored for their long-term performance.

Column 124-1 was repaired with expansive concrete grout using steel formwork (Figure 6). The grout which replaced the original 50 mm thick cover concrete and had the following mix proportions by weight: normal Portland cement 308 kg, hydrated high alumina cement 128 kg, moulding plaster 61 kg, hydrated lime 15 kg, sand 792 kg, 6 mm crushed rock 902 kg, water 217 kg and superplasticizer. About twenty hours after grouting, the formwork was removed and the column was wrapped with a thin polyethylene sheet and then two layers of GFRP with glass fibres aligned in the circumferential direction (Figure 6c). Polyethylene sheet acted as a barrier between the new concrete and GFRP. Three days after the grout application, the column was instrumented with six strain gauges in the circumferential direction installed on the GFRP. Two gauges each, 180^o apart, were installed at mid height, 750 mm above and 750 mm below the mid height of the column.

Column 124-2 was repaired using commercial non-shrink grout which was pumped in place with the steel formwork. The grout mix proportions by weight were: pre-blended sanded grout 75 kg, 6 mm crushed stone 25 kg, silica fume 3 kg, water 15 kg and superplasticizer. Four days later, the steel forms were removed and the column was wrapped in polyethylene sheet and GFRP in the same way as Column 124-1. Eight days after the application of grout, this column was also instrumented in a manner similar to that used for Column 124-1.

Column 124-3 was built to its original shape with EMACO-based mortar. No formwork was needed for this mortar (Figure 6d). A protective epoxy coating was applied six days later followed by the FRP wrapping and instrumentation similar to that in Column 124-1. The gauges were applied to this column eight days after grout application.

To monitor corrosion activity in the repaired columns three half cells were embedded, at the top, middle and bottom along their heights, in each of the three columns discussed above.

FIELD TEST DATA

Monitoring of the bridge columns has been carried out at regular intervals since the repair. This included the corrosion potential readings from half cells and the lateral strain data collection from the electric strain gauges in addition to regular visual observations. Figure 7 shows average lateral strain in FRP against time for the three columns. As expected, Column 124-1, repaired with expansive cement, showed substantial early age expansion while no significant lateral strain was measured in GFRP wrap in the other two columns. The maximum tensile strain in GFRP measured in Column 124-1 was approximately 0.16% that was observed ten days after grouting. This represented about 10% of GFRP rupture strain. Lateral GFRP strain in all columns remained fairly constant for about two years indicating stable expansive cement behaviour and no significant creep in the GFRP wrap.

(a) (b) (c) (d)

