TECHNICAL REPORT

UNIVERSITY OF MANITOBA

INNOVATIVE DESIGN FOR MANURE STORAGE FACILITIES

Final Report

June 2002

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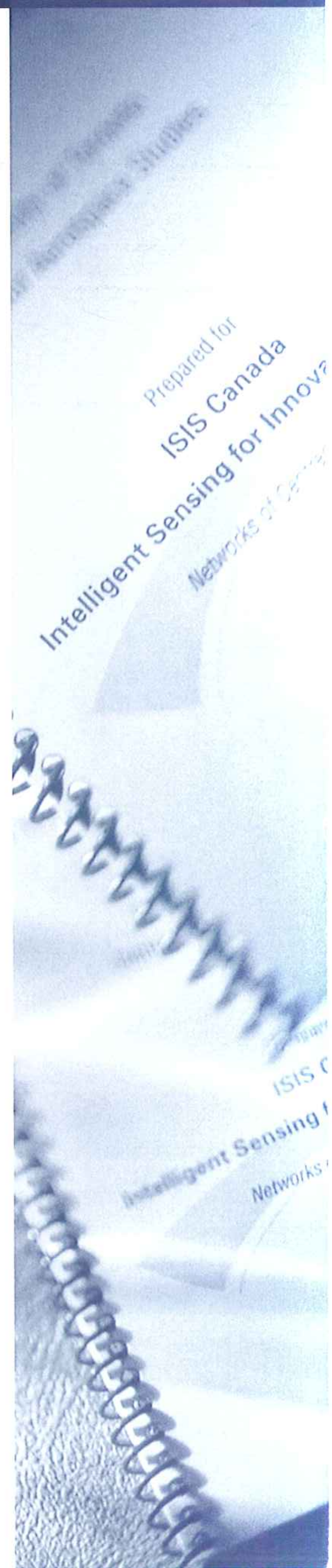

ISIS CANADA RESEARCH NETWORK
The Canadian Network of Centres of Excellence on Intelligent Sensing for Innovative Structures
Le réseau canadien de Centres d'excellence sur les innovations en structures avec système de détection integré

EXECUTIVE SUMMARY

Environmental concerns about the integrity of manure storage facilities have been raised in many regions across the country. Often concerns regarding potential leakage have motivated the public to resist the development of large-scale agricultural facilities. A recent survey performed by the Ministry of Agriculture, Fisheries and Food in Quebec reported that of the 28 concrete tanks that have been inspected, 23 tanks show serious deterioration of the walls (i.e., vertical and horizontal cracks).

Hog manure storage is a major constraint on confined animal production systems. Expanding levels of production, particularly in the hog industry, are making the problem more severe. In the past, liquid manure has commonly been stored in earth lagoons. Increasing environmental concerns related to ground seepage have led to the requirement that more impervious structures be designed and built. This has augmented the use of steel-reinforced concrete for hog-manure storage tanks above ground. The challenge of reinforced concrete is its long-term stability, which controls the so-called durability or service life of concrete structures. Due to the hostile service environment associated with manure storage, corrosion rates of the steel reinforcement are potentially high. These deterioration rates could lead to a significant mechanical weakening and finally to a relatively short service life of the tanks.

To map out the various mechanisms through which the strength of conventional reinforced concrete currently used for manure storage tanks may be improved, thereby to increase the safe service life of the tanks, this research program, entitled the Innovative Design for Manure Storage Facilities Project, has been initiated by the Canadian Network of Centres of Excellence on Intelligent Sensing for Innovative Structure (ISIS Canada). This project was developed in collaboration with Agri-Food Research and Development Initiative (ARDI), the Manitoba Triple S Hog Manure Management Initiative and Manitoba Livestock Manure Management Initiative(MLMMI).

The Project started in June 1999 and ended in May 2002. During this period six Technical Progress Reports were prepared and submitted to the funding organizations. These reports documented the work performed and presented the preliminary results and findings. The work performed focused on the following five major groups of activities:

1) Review the published literature relevant to the project;
2) Perform trial tests to determine the test specimen design, the test set-up and the mode of failure of reinforced concrete beams;
3) Design the experimental program to examine the suitability of GFRP's for hog waste storage facilities;
4) Cast reinforced concrete structural elements (concrete / reinforcement combinations) required for the experimental program and spray concrete specimens with GFRP;
5) Prepare and test sets of specimens that have been in contact with manure for 0 (control), 4, 8, 12 and 18 months and carry out physical and chemical examination of selected samples from the specimens kept in contact with manure for various length of time.

This is the Final Technical Report on the Innovative Design for Manure Storage Facilities Project. In this report, each of the five above-mentioned groups of activities is presented in a separate chapter followed by the Conclusion chapter. Here, these activities are briefly presented.

The primary objective of the research in this project was to investigate the suitability of innovative design procedures for reinforced concrete manure storage tanks using an advanced composite material (FRP) as the internal or external reinforcing element.

The literature reviewed indicated that there has been no serious technical evaluation of the behavior of reinforced concrete used in hog-manure storage tanks currently in place. The studies previously performed on reinforced concrete used in agricultural structures are limited both in scope and in the environmental conditions.

Concrete is traditionally reinforced with steel bars. It is well known that deterioration of concrete structures could in most cases be attributed to corrosion of the reinforcing steel.

The advanced composite materials, FRP's, have an outstanding strength/weight ratio and a high degree of chemical inertness to most civil engineering environments, strongly suggesting their consideration as reinforcement for concrete manure storage tanks.

Trial tests have been carried out to determine the test specimen design and the mechanical test set-up required to investigate the behavior of different designs of structural element. The experimental work performed in this trial led to the selection of the reinforced concrete beam design, testing mode and the test set-up used throughout the project. From the observations of the results and the conditions considered in this study the beams have been designed to ensure that the only mode of failure of the reinforced concrete beam is flexural failure, the rupture of the reinforcement. The geometry of the beam has been established at a cross-section of 170 mm x 170 mm and a length of 1000 mm with a reinforcement ratio, $\rho_{GF} = 0.27$ %. The experimental work also permitted the selection of the testing mode and test set-up providing an excellent solution for eliminating problems such as handling and cost associated with casting very large and heavy reinforced concrete beams.

The concrete specimens required in the experiment were manufactured at Lafarge Canada Inc. Construction Materials Group, Precast Division.

With regard to reinforcement materials, attention was focused on steel rebar (the reinforcement currently used in all designs for manure tanks) and on three types of glass fiber reinforcement polymers that are the least expensive and have a great potential to improve significantly the service life of these tanks. The GFRP materials under investigation were the GFRP C-BAR, GFRP ISOROD and GFRP spray composite.

Test beams have been designed according to Canadian Code standards. They were designed to ensure that the only mode of failure of the reinforced concrete beam is flexural failure with the rupture of the reinforcement. Three different cover schemes were implemented: 1) concrete beams not covered with any protective materials to simulate real-life conditions where the reinforced concrete is in direct contact with manure; 2) beams sprayed on the four vertical sides and the bottom face of the beam with GFRP; and 3) beams covered on the four vertical sides and the bottom face of the reinforced concrete beam with PVC plates.

In order to investigate the effects on long-term performance of composite materials, the experimental program considered methodologies (test conditions) to accelerate the degradation phenomena in reinforced concrete. Acceleration in the experiment has been achieved by using elevated temperature and exposing the structural elements to wet/dry cycles. In addition the manure in contact with the reinforced concrete structural elements was changed every two weeks to prevent a rise in pH and the increased concentration of concrete decay products, maintaining the relatively high reactivity of the manure with respect to reinforced concrete.

Three variables were considered in the experiment: 1) reinforcement types; 2) type of protective isolation of reinforced concrete; and 3) duration of exposure of the reinforced concrete to manure.

Fifty-two reinforcement bars and 62 reinforced concrete beams were tested after they had been exposed to a manure environment for 0 (control), 4, 8, 12 and 18 months to determine the changes in their mechanical properties with exposure time. In addition, systematic microstructure analyses of the reinforced concrete were made to examine the results of chemical attack and physical degradation due to the environmental exposure conditions; factors controlling the initial microstructure development are strongly related to reinforced-concrete durability. Frequent quantitative analyses of the manure were also made to identify changes resulting from contact with the concrete structural elements. Over the duration of the experiment, the experimental conditions were precisely controlled. The pH was measured and the changes in the chemical composition of the manure were quantitatively analyzed. Attention was paid to chemical species that could affect the performance of the structural elements, such as: CO_3^{2-}, HCO^{3-}, Cl^-, Mg^{2+} Ca^{2+}, Si^{4+}, K^+ and Na^+. Particular attention was given to Ca^{2+} and Si^{4+}. These elements were chosen because they reflect the leaching characteristics of two major phases in cement paste: 1) calcium hydroxide [$Ca(OH)_2$]; and 2) calcium silicate hydrates (C-H-S). Analysis results indicated that the pH in as-received manure ranged between 7.3 and 8.5 and that no significant changes in pH took place after two weeks in contact with the concrete structural elements; the pH of the manure ranged from 7.3 to 8. The data suggest that changing the manure regularly prevented the accumulation in the manure of species leached out from the concrete. In these experimental conditions, saturation with respect to the major concrete components such as Ca^{2+} and Si^{4+} was not achieved and the pH remained close to its initial value.

Three types of reinforcement bar have been investigated: (1) steel; (2) GFRP ISOROD; and (3) GFRP C-BAR. Fifty-two reinforcement bars were tested in axial tension after exposure to a manure environment for 0 (control), 4, 8, 12 and 18 months. Changes in the mechanical properties such as: yield strength, ultimate strength, and moduli of elasticity of the reinforcement bars due to exposure were investigated. The analysis of the results is discussed in terms of stress-strain behavior for each type of reinforcement bar.

The experimental results indicate that both the yield strength and ultimate strength of steel reinforcement bars in contact with manure decreased continuously with exposure time. The yield strength after 12 months exposure decreased by about 18% compared with control specimens. The yield strength decreased at an accelerated rate during the second four-month period, about 9%. Starting with 12 months exposure, some or all of the bars failed before reaching the yield strain: three out of 4 bars exposed to manure for 12 months and all specimens exposed to manure for 18 months failed before reaching the yield strain. A significant reduction was also observed in ultimate tensile strength of the steel bars. The average ultimate tensile strength of bars in contact with manure for 18 months was 373 MPa, a decrease of 252 Mpa (about 40 %) compared with the control specimens. It appears that the corrosion rate was high at all times, but in particular during the 4-8 months exposure period. The observed decreases in yield strength and ultimate tensile strength of the steel bars are due to advanced corrosion (i.e. localized corrosion and/or general corrosion). This led to a substantial decrease in diameter of the steel bars in various places and consequently to a significant mechanical weakening. Corrosion of steel reinforcement bars can be initiated and maintained in a manure storage tank under two broad sets of conditions; 1) high pH conditions in the presence of chloride ions; or 2) low pH conditions in the absence of chloride ions. In either condition, iron, oxygen and water must be present in order for corrosion to occur. Although the presence of water is necessary, excess water can limit corrosion severely (insufficient oxygen present). However, the tank wall is exposed cyclically to wet/dry conditions and significant localized corrosion can occur. Once the localized corrosion starts, the tank wall could be damaged in a short period.

A decrease in both modulus of elasticity and the ultimate tensile strength was observed during each exposure interval for the GFRP bars (ISOROD and C-BAR bars). On average, the modulus of elasticity and ultimate tensile strength of the ISOROD bars after 18 months exposure to manure decreased by about 17 % and about 19 %, respectively, compared with the control specimens. It appears that decreases in elastic modulus took place mainly during the first four months of the experiment (0 - 4 months). Very little change in modulus of elasticity took place in the second and third period (4 - 8 months and 8 - 12 months). In the last six months of exposure to manure the ultimate tensile strength decreased by only 3%. For C-BAR a decrease in both modulus of elasticity and ultimate tensile strength was observed after 18 months exposure to the manure environment although the ultimate strength changed very little in the last 10 months. The average decrease in modulus of elasticity after 18 months exposure was 17%. The average decrease of ultimate strength at 18 months exposure was 18 %. However, most of the decreases in ultimate tensile strength took place during the first 8 months of exposure, a decrease by about 16%, compared with the values for the control specimens. In the last six months of the experiment, no decrease took place in ultimate tensile strength. The results for the GFRP C-BAR reinforcement bars follow a similar trend as the GFRP ISOROD bar results. However, the values for the ultimate strength of the C-BAR reinforcement were much higher (average control 862 MPa and after 18 months in contact with manure about 760 MPa) than for the ISOROD (average control 612 MPa and after 18 months in contact with manure about 498 MPa). The observed decreases in the ultimate tensile strength and the modulus of elasticity with time of exposure to a manure environment for both ISOROD and C-BAR reinforcement bars may be attributed to absorption of moisture by the polymer; the diffused moisture weakened the glass fiber/polymer interfacial bond strength. The mode of failure of the exposed bars supports this supposition; The failure behavior of both GFRP's bars was characterized by surface fiber breakage either at the center or close to the lower grips. Microscopic examination carried out to evaluate the morphological changes and the degree of degradation due to interaction with the manure environment on the randomly selected GFRP specimens exposed for 12 and 18 months revealed no visible changes in those exposed for 12 months. Examination of the 18 months-exposed specimens suggests some GFRP degradation only on the GFRP C-BAR bar specimens. However, the degradation area was confined only to the lugs (protrusions) which are to prevent longitudinal movement of the bars relative to the concrete surrounding the bar. This may be related to the imperfections / physical damage of the coating polymer in this area during handling. It should be noted that the imperfections /damage were not due to chemical degradation. The main mechanism of glass degradation is a dissolution process. Glass dissolution was not observed. Nonetheless, even when glass dissolution will start in a manure storage tank, conditions are such that the dissolution rate should be very low. The conditions expected at the tank wall are: exposed surface area of glass to dissolved cations and anions in the liquid phase (flow) low; dissolved silica concentration in solution relatively high; and temperature relatively low. Although dissolution is the main degradation mechanism of glass, stress cracking may have some impact in a real-life storage tank. In this case, the damage caused by stress and dissolution acting together may exceed that produced when they act separately.

Sixty-two reinforced concrete beams were tested after they had been exposed to a manure environment for 0 (control), 4, 8, 12 and 18 months. Four series (A, B, C and D) of reinforced beams consisting of different concrete / reinforcement / covering material combinations were tested at every time interval. The four series were: A) Ordinary Concrete (OC) / Steel rebar; B) Ordinary Concrete / GFRP ISOROD rebar; C) GFRP spray / Ordinary Concrete / Steel rebar; and D) PVC / Ordinary Concrete / Steel rebar.

In series A and B the concrete was in direct contact with the manure, simulating conditions in the manure tank designs commonly in use. In series C and D the GFRP spray composite and PVC were used as cover materials, to prevent exposure of the concrete to the manure.

Analysis of the test results for the beams is discussed in terms of load-deflection behavior, failure mode, and cracking pattern.

