A Decade of Performance of FRP-Repaired Concrete Structures

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ABSTRACT

During the last ten years, several concrete structures were repaired or strengthened with glass and carbon fibre reinforced polymers (GFRP and CFRP). These applications varied from a concrete platform in an oil refinery to bridge columns and bridge culverts along Canada's major highways to high-rise apartment and condominium buildings and a restaurant concrete structure located on the US West coast with its columns and beams submerged in the Pacific Ocean. The repaired components included slabs, beams and walls in addition to the columns. Most of the repairs were carried out to fix the damage caused by the combined effects of weather, environmental hazards such as chemical attacks and overloads. Strengthening was carried out to increase the load carrying capacity of the structural components either to resist the increased load or to compensate for the deficiencies in design and construction.

Some of these structures were instrumented to measure deformations and the corrosion rate before and after the repair. Visual inspection was also carried out routinely over the years. All the structures have been found to behave in an excellent manner. No significant problems have been encountered in the performance of any of these structures. This paper describes brief details of repair of some of these structures and their performance and covers a period of about ten years.

INTRODUCTION

A vast inventory of damaged concrete structures around the world is in a dire need of repair for continued usage. The damage is caused by extreme environmental conditions, overload or simply as a result of routine aging. Environmental effects include exposure to chemicals such as chlorides, sulfates, acids, alkalis, etc., variations in temperature between freezing and thawing and changes in moisture contents. Corrosion of steel in reinforced concrete structures as a result of chloride ion ingress is one of the main reasons for concrete deterioration in areas where de-icing salt is used on roads. Design inadequacies, unsound construction practices, a lack of quality control combined with minimum maintenance have also been responsible for a substantial part of structural damage. Concrete structures discussed here include a platform in an oil refinery, bridge columns along Ontario highways, bridge culverts, and a high-rise apartment building.

Eight reinforced concrete columns in a platform for a pre-heater unit in an oil refinery were damaged during construction and further deteriorated due to severe environmental conditions (Fig. 1). The columns, 610 mm square in cross-section, were first repaired with non-shrink grout and then wrapped with two layers of GFRP. Corners of the columns were rounded off to 35 mm radius before the wraps were applied. The structure is being inspected and monitored regularly. The wrapped columns have been subjected to severe environmental effects that include severe exposure to chemicals typical of an oil refinery for about 10 years and have not shown any deterioration. Several bridge columns were strengthened with carbon and glass FRP in and around Greater Toronto Area. Fig. 1 also shows a few of the repaired columns along Highway 401. In addition, bridge culverts along QEW in Burlington, also shown in Fig. 1, were upgraded with CFRP to enhance their load carrying capacity.

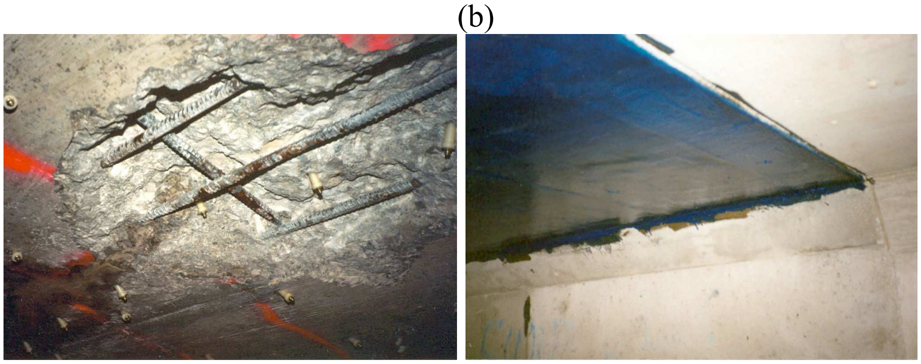
HIGH-RISE BUILDING

A high-rise building consisted of reinforced concrete slabs supported by reinforced concrete columns and walls and contained 14 floors of apartments and 4 levels of parking. At the second floor level, portions of walls and some of the columns were supported on transfer beams that extended outward to increase the width of the building at the ground floor and the parking levels (Fig. 2). The parking structure was approximately 80m long in the north-south direction and 30m wide in the east-west direction. One year after the construction was completed, cracking was observed in slabs, foundation walls, beams and columns. Some of these cracks were deemed to be structural in nature. A monitoring program was established that surveyed a total of 58 cracks for over two years.