Fig. 6. Repair of Bridge Columns

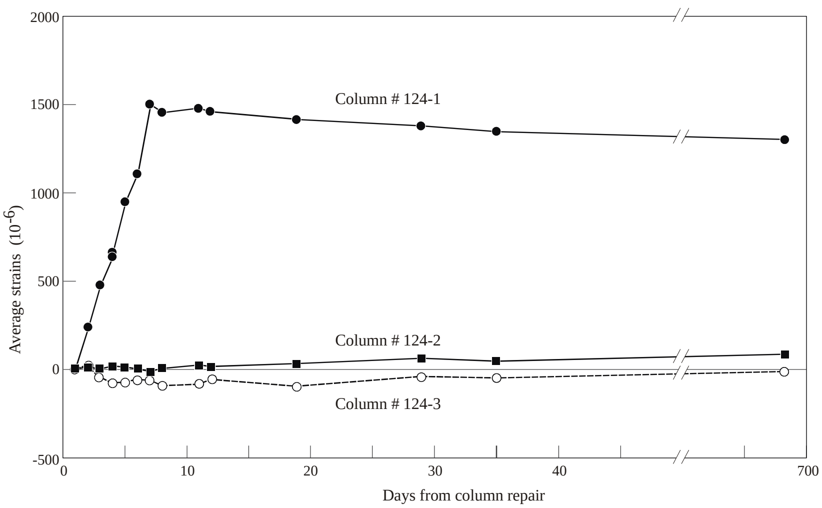

Fig. 7. Lateral Strain in GFRP in Repaired Bridge Columns

Table 2 shows the half-cell potential measurements of three repaired columns taken over a period of six years. The potentials can be used to estimate the risk and probability of corrosion activity in concrete at the time of measurement [Broomfield 1997]. If the potential over an area is less than –256mV, there is a greater than 90% probability that reinforcing steel corrosion is occurring; if it is in the range of –106 to –256mV, the risk is intermediate, but the probability of corrosion is unknown; and if it is larger than –106 mV, there is a greater than 90% probability that no reinforcing steel corrosion is occurring. Soon after the repair, based on the average of potential measurements at three locations along the height of each column, the risk of corrosion in repaired columns 124-1 and 124-2 was high, and in column 124-3 it was intermediate. After six years, the risk of corrosion in columns 124-1 and 124-2 was intermediate and low in 124-3. Reduction in the corrosion activity and risk of corrosion can be clearly seen in Figure 8 which shows the average corrosion potential in three columns at different locations along their heights. It can be concluded that GFRP wraps have protected the columns from adverse environmental effects thus starving the corrosion of essential ingredients, oxygen and water, resulting in reduced corrosion activity.

Table 2. Results of Condition Survey of Leslie Street Bridge.*

Date	Temp (°C)	Rel. Hum. (%)	Corrosion Potential (Embedded Cells, Silver/Silver Chloride)								
			Column 1			Column 2			Column 3		
			Top (mV)	Mid. (mV)	Bot. (mV)	Top (mV)	Mid. (mV)	Bot. (mV)	Top (mV)	Mid. (mV)	Bot. (mV)
19-07-96	23	79	-322	-274	-219	-259	-291	-292	-223	-211	-
18-09-96	20	NA	-280	-234	-187	-230	-231	-	-196	-178	-183
30-10-96	16	70	-266	-212	-176	-219	-209	-238	-169	-145	-149
19-06-97	20	60	-336	-240	-204	-243	-217	-237	-214	-120	-118
18-06-98	28	43	-335	-200	-182	-123	-257	-128	-140	-104	-98
11-08-00	24	46	-172	-195	-152	-72	-336	-146	-62	75	-90
14-08-02	26	44	-234	-174	67	-41	-173	-120	-170	82	-76