For the steel-reinforced concrete beams (Series A) the yielding of the steel reinforcement in the control specimens started at a load level of 2 kN. Flexural cracking was initiated at a load level of 3.16 kN for the control specimen (12 months). A slight increase in cracking load in the first 12 months of exposure was observed; the average cracking loads were: 3.0 kN after 4 months, 3.9 kN after 8 months, 3.5 kN after12 months and 3.0 kN after 18 months exposure. In most cases, this increase was larger in the specimens exposed to manure for 4 and 8 months than for 12 and 18 months exposure specimens. This increase was attributed to the increase in compressive strength of the concrete and in its tensile strength. This may reflect changes in the strength of the concrete due to changes in its microstructure as the result of continued hydration.

Mercury intrusion porosimetry (MIP) analysis on selected concrete specimens indicated that microstructural characteristics (i.e., pore volume, pore radius and pore size distribution) change with exposure time. The pore structure of the reinforced concrete governs to a large extent two of the most important engineering properties of the hardened concrete: 1) mechanical strength; and 2) permeability. The total pore volume and pore diameter decreased during the 4 and 8 months exposure and slightly increased during the10 months exposure. The observed changes were attributed to the continued hydration and therefore progressive densification of the concrete structure as manure progressively penetrated the specimen. Furthermore, microscopic examination of the concrete surface exposed to manure revealed that a distinctive feature of the reinforced concrete/manure interaction was the formation of a surface precipitate layer consisting mainly of Ca phases (i.e., portlandite, calcite), ettringite and new calcium silicate hydrate (CHS) within the first months of exposure. The chemistry of precipitates formed early on is controlled by the concrete and at the later stage, the precipitate composition is controlled by the chemistry of the manure.

The experimental results on concrete beams reinforced with ISOROD bar (Series B) suggest a small, insignificant, increase in the flexural cracking load with exposure time. At 18 months, the beams showed linear behavior up to the first crack at a load level that ranged between 3.0 kN and 3.1 kN. A slight decrease in the ultimate load was observed after 18 months exposure. However, this decrease also appears insignificant. The control specimens failed at an average load of 4.1 kN whereas the average ultimate load for beams exposed for 4, 8, 12, an 18 months to manure were 3.7, 5.3, 3.7 and 3.7 kN, respectively. The observed changes in strength characteristics were attributed to changes in concrete microstructure. Overall, no significant changes have been observed in the mechanical properties with exposure time for the concrete beams reinforced with ISOROD bar.

One of the reasons for considering replacing the steel with GFRP for concrete reinforcement is that steel corrodes. However, use of the GFRP has its own problems; the highly alkaline environment of concrete pore water may create a problem for GFRP reinforcement. The alkaline solution may produce embrittlement of the matrix and damage at the fiber-polymer interface. The chemical processes involved during the curing of concrete create an environment (i.e., high pH) in which GFRP reinforcements are vulnerable to chemical attack. The increase in pH of the concrete is due to cement hydration. When combined with water, the hydration reactions produce calcium silicate hydrates (CSH) and calcium hydroxides ($Ca(OH)_2$). The free lime as well as the alkali oxides react with water to increase the pH to values between 11.5 and 13.7. However, the final pH can be controlled to some extent by concrete mix design, the type of cement used, and the addition of pozzolanic materials (i.e., silica fume, fly ash). Furthermore, the pH in concrete decreases with maturation and carbonation. Greater resistance to the high pH can conceivably be achieved by using either an alkali-resistant polymer or alkali-resistant fibers, or a combination of both in the manufacture of GFRP.

The results produced by the present studies suggest that GFRP reinforcement bars exhibited a high resistance to degradation in a manure environment. Consequently, penetration of the manure into the concrete through diffusion or through the inherent cracks that occur in concrete will have little influence on the degradation of the reinforcement. Furthermore, the manure - pore water exchange that will take place during tank operation will not sustain the high alkalinity in concrete over a long period of time.

A slight decrease in the ultimate failure load with exposure time was observed in the concrete reinforced with steel bar and sprayed with GFRP (Series C). The average ultimate failure loads for control specimens was 11 kN. The average ultimate failure loads at 4, 8, 12 and 18 months exposure were 9.6, 9.9, 10.4 and 9.9 kN, respectively. However, it should be noted that the ultimate failure load of the reinforced concrete beams sprayed with GFRP was about three times higher than the ultimate failure load of plain (uncovered) beams reinforced with steel (Series A) and about two times higher than the ultimate failure load for beams reinforced with ISOROD (Series B). In addition to the increase of the beam strength, the sprayed GFRP acted as a secondary protection layer for the reinforcement; the concrete cover of the reinforcement is well protected from contact with manure.

There were no notable differences in the load-deflection behavior of the steel-reinforced concrete beams and covered with PVC panels between the control beam and those exposed for 18 months. The control specimen failed at an average load of 4.0 kN whereas the average ultimate load value for the exposed beam for 4, 8, 12 and 18 months were 4.3, 3.8, 4.1 and 4.2 kN, respectively. Longer exposure time (18 months) to the manure environment did not appear to further affect the ultimate failure load or the cracking load. The steel-reinforced beams covered with PVC panels (series D) have been found to have a higher load carrying capacity than the steel-reinforced beams without cover (series A). The increase in ultimate load of steel-reinforced beams covered with PVC panels are attributed to the special profile (strip) embedded in the concrete during casting. The experimental results suggested that the PVC panel strip when well embedded in the concrete acts as an additional reinforcement for the concrete. The average cracking load values at 0 (control), 4, 8, 12 and 18 months were similar, i.e., 3.0, 3.0, 2.6, 3.0 and 3.4 kN, respectively. Not much change in the concrete strength due to curing was observed. The concrete being confined by the PVC panels prevents the moisture necessary for continued hydration to come in contact with the concrete. In general, the ultimate failure load of the steel-reinforced beams covered with PVC was higher than those for the bare steel-reinforced beams and lower than for the steel-reinforced beams sprayed with GFRP.

In conclusion, the relatively low pH (7.3 - 8.5) of the manure and the presence of chloride, (500 – 800 mgL^{-1}) destroyed the protective iron oxide film, which makes the steel resistant to corrosion. The uniform corrosion (general corrosion) initiated after the first months exposure becomes localized corrosion (pitting corrosion) thereafter. Localized corrosion is characterized by much more rapid corrosion rates compared with general corrosion. The GFRP reinforcement bars exhibited a high resistance to degradation in a manure environment. Consequently, penetration of the manure into the concrete through diffusion or through the inherent cracks that occur in concrete will have little influence on the degradation of the GFRP reinforcement bars. Furthermore, the manure-pore water exchange that will take place during tank operation will not sustain the high alkalinity in concrete over a long period. Reinforced concrete beams sprayed with GFRP performed the best among structural elements considered in the study. In addition to the increases of the beam strength, the sprayed GFRP acted as a secondary protection layer for the reinforcement. The use of GFRP materials as reinforcement (internal or external) in the construction of manure storage tanks is worthy of further consideration.

In addition to the Technical Reports, the work performed in this study has been also documented and the preliminary results and findings presented at several conferences as follows:

- A paper entitled "Innovative design for manure storage facilities" by M. Onofrei, A.A. Mufti, M.G. Britton and A. Bogdanovic, that describes the research program of this study and the results for 12 months of experiments was presented at the 30th Annual Conference of the Canadian Society for Civil Engineering, 2nd Material Specialty Conference, Montreal, Quebec, June 5-8, 2002.

- An oral presentation of the program and preliminary results of this study was made at the ISIS Annual Conference, Edmonton, Alberta, May 2, 2001.

- A poster presentation describing the objective and the preliminary results of this study was made at the "Livestock Options for the Future" conference in Winnipeg, Manitoba, June 25-27, 2001.

- An oral presentation of the results was made at the ISIS Canada's International Collaboration Workshop, Winnipeg, Manitoba, October15-16, 2001.

- A poster presentation on the objective and the research program of this study was made at the "Heart of the Continent, Farm & Food Days" Exhibition in Winnipeg, Manitoba, September 21 to 23, 2000.

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1 INTRODUCTION

Environmental concerns about the integrity of manure storage facilities have been raised in many regions across the country. Often concerns regarding potential leakage have motivated the public to resist the development of large-scale agricultural facilities. Some communities and municipalities have banned or temporarily prohibited the construction of large manure storage structures. Consequently, research that will assist in ensuring the construction of safe manure storage tanks is urgently needed.

In the present climate of increasing commercial competition and pressure for efficiency in agriculture, extension of the service life of manure storage tanks is an important system optimization strategy. In recent years, non-corrosive Fiber Reinforced Polymers (FRP) have received much attention as a reinforcement alternative to steel in the construction industry. The FRP materials, as reinforcements in concrete, have the potential to increase the longevity of manure storage structures, leading to substantial savings in life cycle cost for concrete structures.

Accurate estimation of service life of any concrete structure is a difficult task and requires knowledge of material behavior, damage tolerance, performance goals and above all, knowledge of the degradation mechanisms (fundamentally physico-chemical processes) of reinforced concrete. To maps out the various mechanisms through which the strength of conventional reinforced concrete currently used for manure storage tanks may be improve, thereby to increase the safe service life of the tanks, a research program entitle Innovative Design for Manure Storage Facilities Project was initiated by the Canadian Network of Centres of Excellence on Intelligent Sensing for Innovative Structure (ISIS Canada). This project was developed in collaboration with Agri-Food Research and Development Initiative (ARDI), Manitoba Triple S Hog Manure Initiative and Manitoba Livestock Manure Management Initiative (MLMMI).

The Project started in June 1999 and ended in May 2002. During this period six Technical Progress Reports were prepared and submitted to the funding organizations. These reports documented the work performed and presented the preliminary results and findings. The work performed focused on the following five major groups of activities:

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This is the Final Technical Report on Innovative Design for Manure Storage Facilities Project. In this report, each of the five above-mentioned groups of activities is presented in separate chapter followed by the Conclusion chapter.

2 OBJECTIVE AND SCOPE

The primary objective of the research in this project was to investigate the suitability of innovative design procedures for reinforced concrete manure storage tanks using advanced composite materials (FRP) as the internal or external reinforcing element. The test specimens were subjected to accelerated aging procedure to:

- Determine the degree of protection of the GFRP-spray as external cover material to protect the concrete in manure storage tank construction.
- Determine the degree of protection of a PVC cover as external material to protect the concrete in manure storage tank construction.
- Determine the performance of different structural elements in manure environment using an accelerated test procedure.
- Determine the type of composite materials that are capable of withstanding a manure storage environment.

This study introduced and developed the use of new advanced composite materials for the design of new concrete technology in manure storage. This investigation provides the foundation for the selection of an optimum composite material to be used to design and build more economical and environmentally benign manure storage facilities.

3 BACKGROWND

Concrete is being increasingly used for agriculture structures because, in comparison with other materials of similar or higher strength and/or impermeability characteristics, it is easier to manufacture and lower in cost. Reinforced concrete is commonly used for flooring animal houses and for forage conservation in silo tanks. During recent years, there has been increasing environmental consciousness regarding use of earthen lagoons for manure storage; this has augmented the use of reinforced concrete for hog manure storage.

The challenge of reinforced concrete is, in all these cases, its long-term stability that controls the durability or the service life of the structure. Any improvement in the service life of a concrete structure will secure safer operation with fewer possible injuries to the animals, and commercially, will permit producers to amortize their capital cost over a longer period, reducing their cash flow.

Because no significant deterioration of existing newly built concrete tanks for hog manure has yet been observed, the industry believes that there are no problems with the use of reinforced concrete for hog manure tanks. It is true that early in the life of a concrete structure, it is difficult to identify the degradation effects. However, degradation processes are generally present even when no apparent damage can be recognized. Loss of durability of reinforced concrete results from numerous external and internal causes. External causes can be physical (stresses, particularly tensile stresses), chemical (a series of chemicals such as acids, salts, alkalis, etc) or mechanical (stress corrosion). Internal causes are associated with the permeability of concrete, the differential thermal expansion of components and probably most important, the electrochemical corrosion reactions of steel.

In a recent publication by Agricultural Technical Information Services (AgTIS), based on a survey performed by the Ministry of Agriculture, Fisheries and Food in Quebec, it was reported that of the 28 concrete tanks that have been inspected, 23 tanks show serious deterioration (i.e., vertical and horizontal cracks) of the walls (AgTIS, 2002).

An other survey of the degradation of concrete floors in pig houses in Belgium, for example, indicated that even concrete slats cast by specialized manufactures showed degradation within a few years (De Belie, 1997). The percentage of farmers that noticed concrete wear in relation of the concrete slats and solid floors, in the above-mentioned study, is presented in Table 3.1.

Table 3.1. Percentage of farms with concrete degradation and slat replacement related to floor age (after De Belie, 1997).

Floor age (years)	Slat Degradation (%)	Solid floor Degradation (%)	Slat Replacement (%)
2	15	-	-
5	40	16	1
15	87	59	16
25	-	-	49

Significant degradation of concrete has been noticed by five years of service: 40% slat degradation and 16% solid floor degradation. The degradation of both slat and solid floor was quite advanced, 87% and 59%, respectively, after relatively short age, i.e., 15 years. After only 25 years almost 50% of the installed slats required replacement.

In conventional reinforced concrete construction, a significant segment of the cross-section of a beam is used solely for location and protecting the steel reinforcement against corrosion. However, this protection diminishes or vanishes in concrete with the presence of chlorides or an advanced degree of carbonation. Once the aggressive species entering the concrete reaches the reinforcement, corrosion is more likely to begin a chain of degradation processes that will affect the safety and serviceability of the structure: The reinforcement cross section diminishes, the concrete cover cracks (steel corrosion products have volume two to four times larger than the metal from which they form) and the steel/concrete bond is weakened. In brief, the bearing capacity of structural elements with corroded reinforcements is reduced and their service life consequently shortens (Clark and Saifullah, 1994; Rodriguez et al., 1996; Cairns, 1998). It is well known that the weakest part of modern (high performance) concrete is the reinforcing steel, because it is unstable and tends to be oxidized by oxygen or other substances. A suitable scheme for the service life of corroding structure suggested by Tuutti, (1982) is presented in Figure. 1.