Three years after the completion of the building, maximum crack width in extensively cracked areas of slab was 0.6 mm (Fig. 3a). At several locations cracks in the suspended slabs and walls extended the entire 80m length of the building. The maximum

(a)

(b)



(c)

Figure 1: Structures repaired with FRP: a) oil refinery, b) highway bridge columns, and c) highway bridge culverts.

width of the cracks in the walls was 1.4 mm. Of particular concern were the two horizontal cracks in each of the two north-south walls in parking levels P3 and P4. These cracks were located in the area of the maximum moment generated by the pressure from the retained soil (Fig. 3b). Initially the horizontal cracks were observed to vary from hairline to 0.1 mm in width but at about 30 months after construction, the crack widths at some locations was measured to be 0.70 mm.

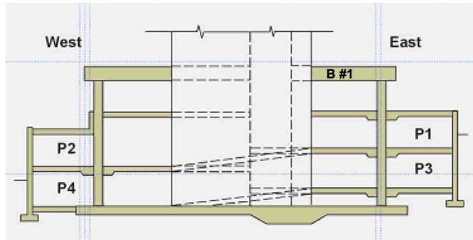


Figure 2: Cross-section of the building shown the transfer beams on eat and west sides

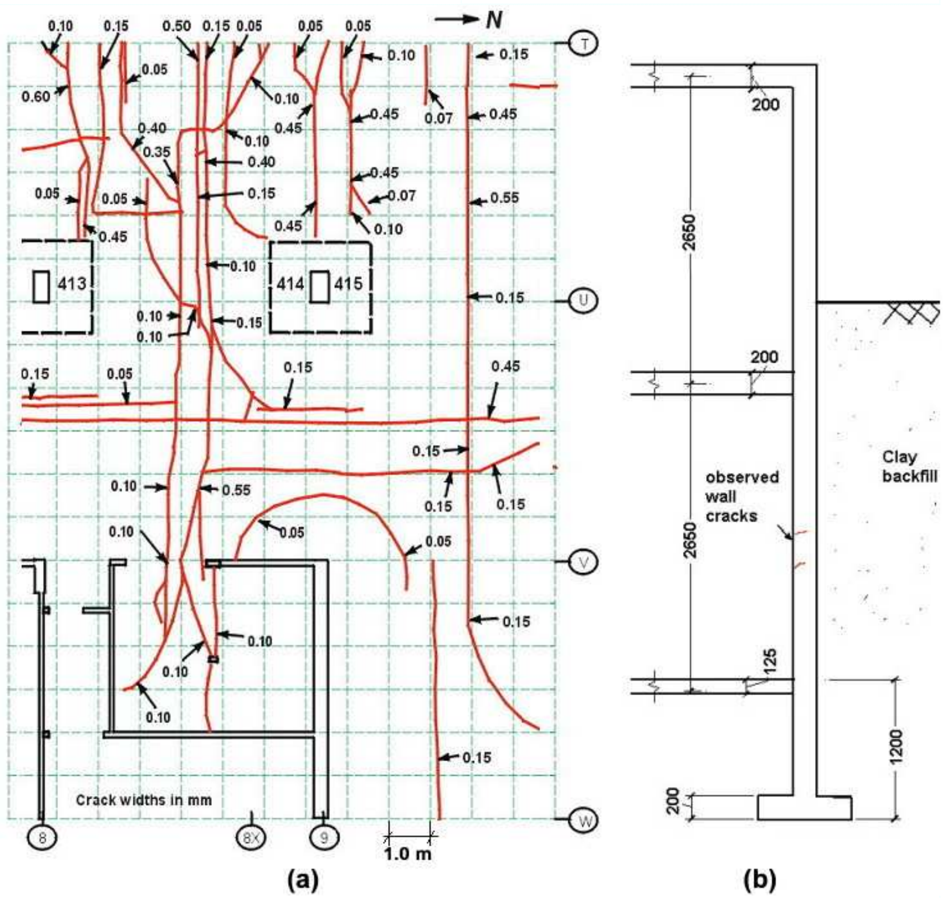


Figure 3: Observed cracks in a) slab and b) foundation wall

Several transfer beams were cracked but the beams (#1 in Fig. 2) located on two sides of the main entrance to the building showed extensive diagonal cracking (Fig.4). The maximum crack width in the beams was 0.7 mm when the building was less than four years old. Several columns supporting external ends of the beams displayed a number of horizontal cracks with width ranging up to 0.2 mm. In the tests carried out to simulate the behaviour of beams, walls and slabs, it was observed that steel crossing a 0.3 to 0.4 mm wide crack had yielded. The average tensile strain of steel was measured to be 0.0033 corresponding to a crack width of 0.4 mm in the slab tests (1).

Figure 4: Diagonal cracks in the beam

Figs. 5a and 5b show, respectively, changes in the width of a horizontal crack in a wall and a diagonal crack in a beam for about 600 days. The calculated change in the width of the wall crack as a result of temperature variation is plotted in Fig. 5c. It is obvious that the changes in crack widths in the wall and the beam do not correspond to changes in temperature indicating that these cracks are active and growing. From Fig. 5a and 5c, it can be seen that the wall crack grew by about 0.25 mm over a period of 600 days. Similar growth was also observed in beam cracks.