* From Ministry of Transportation of Ontario

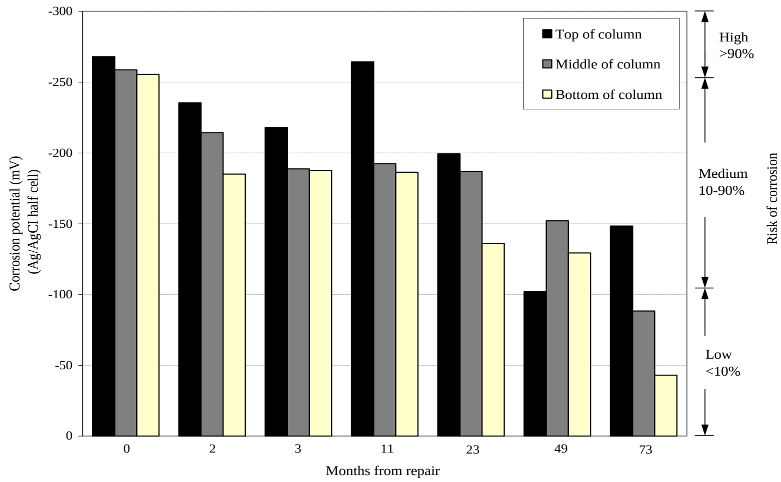

Fig. 8. Corrosion Potential and Risk in Repaired Bridge Columns

Fig. 9. Repaired Columns after Thirteen Years of Service

Visual inspection over thirteen years and field data on strain and corrosion rate indicated a sound performance of the retrofit techniques for the columns. No distress or deterioration was observed in the repaired columns after thirteen years of service as shown in Figure 9. The risk of future corrosion appears to have been reduced. There has also been no need to repeat the repair process in any of the bridge columns repaired more than thirteen years ago. Repairs using traditional techniques in similar situations would have required repeated repairs. In the repair of the columns that was carried out in mid 1990s, a thin polyethylene sheet was used as a barrier to separate new mortar or concrete from the GFRP to avoid any possible adverse effects of alkalis on the performance of GFRP in the long term. Since then, extensive tests have shown excellent long term performance of GFRP sheets under alkaline environment [Homam et al 2000; Homam et al 2001; Steckel, G. L. 1999; Hawkins et al. 1999; Mufti et al. 2005]. Later repairs using GFRP have been carried out without the use of barriers. Although the presence of the barrier is not necessary for isolating glass from alkali, the barrier might have been partially responsible for the reduced corrosion activity in the columns.

LONG-TERM PERFORMANCE OF FRP

Tensile coupons of carbon and glass FRP were subjected to various environmental conditions including freeze-thaw cycles, water exposure, temperature cycles and alkali solutions. The freeze-thaw chamber used for these tests, which cycled between $-18^{o}C$ and $+4^{o}C$, completed about 32 cycles/week. The specimens were tested after 50, 100, 200 and 300 freeze-thaw cycles. Exposure to water was conducted at room temperature. Specimens were immersed in a bath of tap water (kept at about $23^{o}C$) and retrieved and tested after 7, 14, 28, and 84 days. Alkali exposure was conducted using two concentrations of sodium hydroxide solution, pH 10 and pH 12. The FRP coupons were immersed in the alkaline solutions at room temperature and tested for strength and deformation after 7, 14, 28, and 84 days of exposure. They were rinsed with water and dried with a cloth before being tested. The specimens for temperature cycling were placed inside an environmental chamber where only the temperature, and not the relative humidity, was controlled. The temperature varied continuously between $-20^{o}C$ and $+40^{o}C$, in the following cyclical manner; one hour at $-20^{o}C$, a linear ascending ramp lasting two hours to $+40^{o}C$, one hour at $+40^{o}C$ and a linear descending ramp lasting two hours to $-20^{o}C$. Four cycles were performed each day with specimens being tested after 7, 14, 28 and 84 days of exposure. Control specimens for the tests were stored in the laboratory, which was generally maintained at about $23^{o}C$ and 40% relative humidity.

Figure 10 compares the tensile strength of exposed glass and carbon FRP specimens with their control counterparts after exposure to 50, 100, 200, and 300 cycles of freeze-thaw. The strength of CFRP and GFRP coupons was not found to be significantly affected by exposure to freeze-thaw cycles. Similar results were found for the strains at rupture and stiffness of GFRP coupons. The FRP matrix used for specimens in these tests was epoxy.

As shown in Figure 11, the strength of GFRP coupons was only slightly affected by temperature cycling. The average drop in strength after 84 days of cycling between $-20^{o}C$ and $40^{o}C$ (336 cycles) was about 7%. A drop of about 7% was also observed in the strength of the GFRP coupons after 84 days of exposure to pH 10 and pH 12 NaOH solutions at $23^{o}C$. On the other hand, the strength of GFRP coupons dropped by about 11% when immersed in a water bath for 84 days. Changes in stiffness and rupture strain were also small due to these exposures. The moisture absorption appears to produce changes at the macroscopic level that lead to the deterioration of the resin and degradation of its bond with the fibre. The effect, despite severe treatment, however, is limited as indicated by a small reduction in strength. Absorption tests on GFRP specimens submerged in tap water and pH10, pH12, and pH14 NaOH solution at $38^{o}C$ were also carried out. After 60 days, the highest weight gain was observed in the specimens submerged in tap water and the lowest weight gain was observed in specimens submerged in pH14 NaOH solution.