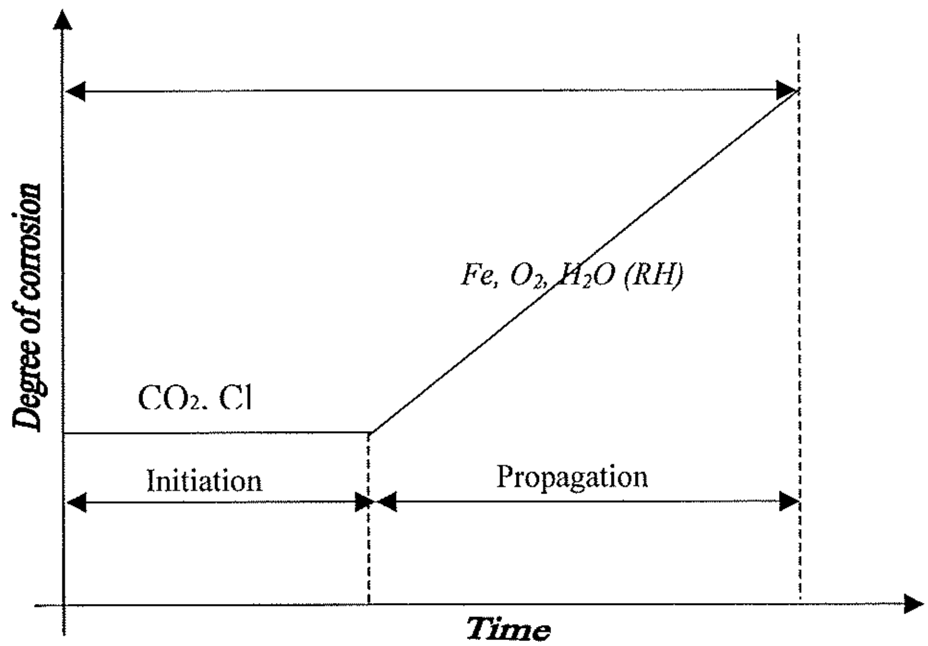


Figure 3.1. Tuutti's model of service life

The model proposed by Tuutti describes corrosion in two parts: 1) initiation period in which external aggressive species enter the concrete cover, and 2) a propagation period which starts when the steel reinforcement depassivates. Bamforth (1993) proposed a model for reinforcement degradation similar to Tuutti's model but slightly more elaborated. It should be emphasized that Tuutti's model is a conceptual device. The deterioration process is much more complex than can not be described by a linear rate, and the deterioration rates for both periods depend on a very large number of factors. Some of these factors are correlated and some processes are coupled.

The construction industry is urgently in need of alternative materials to steel reinforcement that are more resistant to corrosion in order to secure and extend the service life of reinforced concrete structures exposed to aggressive environments. In recent years, Fiber Reinforced Polymers (FRP's) have received much attention as a reinforcement alternative. These fibers are non-corrosive, have a high tensile strength and are electromagnetically neutral.

Acceptance of these materials for a given civil engineering structure will ultimately depend on their long-term durability under a specific set of conditions (application). Accurate estimation of service life of any concrete structure requires considerable knowledge of material behavior, damage tolerance, performance goals, and above all the degradation mechanisms of both components of reinforced concrete, i.e., concrete and reinforcement. Without knowledge of the degradation mechanisms of reinforced concrete, no substantial and long-lasting structural element can be designed.

Because the history of concrete storage tanks is short (in the range of five years) relative to the required service life, there is no empirical database that provides adequate information regarding the behavior of reinforced concrete. Limited data have been published on the performance of concrete used for animal houses or silo storage tanks. However, a large body of scientific data has been published on the degradation of reinforced concrete used in industrial structures. Although these systems can be quite different in their functions, the basic concepts governing the mechanisms of reinforced concrete degradation are similar. The present review was based on data derived from technical research papers that describe the degradation process of reinforced concrete used in a wide range of applications. Following the background chapter, the environmental conditions for agricultural structures are presented, and individual chapters covering the mechanisms specific to each class of degradation mechanisms.

3.1 Environmental Conditions for Agriculture Structures

The most common practice in the hog industry is to use under-floor pits for manure collection, then temporarily stored in earth lagunas or, more recently, in above ground concrete tanks. In both cases, the manure must be effectively contained and managed, usually by land spreading, to avoid pollution. Studies on the distribution of solids, organic matter, pH, and ammonia in fresh swine manure showing that the pH becomes lower as the manure solids become more concentrated, decreasing with time as the manure decomposes. The solids content and temperature of the manure largely affect the kinetics of manure decomposition. A typical composition of swine manure slurry is shown in Table3.2 (Masse et al., 1997).

The main characteristics of the manure that will affect the concrete pore solution, concrete degradation and reinforcement degradation are the neutral pH and high alkalinity level. Analysis of the dry matter in liquid swine manure indicate that it contains a number of elements such as: P, Ca, Mg, Cu, Zn, Mn, Fe, K, B and S. The concentration of these elements was found to vary with the depth in the manure tank.

Significant amounts of aggressive acids are formed in spilled meal-water mixtures, especially in front of wet-feeders in pig houses. Chemical analysis of samples taken from the floors in pig houses showed that acetic and especially lactic acids are formed in meal-water-mixtures, resulting in a pH values below 4.5. Lactic and acetic acid appear to be present in high concentrations and are considered the main cause of severe concrete attack in front of the wet-feeders. In addition, other volatile fatty acids were found in smaller amounts along with aggressive ions such as NH_4^+, Mg^{2+}, Cl^- and SO_4^{2-}.

Silage effluent, a surplus liquid released from wet forage during the ensilage process, presents similar characteristics to spilled meal-water mixtures; the silage effluent contains a range of organic acids (e.g., lactic acid, acid acetic and other volatile fatty acids) and typically has a pH value of approximately 4.

In summary, the chemical components of manure, spilled meal-water mixtures and feed silage effluent in silos create very aggressive environmental conditions for most of the reinforced concrete structures used in agriculture. Changes in other environmental factors such as temperature (freezing/thawing cyclic), moisture content (cyclic wet/dry conditions during different hydraulic loads, i.e., full/empty), do affect the performance of concrete and reinforcement, but there is no quantified information available on such effects.

Table 3.2. Composition of Swine Manure (after Masse et al., 1997)

Constituents	Concentration Mean ± S.D.
Total solids (%)	4.8 ± 0.12
Total suspended solids (%)	3.6 ± 0.20
Volatile solids (%)	3.0 ± 0.16
Volatile suspended solids (%)	2.6 ± 0.30
Soluble chemical oxygen demands (COD) (g/L)	39 ± 9.00
Total COD (g/L)	84 ± 10.00
Ammonia nitrogen (NH_4-N) (g/L)	5.8 ± 0.40
pH	7.4 ± 0.30
Alkalinity (g $CaCO_3$/L)	19.0 ± 2.70
Acetic acid (g/L)	6.3 ± 0.40
Propionic acid (g/L)	1.9 ± 0.15
Butyric acid (g/L)	2.5
Cellulose (%TS)	2.43
Hemicellulose (% TS)	4.15
Lignin (% TS)	1.31
Carbon (% VS)	38.18
Nitrogen (% VS)	4.69
Hydrogen (% VS)	6.10
Oxygen (%VS)*	51.00

* %Oxygen = 100% - (%Carbon + %Nitrogen + %Hydrogen)

3.2 Reinforced Concrete Degradation

The degradation processes in structural units made from conventional reinforced concrete construction materials used for pig manure storage tanks are complex, with many coupled processes in a complicated manner. For presentation purposes, the review of these mechanisms is in two sections. In the first section, the emphasis is placed on the most important degradation processes of the concrete component, and on the dynamics of concrete characteristics that control or influence the degradation of reinforcements. Particular attention is given to processes/properties that control the pH of the concrete pore solution, the movement of O_2 and CO_2, and general liquid and solid mass transport. In the second section, the focus is placed on the most relevant processes for the degradation of reinforcement components.

3.2.1 Concrete Degradation

The degradation of the concrete component might be seen as less relevant to the investigation of the suitability of replacing corrosive steel reinforcement with FRP in reinforced concrete tanks used for manure storage. However, the degradation processes of reinforcements and concrete components are correlated, and many processes are even coupled. For presentation purposes and/or special experimental reasons, these processes can be treated separately, but in real-life systems, they operate simultaneously and interactively.

The most important degradation processes of the concrete component are:

a) Chemical attack,
b) Carbonation,
c) Changes in hydraulic properties, and
d) Cracking.

The degradation of concrete exposed to aggressive aqueous environments received attention in the last decade in evaluating the durability of reinforced concrete (Alekseev et. al., 1993;

Aitcin, 1994). Results from many studies revealed the importance of the water/cement ratio (*W/CM*), which determines the overall porosity and hydraulic properties of the cementitious matrix controlling a series of degradation processes. For example, concrete carbonation will be determined by the migration of CO_2 (or carbonate ion) and by the porosity changes induced by precipitation of calcium carbonate. Chloride-induced deterioration of concrete, and corrosion of steel reinforcement, will depend on the diffusion rate of chloride ions and possibly oxygen in the concrete material. Similarly, the degradation of cementitious materials in acidic environments will be governed by the diffusion of protons into the cement matrix, and by the diffusion of calcium ions towards the solution phases. Since chemical reactions are much faster than the diffusion rates for most species, the degradation process is controlled by the hydraulic characteristics of the concrete cover.

a) Chemical attack - Concrete deterioration can occur through a variety of chemical reactions involving the components of the cementitious matrix, aggregate, and the chemical species present in solution. Such reactions include degradation by acids (organic or inorganic), sulfate and chloride attack, and by magnesium substitution.

Chemical deterioration of concrete can be largely classified into three types of process depending on the predominant chemical reaction taking place. These types are: a) leaching parts, or all, of the hardened cement paste (reaction with water), b) exchange reactions between readily soluble compounds of hardened cement paste and the aggressive solution, and c) swelling, largely due to the formation of new and stable compounds in the hardened cement paste.

For most agricultural structures, the concrete is exposed to organic acids. In animal houses, lactic and acetic acids are dominant in spilled meal-water-mixtures, resulting in a mixture with a pH between 3.3 and 5.5 (De Belie, 1996). In concrete silos, the concrete is exposed to silage effluent which, due to the fermentation processes that is essential to silage production, is highly acidic in nature, with a pH of about 4.0. Both lactic and acetic acid have very soluble calcium salts that can easily be washed away. In such conditions, the concrete porosity increases and the pH of the pores decreases; consequently, the hydrates of the hardened cement paste become unstable and start decomposing. The concrete loses its strength and disintegrates.

Studies carried out to improve the concrete resistance to acid attack in agricultural environments showed that using high alumina cement improves the resistance of concrete to acidic attack under certain conditions (De Belie et al., 1997). The improvement was attributed to the absence of $Ca(OH)_2$ and because the presence of alumina gel protects the hydrated phases down to a pH of about 4 (Robson, 1962). Studies on the resistance of different types of concrete to lactic and acetic acids showed that the use of durable high-quality concrete is an important factor. Furthermore, the performance of the concrete can be improved using admixtures and pozzolanic materials such as: silica fume or fly ash (O'Donnell et al., 1993; De Belie et al., 1997; De Belie, 1996). The positive influence of pozzolanic materials can be ascribed to the pozzolanic reaction, which results in a reduction of the portlandite ($Ca(OH)_2$) content of the concrete as well as a more homogeneous capillary porosity. However, other techniques such as the addition to concrete of a cement-bound surface layer (based on ground tuff) showed the best resistance under highly aggressive exposure (pH 3.8).

The attack by sulfate on cement is complex and comprises a number of possible recrystallization reactions. Various researchers (Reading, 1975; Winkler and Singler, 1972) have identified three main mechanisms: a) formation of ettringite ($C_3A.3(CaSO_4)H_{32}$), b) formation of gypsum ($CaSO_42H_2O$), and c) formation of brucite ($Mg(OH)_2$).

In practice, the above reactions can result in disintegration of the cement structure by expansion and cracking. One method to limit sulfate attack is the use of cement with low C_3A content, such as Portland Type II (< 6% C_3A) and Type V (<4% C_3A). However, a low uncombined CH content in hydrated paste is considered more critical (Fitkos and Parissakis, 1985). The addition of pozzolanic admixtures to portland cement pastes reduces the CH content in hydrated paste, which is then more resistant to sulfate attack (Mehta, 1986).

b) Carbonation - Carbonation of the cement matrix refers to the physico-chemical consequences of the reaction of carbonate ions (CO_3^{2-}, HCO_3^-) with the components of the cement matrix (Parrott, 1992; Lawrence, 1981; Bilcik, 1990). Carbonation will occur spontaneously under many environmental conditions. It is extremely difficult to prevent carbonation even under laboratory conditions; extreme precautions are required if the properties of uncarbonated systems are to be determined.

Carbonation is controlled by diffusion of CO_2 through the carbonated concrete. It is dependent upon the depth of cover, the gas permeability of the carbonated cover concrete and the quantity of cement hydrate available to buffer the carbonation reaction. Water is required in the reaction of CO_2 and $Ca(OH)_2$ to form $CaCO_3$, but increasing water contents slows the diffusion rate of CO_2 through the concrete since diffusion of gaseous CO_2 in water is slower by several orders of magnitude than in air. The reaction between dissolved CO_2 and the cement matrix can either result in enhanced dissolution of calcium as $Ca(HCO_3)_2$ or precipitation of $CaCO_3$, depending on the pH of the aqueous phase.

The depth of carbonation is roughly proportional to the square-root of the exposure time, doubling between 1 and 4 years, than doubling again between 4 and 10 years (Neville, 1981). In one-year aged concrete, the depth of carbonation may vary form 0.05 mm and 19.5 mm. The extent of carbonation is very much dependent on the initial quality of the concrete (low permeability and diffusibility). In a good quality concrete with low *W/CM* ratio, the carbonation rate will typically be 2 mm/y.

Carbonation can have beneficial effects on the longevity of concrete. Carbonation markedly increases the compressive and tensile strength and decreases the permeability of concrete. However, the main consequence of carbonation is the decrease in the pH of the pore solution. In the fully carbonated zone, the pH value is about 8.0. For steel reinforced concrete, carbonation will have a negative effect; the passive layer that covers and protects the reinforcing steel against corrosion becomes unstable and carbonation-induced corrosion starts. For non-ferrous reinforcement, e.g.GFRP, carbonation will have a positive effect; the glass dissolution rate is inversely related to pH value of concrete pore solution.

c) Changes in hydraulic properties - In many engineering applications the permeability coefficient is being used as reference for material durability and for structure service life predictions. It is generally accepted that the durability of hardened cement exposed to aggressive aqueous environments is related to permeability and thereby to the continuity of the pore structure of the hardened cement (Roy, 1989; Regourd, 1985; Aitcin et al., 1983; Mehta and Gjorv, 1982; Powers, 1958). However, the concrete permeability is a dynamic property, considering the time frame of the manure storage tank.

Factors controlling the initial microstructure are also strongly related to material permeability. It is generally agreed that the *W/CM* of a cement paste is the most important single factor in determining the subsequent porosity and therefore permeability of the hardened material. A decrease in *W/CM* decreases both permeability and porosity, and in most cases increases strength. In some cases significant decreases in the *W/CM* ratio can result in difficulties with placing and consolidation of concrete materials. These issues are relevant to designers and constructors using cast in place method for manure storage tanks. However, these difficulties can be overcome by appropriate design of the concrete.