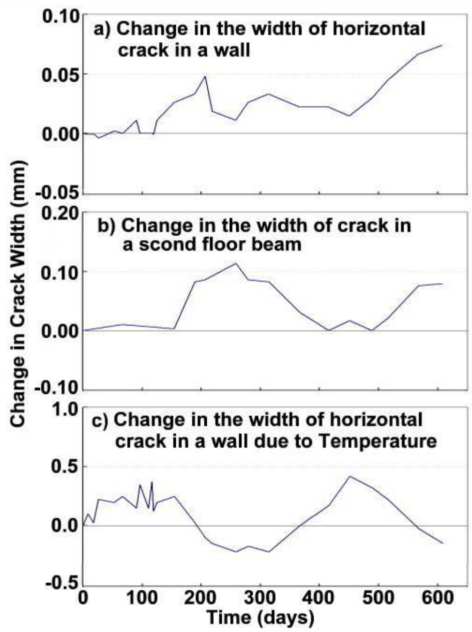


Figure 5: Crack growth wall and beam

Based on the condition survey, analysis of the building, design details and the construction record, the following were considered to be the main causes of structural distress (1,2,3): Differential settlement of the raft foundation, deficiency of shear reinforcement in beams, larger than specified cover to the main reinforcement in the foundation walls, smaller than specified thickness of walls, and clay backfill behind the walls instead of the specified well-graded granulated soil. The calculated applied moment on the walls exceeded the nominal flexural yield capacity and approached the ultimate capacity of as-built walls at several locations. The factor of safety against shear in the beams ranged between 0.45 and 1.28 while the required safety factor was approximately 1.65. In addition to providing an additional support to one of the beams, retrofitting was required to strengthen the walls and the beams in the building.

The primary factor in the selection of retrofitting technique was the minimization of interference in normal lives of the residents. Retrofitting with CFRP was selected as the preferred method. As mentioned above, full-scale tests were carried out on beams and slabs before carrying out the field applications. Sample results from two beams tests, shown in Fig. 6, clearly demonstrate the beneficial effects of retrofitting. The brittle shear failure was suppressed with one layer of one mm thick CFRP wrap around the section and the beam behaved in a very ductile flexural manner. The materials and techniques used in the laboratory tests were employed to retrofit beams and walls in the building. Fig. 7 shows a typical beam and a column just after wrapping with CFRP. Walls were retrofitted with 300 mm wide CFRP sheets at 300 mm clear spacing to act as flexural reinforcement. The repaired components have performed without any problems during about four years since repair.