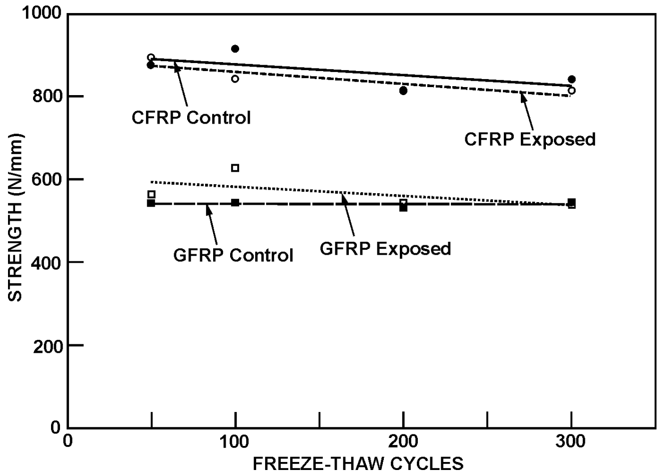

Fig. 10. Tensile Strength of FRP Coupons Exposed to Freeze Thaw Cycles

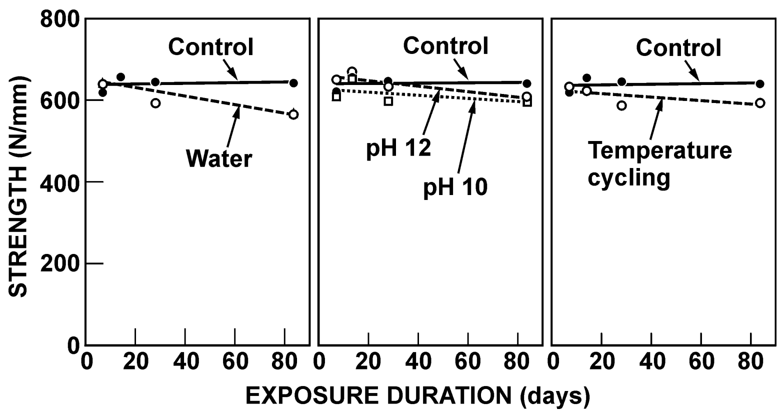

Fig. 11. Strength of GFRP Coupons Exposed to (a) Water, (b) Sodium Hydroxide Solution and (c) Temperature Cycling

CONCLUDING REMARKS

The research program on the behaviour of concrete columns under a variety of loads simulating field conditions was initiated by the author with the assistance of Dr. Koji Sakai while both were at the University of Houston. This program in later years was focused on economical and durable upgrades of existing structures. Brief details of a program are presented in this paper in which material and structural research, and laboratory and field experiments were utilized to develop a unique and innovative solution for rehabilitation of structures damaged as a result of steel corrosion. The first structure was repaired in 1996 in which new materials at the time including an especially developed high potential expansive cement and glass FRP were used. The lab tests showed that the observed field damage caused about 20% reduction in the axial load carrying capacity of the columns and much larger reductions in ductility and energy dissipating capacity. The lab tests also showed that the performance of the repaired columns with respect to strength, ductility and energy dissipation capacity was similar or superior to that of the undamaged control columns.

Three of the bridge columns repaired using different techniques were monitored for several years. Although the corroded steel and the contaminated concrete were not removed from the structure, field measurements indicated that the corrosion activity and risk of corrosion have reduced with time in the repaired columns. From the laboratory studies and field monitoring, it is concluded that the repaired bridge columns will have load-carrying capacities at least equal to those of comparable undamaged columns and significantly higher ductility and energy dissipation capacity. The FRP-repaired columns have been in service for thirteen years and have displayed excellent performance without requiring any additional repair work.

Results from tests on glass FRP coupons subjected to freeze-thaw cycles, temperature variations, alkaline environment (pH10 and pH12 of NaOH Solution) and water show that the maximum reduction in tensile strength was only 10%. This was caused by exposure of GFRP to water. This and other test data show that overall the FRP materials have very good resistance to various environmental exposures.

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