Changes in microstructure may also take place due to solid-solid phase changes and chemical breakdown of hydrated and unhydrated phases of hardened cements. The chemical breakdown may occur through processes such as leaching/dissolution, precipitation, and reprecipitation of certain previously existing components independently of additional hydration (Onofrei et al., 1991). Reaction of the cement with water produces cement-hydrated solids, with an overall increase in solid volume.

d) Cracking - Concrete durability depends on the micro-structural characteristics of the hydrated cement paste, of the transition zone and of the concrete skin. It also depends on the development of micro- and macro-cracks, which can originate from expansion of concrete (sulfate attack), drying shrinkage, autogenous shrinkage, thermal shrinkage and from overloading. Cracking of concrete is one of the most difficult types of degradation to estimate accurately; it is very difficult to predict precisely where and when cracks will occur. However, development of cracks in concrete is rather a norm than an exception. This will alter substantially the hydraulic characteristics of the reinforced concrete. Even a very small crack can change the transport properties of concrete by an order of magnitude. Cracks are particularly damaging when they extend down to the reinforcing steel, which is then no longer protected by the concrete.

Since the concrete is not a truly elastic material and the stress distribution in concrete tends to vary from point to point, the stress – strain relationship in concrete is complex. It is now well know that even before the application of an external load, microcracks already exist in the transition zone between the matrix mortar and coarse aggregate in concrete. The number and the width of these cracks in a concrete specimen would depend, among other factors, on bleeding characteristics, strength of the transition zone, and curing history of concrete. Under ordinary curing conditions, due to the differences in their elastic moduli, differential strains will be set up between the matrix and coarse aggregate, causing cracks in the transition zone. Below 30% of the ultimate load, the transition zone cracks remain stable. Above 30% of the ultimate load, as the stress increases, transition zone microcracks begin to increase in length, width and number. It is generally assumed that below 50% of the ultimate load a stable system of microcracks exists in the transition zone. Further increase in stress up to about 75% of the ultimate load causes the crack system in the transition zone to become unstable and the proliferation and propagation of cracks in the matrix increases.

3.2.2 Reinforcement Degradation

Traditional materials used for primary structures are steel, concrete and combinations of these, i.e., reinforced concrete. The complementary properties of steel and concrete led to widespread use of reinforced concrete beams and slabs in civil engineering industry. However, it is well known that the reinforcing steel is unstable and oxidizes and alternative materials are needed. In recent years, non-corrosible FRP's have received much attention as reinforcement alternatives.

In reinforced concrete construction, a significant segment of the concrete cross-section in the structural element is used solely for locating and protecting the reinforcement component. However, this protection diminishes as aggressive species from the environment ingress the concrete via pore solution. Once the aggressive species reach the reinforcement, a chain of degradation processes starts. As a result the reinforcement cross section diminishes, the concrete cracks and the reinforcement/concrete bonds weaken.

Regardless of the type of reinforcement material used (steel or FRP) the reinforcement is eventually degraded. Although the degradation mechanism of iron corrosion is different from the mechanism of glass dissolution, they have at least two very important similarities. First, they are controlled to a large extent by the alkaline environment in the concrete (i.e. concrete pore solution). Second, the bearing capacity of structural elements with corroded / dissolute reinforcements are reduced, leading to potential structural instability and shorter life service. Numerous technical papers have been published on these two mechanisms. Brief reviews of iron corrosion and glass dissolution are presented in the next sections.

3.2.2.1 Steel Corrosion

The degree to which strength, safety and serviceability of conventional reinforced concrete are impaired because of steel corrosion is a matter of great concern to those responsible for operation and maintenance of structures.

It is well known that steel and iron are unstable materials which tend to be oxidized by oxygen or other substances. The mechanism of corrosion of steel in concrete is a complex phenomenon generally defined as gradual wearing away or alteration by chemical and/or electrochemical oxidizing processes (Taylor, 1990; Schweitzer, 1989; Rosenberg et al., 1989).

Steel or iron corrosion may be produced by several mechanisms. Craig and Pohlman (1987) identified five groups: 1) general corrosion, 2) localized corrosion, 3) environmentally induced cracking (stress corrosion cracking), 4) metallurgical corrosion (inter-granular corrosion) and 5) mechanically assisted degradation (erosion and corrosion fatigue). Although corrosion of iron by direct oxidation (burning) or by acid attack can occur, such corrosion was considered of little concern for reinforced concrete. Corrosion of steel in concrete due to stress corrosion, hydrogen embattlement, or electrolysis due to “stray” electrical current have rarely been reported as possible causees of distress. Electrochemical corrosion is always considered the main cause for essentially all of the corrosion distresses that occur in reinforced concrete.

From the point of view of the physical effect of corrosion on reinforced element, the first two mechanisms, general and localized corrosion, are important. General (uniform) corrosion normally results in a relatively uniform thinning of materials without significant local attack. Uniform corrosion rarely exists in real-life systems; uniform, well-controlled conditions for a reinforcement element are only possible in the laboratory environment. Localized corrosion (pitting and crevice) induced by local variation in electrochemical potential on a micro-scale is the main cause of reinforcement distress in terms of both strength and elongation capacity.

Electrochemical corrosion (indirect oxidation) can result from dissimilar or non-uniform metals (or active sites on the steel surface) or non-uniformity in the chemical or physical environment afforded by the surrounding concrete. Environmental conditions of reinforced concrete used for manure tanks are not uniform; for example, the exterior of the tank will be permanently in contact with air, whereas the interior of the tank wall will be for a period of time in contact with manure, and for other periods in contact with air. The upper part of the concrete wall will be in contact with air on both sides for much longer times that will be the lower part of the tank. Obviously, temporal and spatial variations in terms of moisture content, and oxygen and CO_2 availability within the reinforced concrete wall are to be expected.

When the pH is high and oxygen and moisture are available, a submicroscopic, transparent, continuous oxide layer, thinner than 2 nm and consisting of hydrated ferric oxide (FeOOH) or ferric oxide (Fe_2O_3) is deposited on the surface of the metal (Haussmann, 1964; Steinour 1964). During the hydration of the cement, the principal soluble product is calcium hydroxide, $Ca(OH)_2$, and the initial alkalinity of the concrete is at least that of the saturated-water pH of about 12.4, depending upon concrete composition and the temperature. If small amounts of sodium and potassium oxides are present in the cement, a further increase in the initial alkalinity of the cement paste extracts is expected, i.e., pH values of 13.2 and higher. At a low pH, an oxide or oxyhydroxide is deposited in an incoherent form on the iron surface (Taylor, 1990). Chloride ions can attack the regions exposed so that they become anodes and unaffected areas become cathodes. Since the areas of breakdown are small, high current densities can develop at the anodes, causing pitting corrosion and localized decrease in pH.

The formation, protection capability and disruption of the passivation layer are strongly controlled by the pH of the liquid phase. The electrical potential versus pH diagram is presented in Pourbaix's diagram (Pourbaix, 1966). This diagram is a compact summary of thermodynamic data in the form of electrical potential versus pH curves, which include the electrochemical and corrosion behavior of iron in an aqueous medium and suggests that iron is in a passive state at a pH value in the range of 8 – 13. The passivation layer can be disrupted and localized corrosion (pitting and crevice) can be initiated. The critical pH value for initiation of localized corrosion was established for steel at pH = 9.4 (Marsh et al., 1983). This value decreases with rising the temperature and increases with increasing Cl^- concentration. Experimental results suggested that pitting corrosion will occur at pH values in the range of 9.4 to 10.8 when the pH was adjusted with HCO_3^- and in the range of 9.4 to 11.5 when the pH was adjusted with NaOH (Nakyama and Akashi, 1990).

Carbonation is always associated with a drop in pore water pH and the initiation of corrosion. The protection of steel from corrosion by alkaline conditions in hydrated concrete paste is neutralized by carbonation, where the pH drops from over 12 to about 8 (Neville, 1981). Papadakis et al. (1989) reported the depassivation of steel reinforcement and initiation of corrosion when pH near the bars drops below 9.

It summary, corrosion of steel in concrete can be initiated and maintained under two broad sets of conditions:

1) Alkaline concrete pore solution and presence of chloride ions, or

2) Reduced alkalinity of concrete pore solution due to either leaching of alkaline substances or their reaction with a pozzolanic material, or partial neutralization by reaction with carbon dioxide or other "acidic" materials in the absence of chloride ions.

In either condition, iron, oxygen and water must be present in order for corrosion to occur. Although the presence of water is necessary, excess water can limit corrosion severely. Continuously submerged reinforced concrete rarely exhibits corrosion-induced distress because there is insufficient oxygen present. However, when the concrete structure is exposed cyclically to wet/dry conditions as is the case for manure storage tanks, the corrosion can be accelerated. Air that enters in pores during dry conditions can be trapped in the concrete matrix when wetted and corrosion can progress at high rates (Andrade et al., 1990).

The corrosion rates are controlled by the numerous environmental conditions (i.e., availability of water, oxygen, carbon dioxide, relative humidity and temperature) in which the reinforced structure functions. The corrosion rates are also controlled by the characteristics/conditions of the concrete component, i.e., the degree of concrete degradation, that control the transport processes to and from the concrete-steel interface. The effect of an individual factor on corrosion is not always a straightforward problem. An increased temperature, for example, will enhance the diffusivity of oxygen molecules and reaction rates, but at the same time will decrease the solubility of oxygen. As a result, the net mass transport of oxygen increases with temperature until a maximum is reached where upon the oxygen concentration begins to decrease (Boden, 1994).

3.2.2.2 Glass Dissolution

Sometimes, GFRP is presented as a corrosion resistant alternative to steel reinforcement. It should be emphasized that GFRP is non-corrosive reinforcement; without ferrous iron present, corrosion does not exist. The mechanism of glass degradation is not related to corrosive elements, corrosive fluids or a corrosion mechanisms; it is a dissolution process.

Experimental and modeling work on alteration of borosilicate glass indicates that the most important parameters for glass dissolution rates are: a) exposed surface area of glass to dissolved cations and anions in the liquid phase, b) pH of concrete pore solution, c) dissolved silica concentration in solution, and d) temperature (Bourcier, 1994; O'Connell et al., 1997).

It should be noted that many FRP's are composites (fibers and resin matrix). In addition to the reinforcing fiber itself, the strength and stiffness of composite materials are influenced by the characteristics of the resin matrix and the shear strength of the interfacial bond between fiber and resin (Proctor, 1985). The exposed surface area of the glass component to dissolved cations and anions in the liquid phase is highly dependent on resin matrix characteristics. However, the diversity and recent advances in resin matrix compounds has made possible the development of fiber composites offering a broad and interesting field of application (Keble and Scherer, 1999).

Based on thermodynamic considerations, accelerated dissolution of glass would likely result in the glass being exposed to an elevated-pH solution. Experiments in which the GFRP was exposed directly to synthetic solutions with high pH values may be considered as worst scenarios but they are not necessarily representative for real reinforced-concrete structures. Moreover, the pH value in the concrete pore solution is a dynamic variable during the service life of manure storage tanks. A high silica concrete type in contact with manure could change the pH and composition of the pore solution that will come in contact with the glass fiber. These changes may enhance or retard the glass dissolution rate.

Evaluations of glass fiber–reinforcement cement composites (GFRC) indicate that they undergo embrittlement and strength loss when exposed to a humid environment, although the damage observed to the glass surface is small (Bentur, 1985). When aged in a wet environment, hydration products (mainly CH) eventually deposit between the filaments, and whole strands become effectively bonded to the matrix. As a result, such a strand can no longer be considered a flexible reinforcing unit. These changes can lead to embrittlement, which can be explained on the basis of an increase in flexural stresses in the filaments at the point of intersection with a crack, or of effectively improving bond strength (Bentur, 1985).

Although interfacial changes play an important role in the aging of GFRC, a contribution from chemical attack cannot be completely ignored. Proctor et al. (1982) suggest that this is an important factor in controlling the long–term performance of GFRC, which implies that the currently available glass fibers are not sufficiently immune to alkali attack. This led to the development of new glass compositions with improved resistance to an alkaline environment (Proctor and Yale, 1980). Attempts have been made to improve the performance of GRFC by modifying the matrix with pozzolans (fly ash and silica fume), which can reduce the alkalinity and eliminate deposition of CH around the filaments, the two main mechanisms which can lead to embrittlement (Leonard and Bentur, 1984). It is generally accepted that the use of FRP improves the resistance of glass fibers to alkali.

3.3 Current Designs of Manure Storage Tanks

Traditionally, hog manure is stored in lagoons. To improve manure management (and remain environmentally responsible) when land application is not possible, aboveground structures are increasingly used to store liquid manure.

There are several aboveground structural designs in use today. The most common shape of aboveground structures for manure storage is cylindrical; evenly distributed loads on the cylindrical wall substantially diminish the bending moments. There are two main structural components of a tank: the floor, and the wall with its footing. Often, the two components are constructed separately but the system, as a whole should be watertight to prevent pollution of groundwater and corrosion of the reinforcing steel. However, the major structural component that varies from one design to another is the tank wall.

Several loads can act on the tank at a given time and their magnitude depends on the design and construction of the tank as well as on the environment in which the tank functions. Among the most significant loads are the liquid manure pressure, ice pressure, groundwater pressure, watertable uplift, soil backfill pressure, and vehicular loads. In addition to the dead load, the most important load for which the walls of cylindrical tanks are designed is the circumferential (hoop) load. Generally, the hoop tension is a function of the pressure on the wall and the support at the bottom. Outward pressure on the tank wall is induced mainly by the manure weight and, during the winter, by both liquid manure and ice. The magnitude of hoop tension produced by the liquid manure depends on the tank capacity and geometry (diameter and height), manure specific weight and the ice thickness. For example, for a tank with D=16.00m, H=2.50m and a wall thickness, t = 0.02m, (Jofriet at all., 1995) calculated hoop tension values in the range of 29 to 94 kN m^{-1} along the tank height. Carrier et al. (1995) estimated ice thickness values ~ 0.34m at Quebec City, ~ 0.16m at Guelph, 0.53m at Winnipeg and 0.51m at Saskatoon. Based on these values they suggest that a uniform pressure of 50 kPa over the thickness of the ice at the centre of the tank (or a pressure varying linearly from a maximum of 50 kPa at the surface to zero at the bottom of the ice) should be considered present at the wall in the design of a tank.

Several designs are presently used for manure storage tanks. Each design presents its own advantages (technical and/or economical) as well as drawbacks. In this section are presented briefly the main characteristics of each type of manure storage facility, focussing on the new trends in design and construction of manure storage tanks. It should be noted, however, that this chapter is not a critique on existing methods; this is beyond the scope of this review. This section presents only some general technical information on manure storage facilities gathered from various construction companies.

Earthen manure storage (lagoon) - In areas with soils high in clay content, i.e. > 30%, earth manure storage is the preferred facility because of its low cost. When the soil at the site is not suitable, a 1m clay layer, compacted to secure a very low hydraulic conductivity of ~ 10^{-9}m s^{-1}, is used as a liner. The dimensions of a typical storage lagoon are: 43 to 244m long, 43 to 55m wide and 3.7 to 4.9m deep.