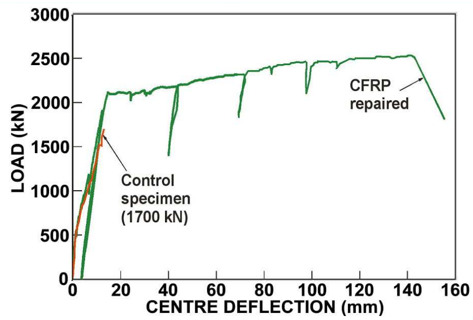


Figure 6: Load-deflection behaviour control and retrofitted beam

Figure 7: Structure retrofitted with CFRP

HIGHWAY BRIDGE COLUMNS

The second case study detailed here is related to a bridge on Highway 401 in Toronto that was damaged primarily by the chlorides from the de-icing salt (Fig. 8). The columns are about 920 mm to 1010 mm in diameter. Most of the damage in the bridge was concentrated at the bents that are directly below the expansion joints where the joint sealant and other materials deteriorated due to extreme weather conditions and the chemicals from the de-icing salts. This allowed the water-ice-salts solution to pass through the joints to the reinforced concrete bents. As a result of steel corrosion, the cover concrete in columns and girders was delaminated. The spiral steel was corroded severely but the longitudinal steel bars were relatively healthy. From the tests on half-scale models of the bridge columns, the damage similar to the one shown in Fig. 8 reduced the axial load carrying capacity by about 20%-22% (4) as shown in Fig. 9. The GFRP-repaired column was able to recapture the lost strength as well as enhance the ductility of the column exceeding the performance of the original companion specimen. The field columns were subsequently repaired using the schemes developed from the test series.

Figure 8: Damaged highway bridge

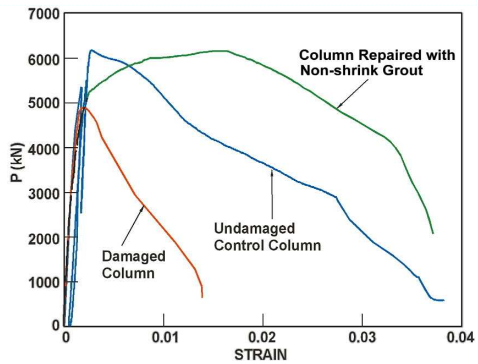


Figure 9: Behaviour of control and retrofitted columns under axial load

Repair Schemes

The entire bridge was repaired in 1996. Three columns were repaired using three different repair schemes and monitored over several years. Concrete cover had spalled off in all the columns. No attempt was made to remove the contaminated concrete from the columns. Only the loose concrete that could be removed without using any force was taken away from the columns. Fig. 1b shows steps during the repair operation.

Column 1 was repaired with expansive concrete grout using steel formwork (5,6). The expansive cement used was developed for special applications and has the setting properties similar to those of Normal Portland cement thus avoiding the flash set typical of most available expansive cements. About twenty hours after grouting, the formwork was removed and the column was wrapped first in polyethylene sheet and then with two layers of GFRP with fibres aligned in the circumferential direction. The polyethylene sheets were installed to avoid direct contact between GFRP and fresh concrete in an effort to minimize the effects of alkali on glass fibres. Two days after the FRP application, the column was instrumented with six strain gauges in the circumferential direction installed on FRP. Three sets of two gauges each were 750 mm apart vertically in the middle of the column height.

Column 2 was repaired using a non-shrink concrete grout that was pumped in place. Four days later, the steel forms were removed and the column was wrapped in polyethylene sheet and two layers of GFRP. Eight days after the application of the grout, the column was instrumented in a manner similar to that for Column 1. Column 3 was patched using a rheoplastic mortar. A protective epoxy coating was applied six days later followed by the GFRP wrapping and instrumentation similar to that in Column 1. The gauges were applied eight days after the grout application.

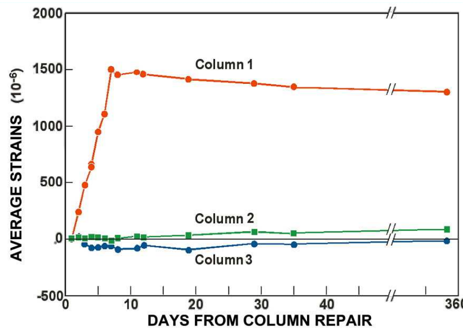


Figure 10: Lateral strain in repaired columns

Test Data

The average compressive strength of wrapped cylinders of expansive concrete grout varied between 44 MPa at eight days and 50.5 MPa at 93 days. The cylinders failed by rupture of the FRP wrap at an axial strain of approximately 0.017 at which point the cylinder was still showing positive stiffness. Comparable peak strain in unconfined concrete is in the range of 0.002 to 0.0025. The maximum weight loss of the specimens (75 x 75 x 300 mm) subjected to 300 freeze-thaw cycles was under 8%.

Lateral strain data from the field columns is shown in Fig. 10. As expected, column 1 with expansive cement cover showed substantial expansion, while no significant lateral strain was measured in FRP in the other two columns. The maximum expansion in column 1, approximately 0.16%, was observed ten days after grouting. Lateral strain in GFRP remained fairly constant at about 10% of its rupture strain for over a year when monitoring of this data was terminated indicating stable expansive cement behaviour and no significant creep of GFRP.

Table 1 shows half-cell potential measurements of three repaired columns carried out over a period of six years. The potentials can be used along with Table 2 (7) to find the probability of corrosion activity in concrete at the time of measurement. If the potential over an area is more negative 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 less negative than –106 mV, there is a greater than 90% probability that no reinforcing steel corrosion is occurring.

Table 1: Results of condition survey of Leslie Street bridge

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

In 1996, based on the average of potential measurements at three locations along the height of each column, the risk of corrosion in repaired columns 1 and 2 was high, and in columns 3 it was intermediate. In 2002, the risk of corrosion in columns 1 and 2 was intermediate and in column 3 it was low.

Table 2: Ag/AgCl potential for determination of probability of corrosion

Measured potential	**Risk of corrosion**
<-256 mV	High (90%)
-106 mV to –256 mV	Intermediate
> -106 mV	Low (10%)

Visual inspection over ten 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 these columns after ten years of service, and the risk of future corrosion has been reduced.

CONCLUDING REMARKS

Monitoring plays an important role in maintaining the health of structures and providing help when indicated by the data collected. This paper presents case studies in which FRP retrofitting was carried out for repair, strengthening and life extension of a variety of structures. These structures included bridges, apartment buildings, and refinery structures. In most cases, structures were monitored for the extent of damage and model structures were tested in a laboratory to determine the most effective repair techniques. The non-traditional materials used in the repair included high potential expansive cement and glass and carbon fibre reinforced polymers. While all the repaired structures were inspected regularly, some were monitored for years. Based on the laboratory studies, it is concluded that the repaired structures will have significantly better performance compared with the original structures with respect to ductility and energy dissipation.

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REFERENCES

1. DeRose, David and Sheikh, Shamim A., (1997), “The rehabilitation of a concrete structure using fibre reinforced plastics”, Research Report, Department of Civil Engineering, University of Toronto, 177 p.

2. Sheikh, Shamim, A., (1998, 2000), “Evaluation of structural safety of the AAP apartment building”, Reports submitted to AAP Management.

3. Collins, M. P., and Bentz, E., (1997), “An investigation into the structural safety of the AAP apartment building”, Reports submitted to AAP Management.

4. Sheikh, S.A., Pantazopoulou, S.J., Bonacci, J.F., Thomas, M.D.A.T., and Hearn, N., (1996). "Repair of delaminated circular pier columns by ACM". Research Report submitted to the Ministry of Transportation of Ontario.

5. Sheikh, S.A., Fu, Y. and O'Neill, M.W., (1994), "Expansive cement concrete for drilled shafts", ACI Materials Journal, 91(3), pp 237-245.

6. Sheikh, S.A., Agarwal, A.C., Pinaca, F., (1997), “Repair of the Leslie Street Bridge Columns Using Glass Fibre Wraps”, Six International Colloquium on Concrete in Developing Countries, Jan 4-6, Lahore, Pakistan, Ed. S.A. Sheikh and S.A. Rizwan, pp 379-388

7. Broomfield, J.P., (1997), “Corrosion of Steel in Concrete,” E&F Spon, London, UK, p. 45