Generally, the lagoon is formed from two cells. The manure is collected in the first cell, about 1/4 of the total volume of the lagoon, where it is kept agitated, and the liquid overflows to the second cell. These facilities are usually designed for 400 days of production. In addition to compacted clay, PVC (30 mil), geotextile containing bentonite, and more recently high density polyethylene, are used as the impermeable layer.

Cast-in-place reinforced concrete tank - Cast-in-place reinforced tanks are designed and built in a wide range of sizes. In Manitoba, DGH Engineering Ltd. builds these tanks with D = 30.48 to 60.96m, H = 3.66 to 4.88m, and t = 0.20 to 0.23 m. The wall of the tank is backfilled within 1.5m of the top, as shown in Figure 3.2.

The backfill consists of compacted material and it is used as structural support for the wall. In addition, the backfill provides protection for the footing from frost. To prevent any possible water pressure if the tank is emptied during periods of high watertable, drainage is ensured by a granular fill base for the floor, and a perimeter drainage system.

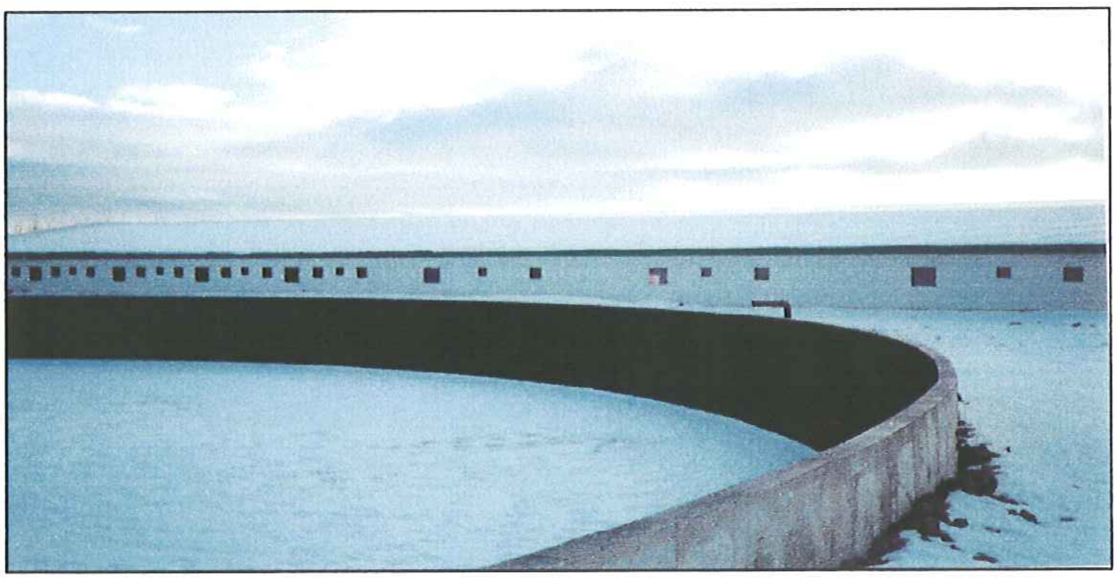


Figure 3.2. Typical cast-in-place reinforced concrete tank (built by DGH Engineering Ltd.).

In the DGH design, vertical steel reinforcing bars are used to transfer the load to the bottom and the top of the tank wall, and the hoop reinforcement is concentrated at the top of the tank. Two major live outward loads are considered in designing the tank wall: the pressure of the manure, and the pressure of the ice. The design and construction of tanks that include concrete, steel reinforcement and backfill materials follow the National Building Code (CCBFC, 1995)

Steel tank - Steel tanks are common in Europe but they have not been used extensively in North America. The common brand name available on the market in Canada is "Slurrystore systems" build by authorized dealers of A.O. Smith Harvestore. There are 21 tank models of various diameters and heights with capacities from 140 to 2.3 million gallons. A typical steel tank is presented in Figure 3.3.

Figure 3.3. Typical steel tank (Slurrystore systems, built by Mannagro Harvestore).

There are two main structural components of the tank: the floor, and the wall with its footing. The floor is built of reinforced concrete and the wall from high-strength steel sheets coated on both sides with a layer of liquid glass. A bentonite seal strip is installed between the foundation starter sheet and the concrete floor. When exposed to liquid, the bentonite strip swells, acting as an effective sealing barrier. The wall panels are assembled and raised on special jacks and the bolts tightened, creating a sound joint. Sidewall stiffeners, made from galvanized web trussing, provide resistance to sidewall buckling in strong winds.

Precast reinforced concrete tank - Tanks are post-tensioned, prestressed systems built with wall panels and grouted joints. Large capacity tanks with heights between 7.62 and 8.84 m, are generally built with precast panels. For example, a recent 8.0 million gallon tank was built at Waldheim Colony Marquette, in Manitoba. A typical precast reinforced concrete tank is shown in Figure 4.

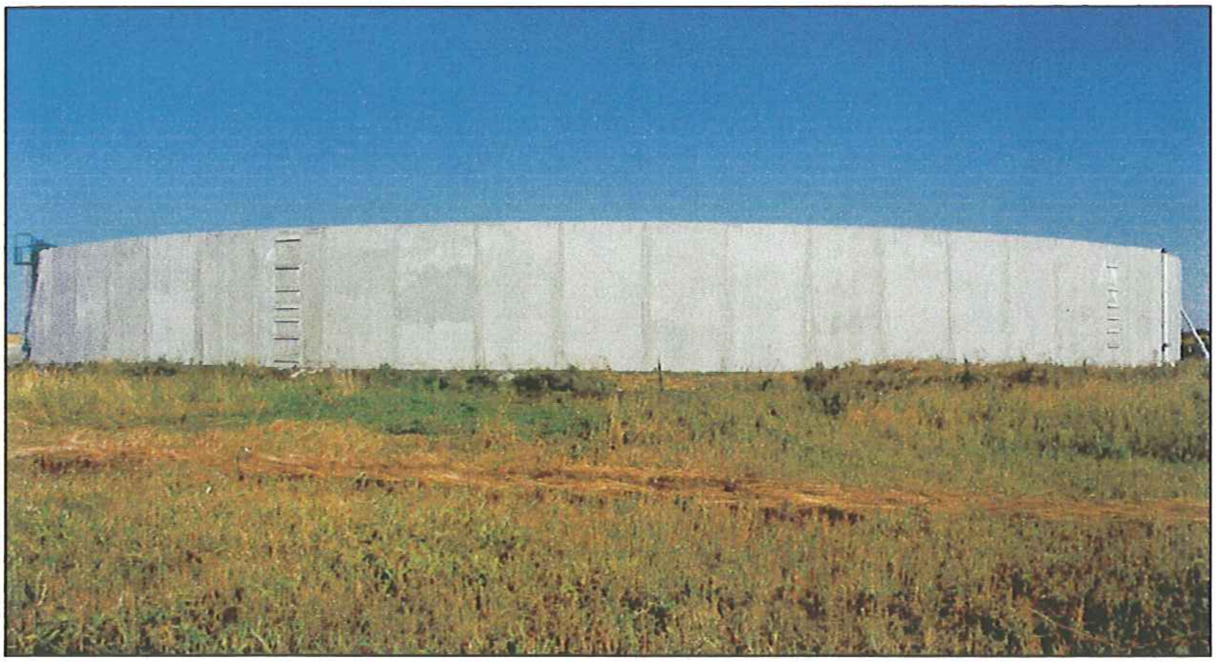


Figure 3.4. Typical precast reinforced concrete tank (built by Con-Force Structure Ltd.).

The concrete panels are cast on steel forms, as flat slabs, in a climate controlled factory environment resulting in concrete panel with no "honeycombing structure" and high strength ($f_c \sim 45$ MPa). Each panel is designed with vertical pre-stressing to resist the calculated vertical bending stresses and an additional ~ 1.38 MPa residual compression. The panels are lifted into place, and horizontal ducts in the panel walls allow them to be post-tensioned horizontally after they are installed and joined to form a continuous wall. The provision of vertical pre-tensioning and horizontal post-tensioning provides a high degree of crack control in the reinforced concrete. The precast panel is designed to be under compression in both the vertical and horizontal direction when the tank is full. The high durability concrete of both floor and wall, combined with the vertical pre-stressing and horizontal post-tensioned design of the panels, ensures structural integrity of the tank in most critical operating conditions.

Cast-in-place, OCTAFORM (PVC) reinforced concrete tank - Essentially, this design is very similar to the standard cast-in-place design presented above. The difference is that the casting forms for this design are PVC panels of a special profile and joining system (known as the OCTAFORM system) which remain in place after the concrete is cast. The height of the panel is 4.877 m. Since the thickness of the wall is variable, tanks of various capacities can be built by adjusting the diameter. A typical cast-in-place OCTAFORM reinforced concrete tank is presented in Figure 5.

Figure 3.5. Typical cast-in-place OCTAFORM reinforced concrete tank (built by Excel / Samson Engineering Services).

From a structural viewpoint, the PVC cover is considered to have no effect on the strength of the wall. However, the PVC panels provide a watertight cover of the reinforced concrete. Unless the PVC panels deteriorate (aging, ultraviolet radiation or chemical attack) the concrete and the steel reinforcement is completely isolated, consequently their durability is significantly enhanced.

3.4 Current State in FRP Applications

The construction industry is urgently in need of alternatives to corrosive steel reinforcement. Various solutions tried in the past to deal with problems associated to corrosion of steel reinforcement included epoxy coatings, increased concrete cover thickness and polymer concrete. These did not provide a long-term solution. In recent years, FRP's have received much attention as reinforcement alternatives. These fibers are non-corrosive, have a high tensile strength and are electromagnetically neutral. Because all these properties, the use of FRP materials in concrete clearly has the potential to increase the longevity of structures, leading to substantial savings in life cycle cost for concrete structures in almost all fields of technology. The most common fibers are carbon (CFRP), aramid (AFRP), and glass (GFRP). The most appealing composite to many industries is GRFP because of its lower cost.

A large number of research programs and studies of physical and mechanical properties of the most common FRP's have been conducted in the last decade (Holloway, 1990; Masmoudi et al., 1996; Rahman et al., 1996; Benmokrane et al., 1997; Rizkalla et al., 1997, Keble and Scherer, 1999; Boyd and Banthia, 1999). Schematic representations of stress-strain relationship of most common FRP composites are presented in Figure 3.6 and details of the mechanical properties of a range of reinforcing fibers and laminates are presented in Table 3.3 and Table 3.4, respectively.

Tables 3.3 and 3.4 present the typical mechanical values of a selected group of FRP composites. There are many other composite materials on the market.

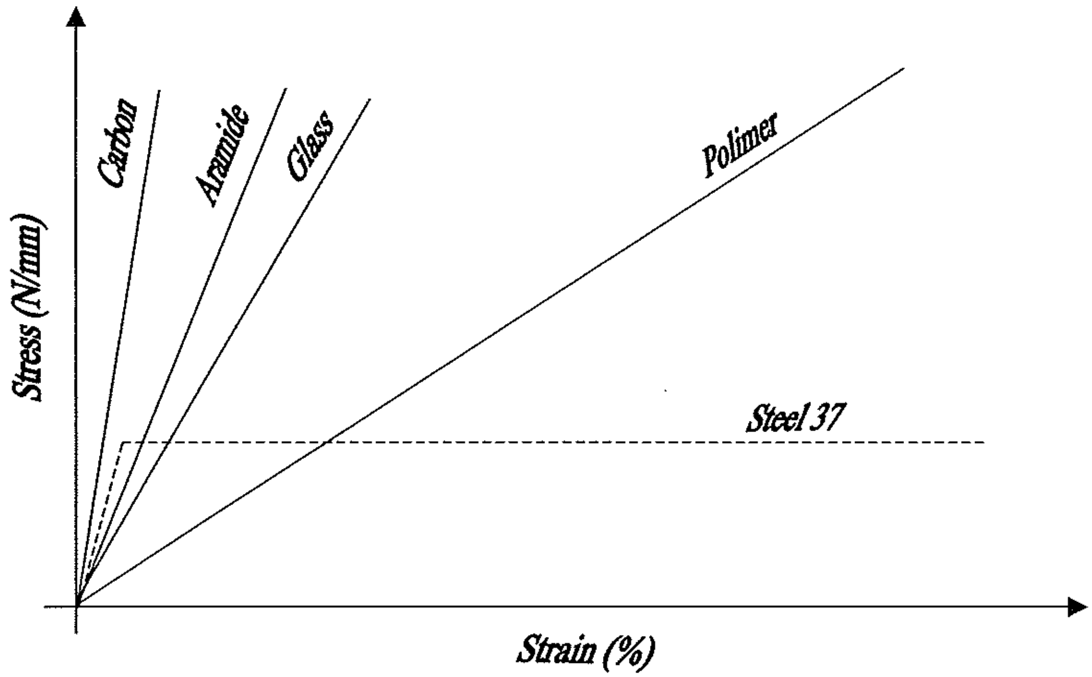


Figure 3.6 Stress-strain relationship (After Keble and Scherer, 1999)

The mechanical properties of specific fiber and laminate are highly dependent on manufacture techniques, quality, fibre volume fraction, control of fibre direction and straightness, etc. Their costs are also variable depending on fabric weights, styles, quantities, etc.

E-Glass is of lower strength and stiffness than the other fibres considered, but it should be pointed out that it is also considerably lower in cost, i.e., about 10 times lower than aramid and carbon and about 20 times lower than polyethylene.

Table 3.3. Fiber mechanical properties for common FRP's (after Kendal, 1999).

	E-Glass	Aramid (Kevlar 49)	Carbon (High Str.)	Polyethylene (Dyn SK65)	Steel (Gr.S275)
Tensile Strength (MPa)	2400	3600	3600-6370	3000	270 Yield 430
Tensile Modulus (GPa)	70	130	230-300	95	205
Failure Strain (%)	3.5	2.5	1.5-2.2	3.6	20
Density (Kg m^{-3})	2560	1440	1800	970	7900
Coeff.of Thermal Expansion $10^{-6}/^{o}C$	5.0	- 2 Long. +59 Trans.	- 1 Long. +17 Trans.	-12 Long.	12

Table 3.4. Laminate mechanical properties of common FRP's (after Kendal, 1999).

	E-Glass	Aramid (Kevlar 49)	Carbon (High Str.)	Steel Grade S275	Concrete
Tensile Strength . (MPa)	650	900	1000-1900	275 Yield 430 Ultimate	2-3
Compressive Strength (MPa)	550	250	~1000	275 Yield 430 Ultimate	25-60
Tensile Modulus (GPa)	30	50	100-120	250	25-36
Tensile Failure Strain (%)	2.3	2.2	1.5-2.2	20	0.01
Density (Kg m^{-3})	1700	1300	1440	7900	2400
Coeff.of Thermal Expansion (10^{-6} / °C)	10	-1	0	12	7-12

Laminates with E-Glass exhibit ultimate strength higher than those provided by conventional structural steel, but with much reduced stiffness. The weight of the GFRP composites are considerably lower than conventional structural materials such as steel, concrete and even aluminum. Use of GFRP will reduce the dead weight and consequently will reduce the load on any substructure. The fiber is non-corrosive, but if unprotected, degrades in alkaline environments. However, alkali-resistant glasses (fiber glass combined with epoxy resins) are available and often used as reinforcement of concrete structures. To overcome the problem associated with low glass stiffness, suggestions have been made to use GFRP in combination with steel (Aitcin, 1998) or as a laminate combination of glass with carbon in a sandwich beam structure (Canning et al., 1999). Although the GFRP is by far the most common FRP used in the industry because it is the least expensive, the price of all FRP composites have dropped steadily in recent years. For example the price of carbon fibers has dropped from US \$20.00/lb in 1990 to \$6.50/lb in 1998; for next year, the projection is \$5.00/lb (Buyukozturk et al., 1999). Selection of a given FRP composite will be controlled by the technical challenges of the problem to be resolved. However, the project cost analysis will determine the selection of the fiber for the most economic solution.

A number of researchers have investigated the combination of advanced polymer composites in conjunction with conventional material to form structural units such as beams (Smart and Jensen, 1997, Evbuomwan, 1998; Norris et al., 1997), columns (Mirmiran and Shahawy, 1996), fibre-reinforced plastic wrap or laminate (Boyd and Banthia, 1999). The use of FRP started in the mid-1980s for repairing and retrofitting infrastructure located in regions with high seismic activity in Japan. By 1996 more than 450 projects were completed that involved seismic retrofitting of tunnels, stacks and other structures (Kobatake, 1998). In Europe the emphasis was placed on flexural strengthening of beams and slabs. (Rostasy and Budelman, 1992; Meier, 1997). In North America use of FRP was made for repairing bridges (Fyfe, 1994; Rizkalla, 1999) as well as in repairing fatigue-damaged steel members (Buyukozzuturk et al., 1999), and wood and masonry structures (Triantafillou, 1998; Saadatmanesh, 1997). Since the beginning of the 1990s, more than 1,500 structures around the world have been repaired and/or retrofitted using FRP composites.

Buyukozturk (Buyukozzuturk et al., 1999), in his concluding remarks of the keynote paper presented to the Stuctural Faults + Repair – 99, International Conference on "Extending the Life of Bridges, Civil+Building Structures" held in London, summarized the current state in FRP applications as follows:

- To date, research and application design topics concerning FRP retrofit of reinforced concrete systems have concentrated on flexural and confinement retrofit through reductionism laboratory and theoretical studies.
- These topics should be expanded to include more far-sighted goals including design methodologies, durability issues, quality control, large-scale applications and educational communication.
- Research is needed on the cyclic performance of FRP laminated systems, and characterization technologies must be developed.
- Design standards and certification programs are needed to assure reliable and durable application of new FRP materials.
- To meet these goals, a coordinated effort between industry, academia, and government is required to develop appropriate specifications and composite structural products.

4 TRIAL TESTS

Two trial tests have been carried out to determine the test specimen design and the test set-up. The primary purpose of the investigation in this phase of the project was to design a reinforced concrete beam that in the test set-up, the main mode of beam failure will be due to rupture of reinforcement. This mode of failure will allow assessing the effectiveness of FRP reinforcement compared to standard steel reinforcement. In addition, it was decided to use small diameter reinforcement in order to obtain information on the behavior of both components, i.e. the concrete and the reinforcement.

4.1 Materials and Test Specimens

4.1.1 Concrete

The concrete type currently used in the construction of hog manure tanks is ordinary concrete with compression strength of about 25-35 MPa for in cast-in-place designs. Construction Materials Group at Lafarge Canada Inc. provided the concrete mix design for the reinforced concrete specimens. The concrete mixes had the following composition:

Cement type 10	240 -260 kg/m^3
Maximum course aggregate 10 mm	897.3 - 930 kg/m^3
Fine aggregate	1025.0 - 1120.6 kg/m^3
Water	136 - 142 kg/m^3
Water reducing agent (322N)	768 – 832 kg/m^3

Two concrete batches have been used to manufacture the reinforced concrete beams used in the trial tests. For each batch the compressive strength of concrete was monitored using 100 mm x 200 mm cylinders, tested as per ASTM A23.1-94 specifications for measuring the compressive strength of concrete. The results of the compression tests are shown in Table 4.1.

Table 4.1. Compressive strength of concrete used in the trial tests.

Batch #	Compressive Strength at 28 days MPa
Batch 1	31
Batch 2	23

4.1.2 Reinforcement

The reinforcement materials used in the trial tests consisted of steel bar (the reinforcement currently used in all designs for manure tanks) and two types of glass reinforcement polymers. Cowin Steel Co. Winnipeg Manitoba supplied the steel rebars. The steel bars were grade 400 W with 400 MPa normal yield strength and 200 GPa elastic modulus. The GFRP materials used in the trial tests were GFRP C-BAR™ and GFRP ISOROD. The MARSHAL COMPOSITE INC. U.S.A. supplied the GFRP C-BAR™. This type of FRP is manufactured using the hybrid pultrusion process. The resins that go into the finished product as well as its closed molding process enhance the resistance of C-BAR™ to high alkaline environment. The physical and mechanical properties of the C-BAR™ reinforcing rod are shown in Table 4.2.

The ISOROD glass fiber rod is manufactured by Pultral Inc. Quebec, Canada. It is made of continuous longitudinal glass-fiber strands bound together with a thermosetting polymer resin, using the pultrusion process. A covering of sand particles with a specific size distribution enhances the surface bonding potential of ISOROD glass fiber rod. Summary of the mechanical characteristics of ISOROD rod is shown in Table 4.3.

Both C-BAR™ and ISOROD bars are characterized by having very high tensile strength. The diameters of the reinforcement bars used in trial 1 and 2 are shown in Table 4.4

Table 4.2. Summary of the physical and mechanical properties of C-BAR™

Property	Units
Weight	0.25 kg m^{-1}
Water absorption	0.25 % max.
Barcol hardness	60 min.
Ultimate tensile strength (average)	770 N mm^{-2}
Ultimate tensile strength (standard deviation)	19 N mm^{-2}
Design Ultimate tensile strength	713 N mm^{-2}
Modulus of elasticity (average)	42,000 N mm^{-2}
Bound strength	17 N mm^{-2}

Table 4.3. Summary of the mechanical characteristics of ISOROD

Property	Units
Tensile strength	690 MPa
Compressive strength	530 MPa
Shear strength	184 MPa
Modulus of elasticity in tension	42 MPa
Modulus of elasticity in compression	43 MPa
Modulus of elasticity in flexure	64 MPa
Ultimate deformation in tension	1.8 %
Ultimate deformation in flexure	2.0 %
Poisson's ratio in tension	0.28
Poisson's ratio in compression	0.31
Coefficient of thermal expansion	9×10^{-6} °C^{-1}
Density	2,000 kg m^{-3}

Table 4.4 Diameter of the reinforcement bars used in the trial tests.

Trial #	Reinforcement type	Diameter (mm)
Trial #1	Steel	11.3
	C -BAR™	12.0
	ISOROD	12.7
Trial #2	Steel	10.0
	C -BAR™	10.0
	ISOROD	10.0

4.1.3 Test Specimens

The reinforced concrete beams were cast in two series, both using the same specified concrete. A total of 12 reinforced concrete beams were cast at Lafarge Canada Inc. using conventional fabrication and curing techniques. The beams were cast in plywood forms. After casting, the beams were left covered in air for 24 hours as shown in Figure 4.1, after which they were de-molded and transferred to the curing room. At twenty-eight days, the beams were removed from the curing room and tested.

The detail design of the concrete beams used in trial 1 and 2 is shown in Figure 4.2. The beams were designed to the extent that was necessary to avoid failure in shear when the beams had been reinforced with FRP. In order to eliminate the shear stirrups and to ensure that the failure of the reinforced concrete beam is due to rupture of the FRP, the dimension of the reinforced concrete beams was enlarged. Two series of six beams were tested. In the first trial beams had a rectangular cross section of 150 mm x 100 mm and length 1000 mm (Figure 4.2 A). The beams in second trial had a square cross section of 170 mm x 170 mm and length 1000 mm (Figure 4.2B). The cross section dimensions were designed to induce high tensile stresses at the top of the reinforced concrete beam where the FRP bars were located. In both trials two reinforced concrete beams were tested for each type of reinforcement.

Figure 4.1. Reinforced concrete beams cast in plywood forms.

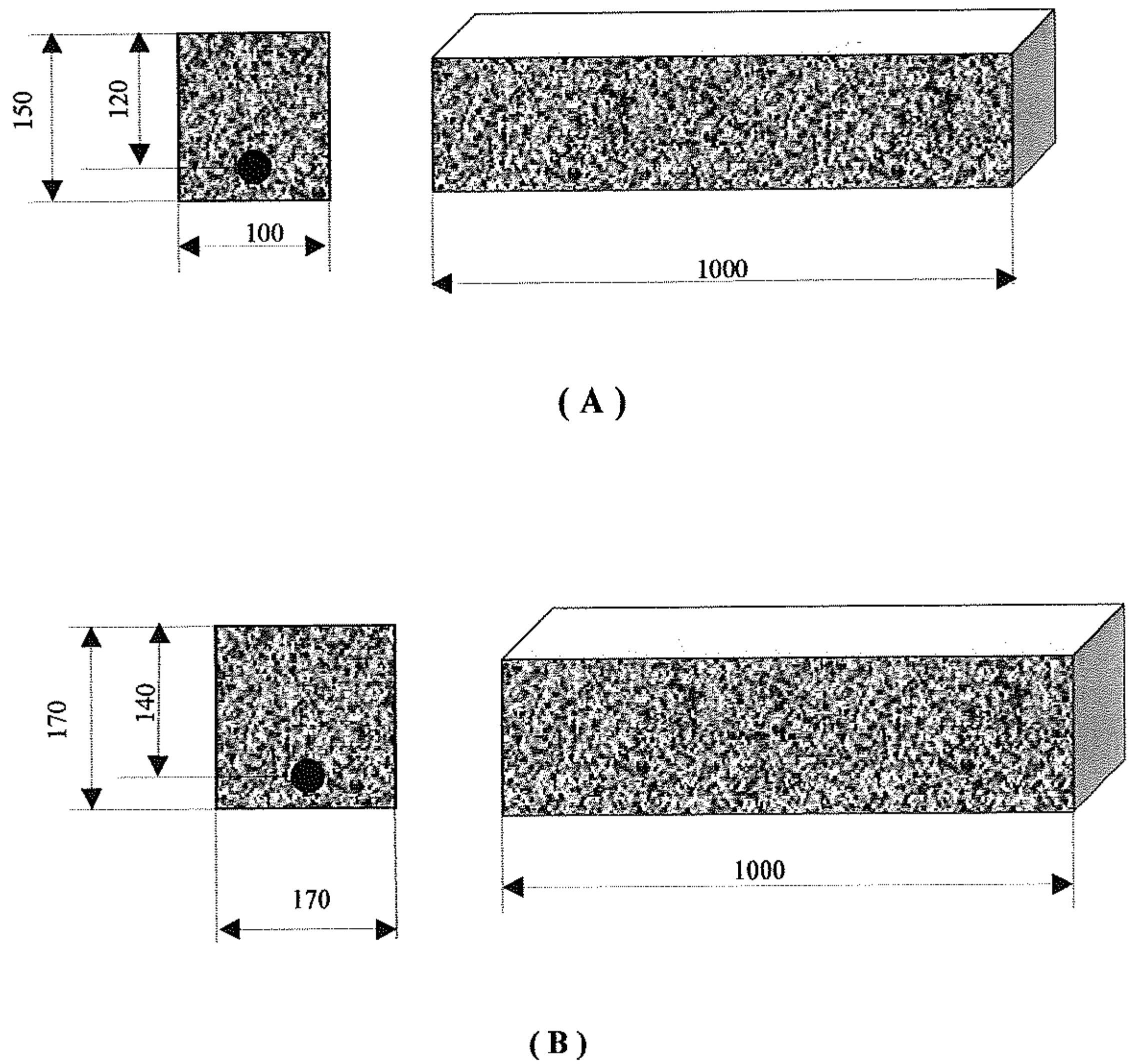


Figure 4.2. Reinforced concrete beams configurations for: (A) trial 1 and (B) trial 2.

4.2 Test Set-up

Two experimental test set-ups were used in the first trial. In the first test set-up the reinforced concrete beam was tested in bending using one single–point loading at the mid span with a support span of 900 mm. A schematic configuration of the test is shown in Figure 4.3A and typical testing of one of the reinforced concrete beams is shown in Figure 4.4A. In the second test set-up the reinforced concrete beams were tested in cantilever mode (Figure 4.3B and Figure 4.4B).

4.3 Trial Test Results

The most common method used to increase the shear strength of a reinforced concrete beam is the use of metallic shear stirrups. However, due to the corrosive nature of the manure storage environment, the use of stirrups was avoided. Consequently, two trial tests were carried out to determine the minimum size of the reinforced concrete beams required to prevent premature failure due to shear in concrete beams without shear reinforcement and to confirm the mode of failure.

Analysis of the results of testing the reinforced concrete beams using two test set-ups and steel and GFRP as reinforcements are described in terms of failure modes and cracking patterns and load-deflection behavior.

4.3.1 Failure Modes

The various modes of failure of the reinforced concrete beams tested in the first and second trials are summarized in Table 4.5 and Table 4.6, respectively. Although the reinforced concrete beams were designed to fail in flexural mode, three types of failure modes were observed in the first trial: shear failure, combined bond/shear failure and crushing of concrete (Table 4.5). Shear and crushing of concrete modes of failure were observed in the concrete beams reinforced with steel bars.

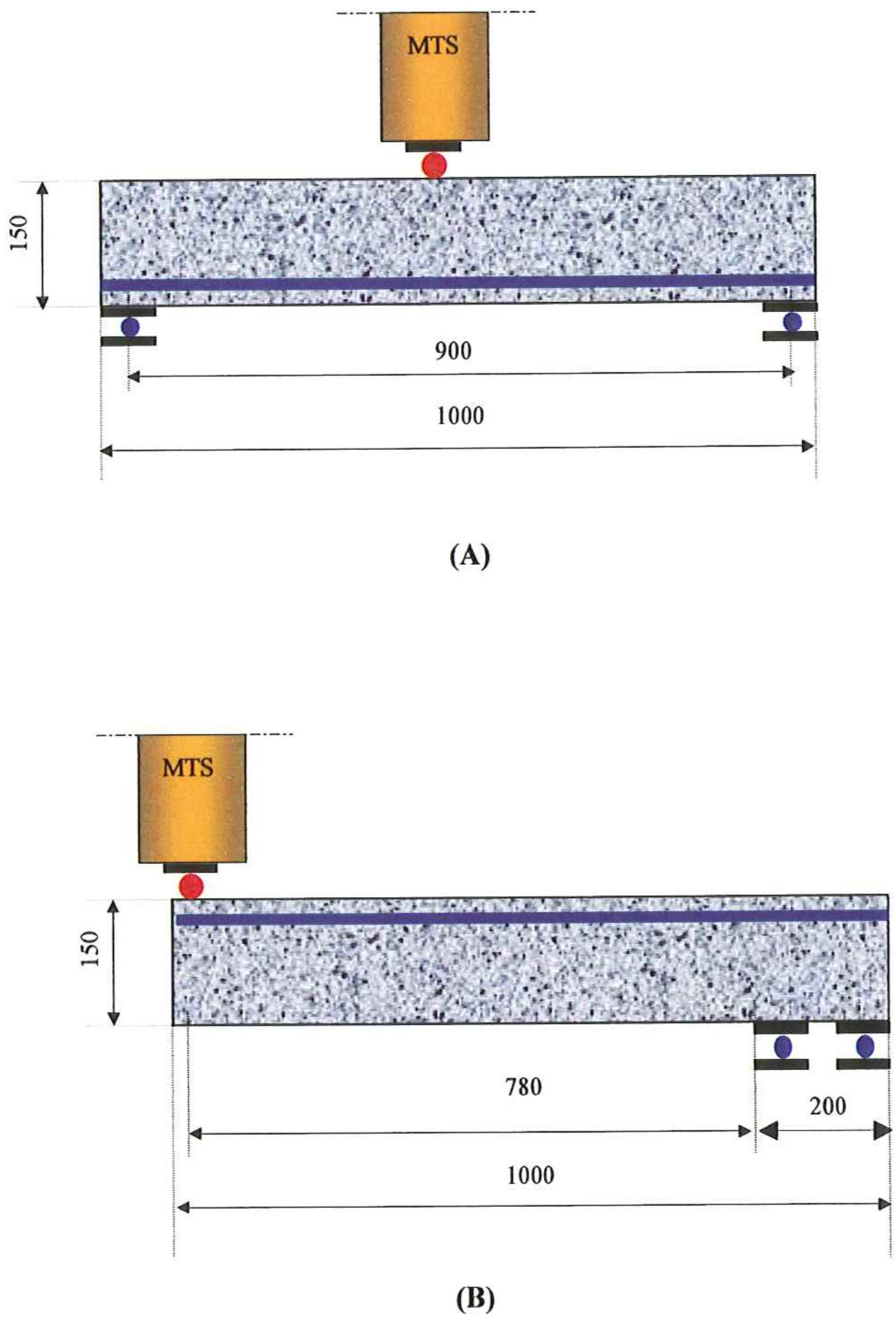
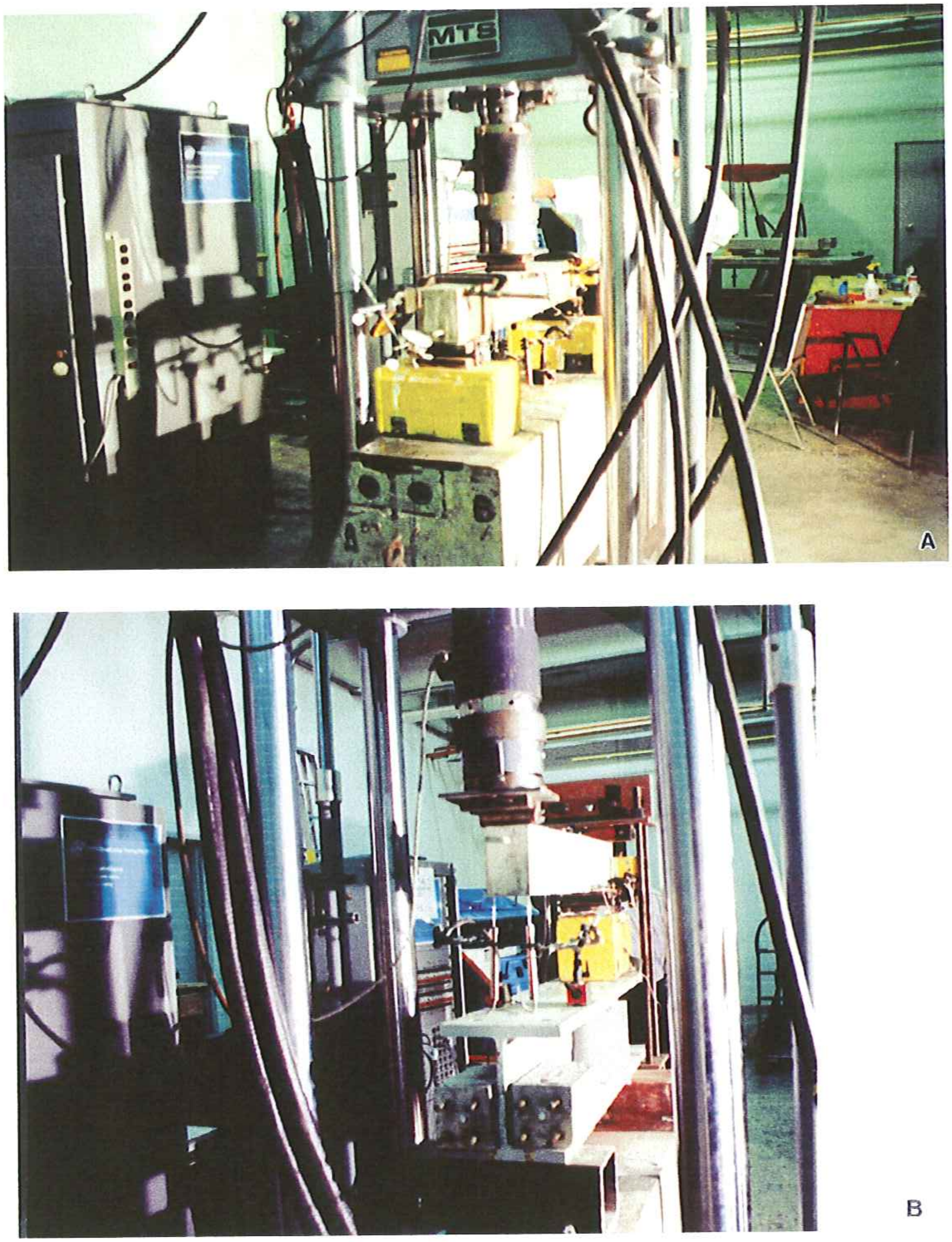


Figure 4.3. Test set-ups in trial 1; A) bending mode and B) cantilever mode.



B

Figure 4.4. Test set-up and location of the instrumentation in the first trial.

A) Beam tested in bending mode, B) Beam tested in cantilever mode.

Table 4.5. Reinforced concrete beams modes of failure in first trial.

Type of reinforcement	Specimen #	Failure load (kN)	Testing mode	Mode of failure
GFRP C-BAR	1	15.7	Bending	Shear (flexural cracks also present)
	2	5.8	Cantilever	Bond/shear (flexural cracks also present)
GFRP - ISOROD	1	8.0	Cantilever	Shear (flexural cracks also present)
	2	8.4	Cantilever	Bond/shear (flexural cracks also present)
Steel rebar	1	8.2	Cantilever	Shear (flexural cracks also present)
	2	4.0	Cantilever	Crushing of concrete (flexural cracks also present)

Premature failure due to shear and combined failure (bond/ shear) modes were observed in the concrete beams reinforced with GFRP before the expected rupture of the reinforcement. The failure load ranged between 5.8 kN and 15.7 kN (Table 4.5).

Based on the results obtained in the first trials, both the test set-up and the dimensions of the reinforced concrete beam were modified (as shown in Figure 4.2B) to ensure that the only mode of failure of the reinforced concrete beam was flexural failure, rupture of the reinforcement.

Table 4.6. Reinforced concrete beams modes of failure in second trial.

Type of reinforcement	Specimen #	Failure Load (kN)	Testing mode	Mode of failure
GFRP C-BAR	1	5.7	Cantilever	Balanced failure Rupture of C-BAR/ Concrete crashing
	2	6.0	Cantilever	Flexural Rupture of C-BAR
GFRP - ISOROD	1	3.6	Cantilever	Flexural Rupture of ISOROD bar
	2	4.1	Cantilever	Flexural Rupture of ISOROD bar
Steel rebar	1	5.7	Cantilever	Crushing of concrete (flexural cracks also present)
	2	6.0	Cantilever	Crushing of concrete (flexural cracks also present)

Only two types of failure mode were observed in the second trial as shown in Table 4.6. All the concrete beams reinforced with GFRP, with exception of one, failed in flexural mode, the rupture of the GFRP in the tension zone at the upper face of the beam at a failure load that ranged between 3.6 kN and 5.7 kN (Table 4.6). The ultimate failure of one of the beams reinforced with GFRP ISOROD was considered to be a balanced failure, rupture of reinforcement and crushing of concrete. The increase in the cross section of the beams as well as the increase in the length of the beam using the HSS section prevented the reinforced concrete beams from premature shear failure. In addition, the failure patterns indicated that the bond between the GFRP and concrete was strong enough that no debonding was seen to occur and no GFRP end slip was measured. The failure in the concrete beams reinforced with steel bars was characterized by yielding of steel bars followed by concrete crushing in the compression zone at the bottom face of the beam at a load level of 5.7 kN and 6.0 kN (Table 4.6). In both trials, the failure in the beams was always seen to occur near the support point position, where initial flexural cracks had been observed.

4.3.2 Crack Patterns

The crack patterns at failure of the reinforced concrete beams tested in the first trial are show in Figure 4.6. Initial cracking occurred at a load that ranged between 1.6 kN and 2 kN. Cracking consists predominantly of flexural cracks (vertical cracks). The stabilization load of flexural cracks ranged between 3 kN and 6 kN. As loading progressed, the flexural cracks bent over in the shear regions. However, only one crack led to failure as shown in Figure 4.5.

Figure 4.5. Crack pattern and mode of failure in reinforced concrete beams tested (trial 1).

The arrows in Figure 4.5 indicate the crack that led to failure. The beam 1, 3 and 5 failed in typical shear mode, beams 2 and 4 failed in a combined mode (shear/bound failure initiated by shear crack) and beam 6 failed in a typical crushing mod.

The typical cracking patterns of beam specimens tested in the second trial at failure are shown in Figure 4.6. The occurrences of cracks in the reinforced concrete beams tested in second trail were considerably fewer in number than in the beams tested in the first trial. In general, the flexural failure observed in the concrete beams tested was characterized by only a few flexural cracks. The cracking loads ranged between 1.9 kN and 4.5 kN as shown in Table 4.7. The steel reinforced concrete showed a higher cracking load level than the GFRP reinforced concrete beams. In most beams, the first crack was initiated in the vicinity of the metallic plate, which was used to ensure that no slippage of reinforcement would take place (Figure 4.7). After the formation of the first crack, the increase in the applied load resulted in the formation of new cracks randomly distributed in the vicinity of the first crack. The results indicated that the stabilization of the flexural cracks for the beams tested in the second trial occurred at a load level ranging between 3.4 kN and 4.8 kN (Table 4.7). In the stabilization stage of the crack pattern, the increase of the applied load contributed to the growth of both width and height of the cracks, particularly in the crack that ultimately led to failure.

Table 4.7. Cracking load and deflection at cracking in the reinforced concrete beam trial 2.

Reinforced concrete beam type	Cracking load (kN)	Stabilization load (kN)	Deflection at cracking (mm)
Concrete Beam reinforced with GFRP - ISOROD	2.87	3.6	5.51
	2.17	3.5	0.76
Concrete Beam reinforced with steel	4.50	4.8	5.2
	4.50	4.8	5.7
Concrete Beam reinforced with GFRP – C - BAR	2.24	3.4	0.6
	1.90	3.5	0.5

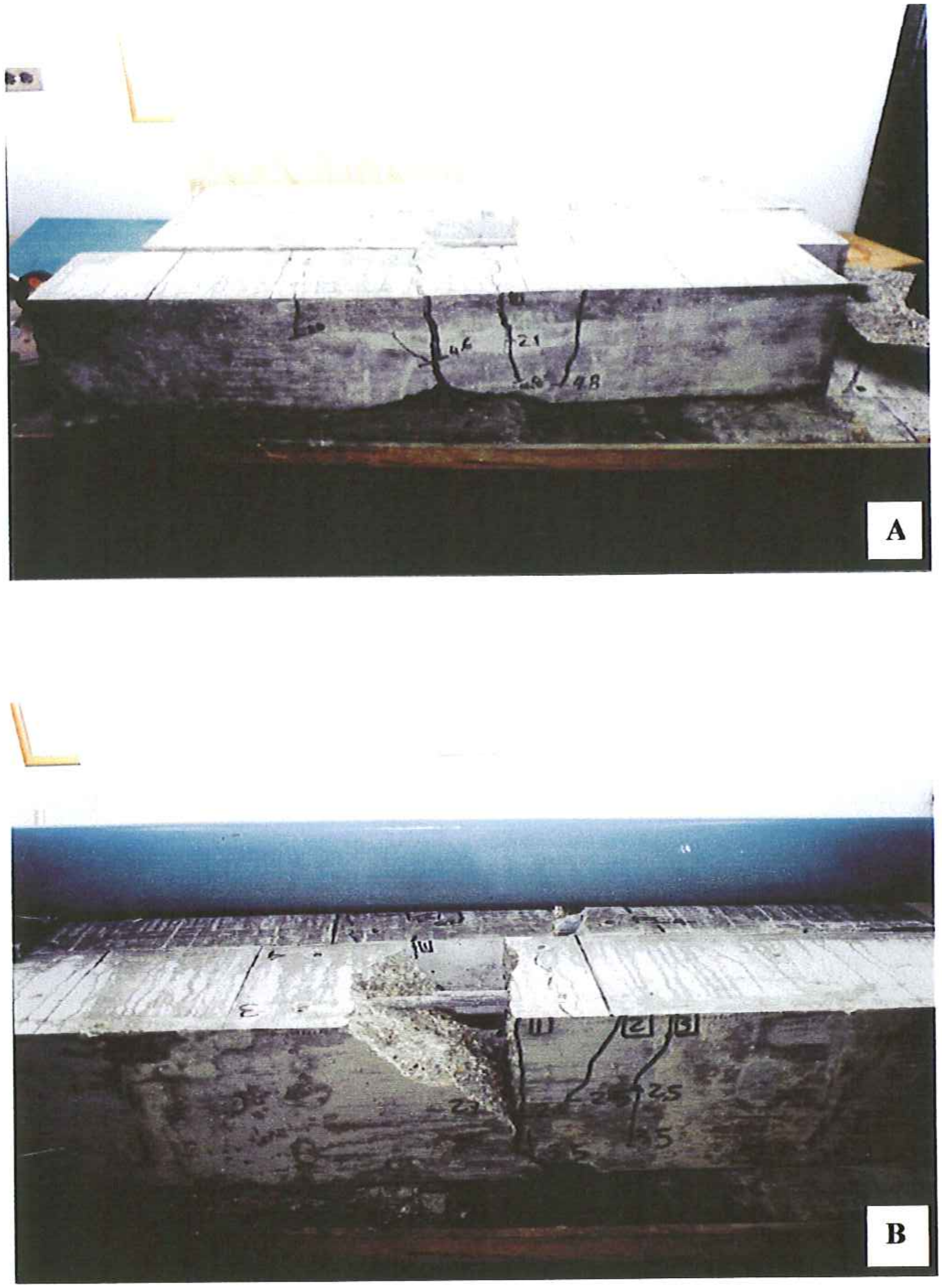


Figure 4.6. Typical crack pattern and failure modes in reinforced concrete beams tested in second trial: A) Crushing of concrete and B) Rupture of reinforcement.

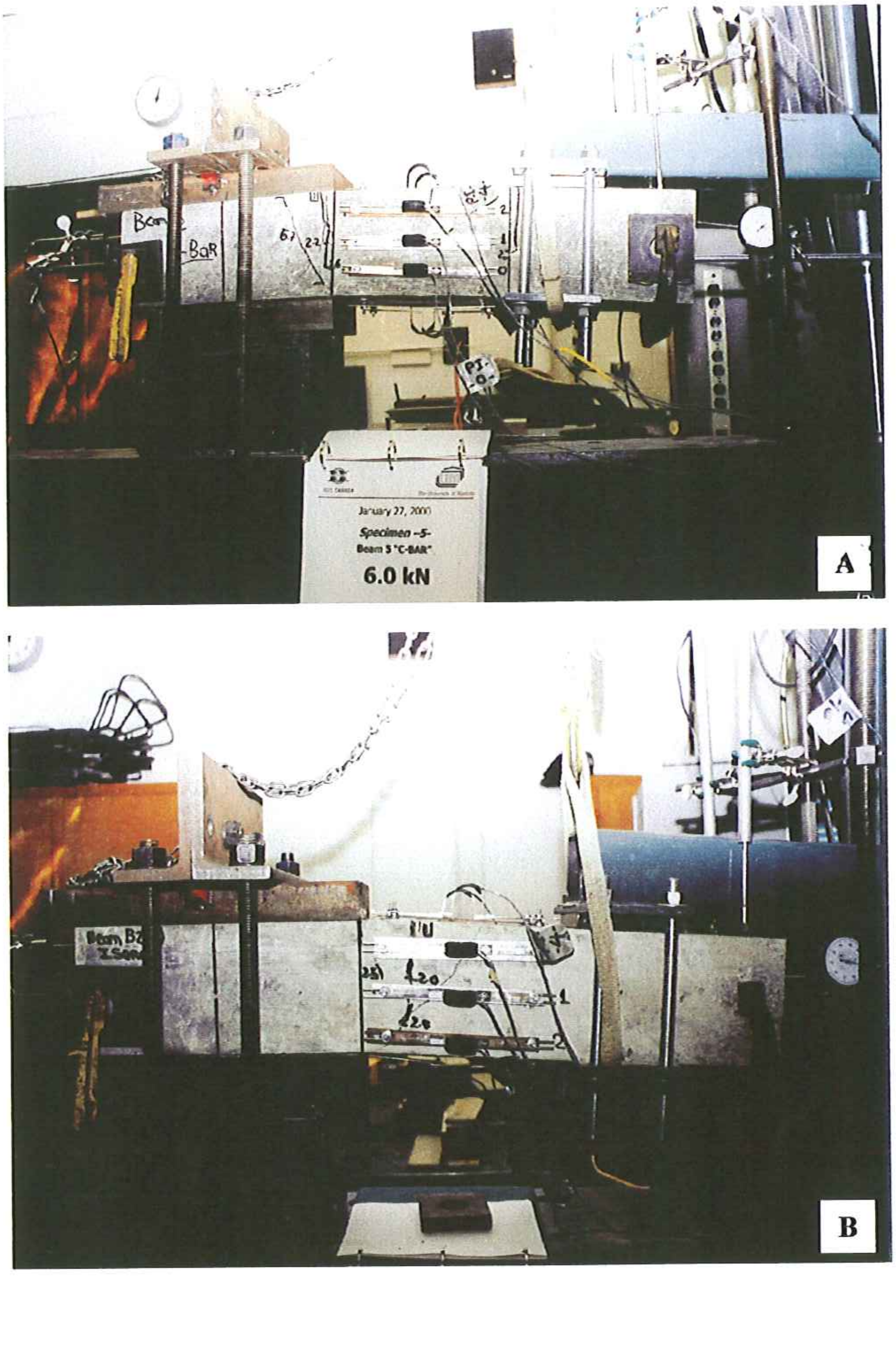


Figure 4.7. Crack pattern and load - deflection behavior in concrete beams before failure.

A) Beam reinforced with C-BAR and B) Beam reinforced with ISOROD.

4.3.3 Load-Deflection Behavior

The load-deflection curves for the reinforced concrete beams tested in the first trial are shown in Figure 4.8. The results show large variation in the measured strength for the reinforced concrete beams. The failure load ranged between 4 kN and 15.7 kN (Figure 4.8). The highest failure load (15.7 kN) was measured in the reinforced concrete beam tested in the bending mode. In general, the load–deflection relationship of all the reinforced concrete beams tested in the first trial are characterized by linear behavior up to cracking load and a non-linear behavior with significant increase in deflection with the formation of new cracks. The discontinuities and the decrease in the applied load in the load-deflection curves reflect cracking of the reinforced concrete beams (Figure 4.8). The concrete beams reinforced with steel exhibit significant deformation after yielding of the steel without a significant increase of applied load until crushing of concrete occurred. In contrast, the load-deflection behavior of the beams reinforced with GFRP show a deflection that increases with the applied load until failure.

Figure 4.9 shows the load-deflection relationship of reinforced concrete beams tested in the second trial. In general, the tested beams exhibited a similar behavior as those tested in the first trials. The beams showed a linear behavior up to the first crack at a load level that ranged between 1.9 kN and 4.5 kN and a non-linear behavior after that with an increase of deflection due to reduction in stiffness with the formation of new cracks. Gradual drops in beams resistances are typically observed after reaching the cracking load. The failure load ranged between 3.6 kN and 6 kN as shown in Table 4.6. The concrete beams reinforced with steel exhibited greater deflections at cracking than the GFRP reinforced concrete beams.

The experimental work performed led to the selection of the reinforced concrete beam design, testing mode and the test set-up that will be used throughout the project. From the observations of the results and the conditions considered in this study the minimum size of the reinforced concrete beam required to prevent the premature failure due to shear in concrete beams without shear reinforcement has been established at a cross-section of 170 mm x 170 mm and a length of 1000 mm with a reinforcement ratio $\rho_{GF} = 0.27$ %.

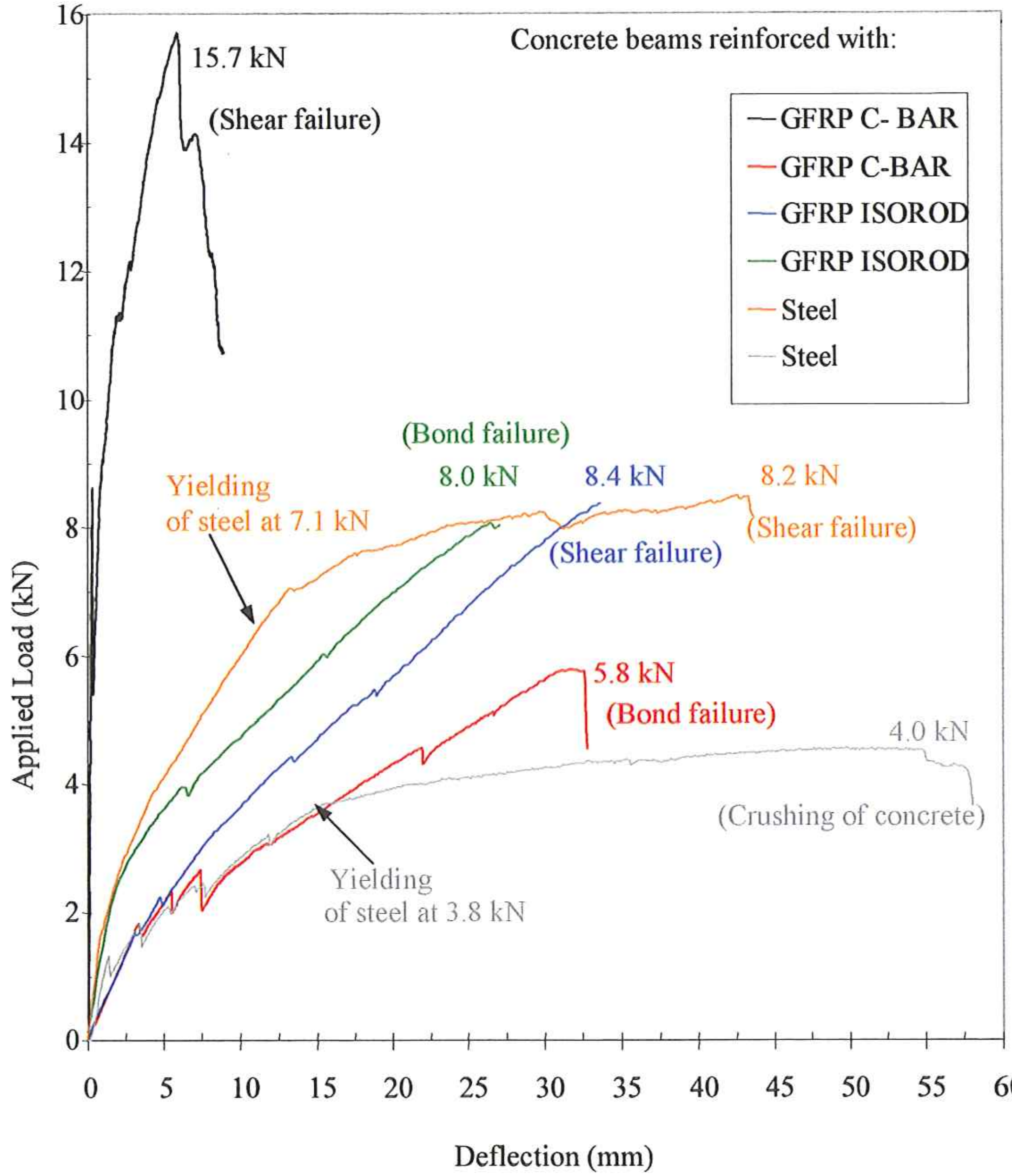


Figure 4.8. Load–deflection behavior of the reinforced concrete beam tested in the first trial.

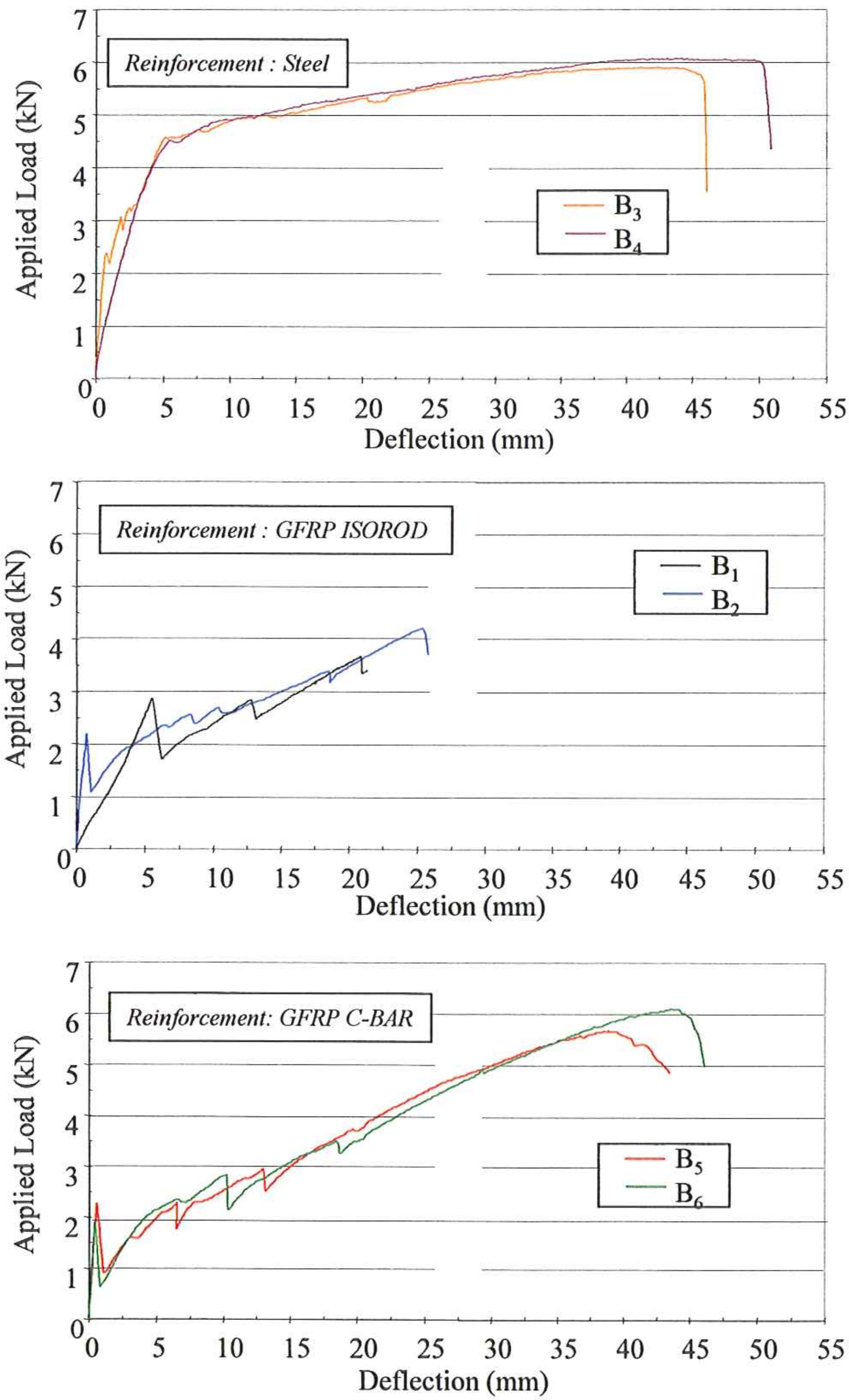


Figure 4.9. Load–deflection behavior of the reinforced concrete beam tested in second trial.

The experimental work also supported the selection of the testing mode and test set-up. For determination of the flexural behavior of the reinforced concrete beams, a cantilever-testing configuration with the load applied at 20 mm from the edge of the cantilever was chosen. The chosen test set-ups facilitated the selected cantilever configuration through mechanical modifications to enhanced flexural failure in the reinforced concrete beams and impede slippage of the reinforcement without increasing the dimensions of the beams. The length of the concrete beams was increased by 1020 mm using a hollow structural steel section (HSS). A steel plate with a 400mm width was used to impede slippage of the reinforcement. These provided an excellent solution for eliminating problems such as handling and cost associated with casting very large and heavy reinforced concrete beams.

5 EXPERIMENTAL PROGRAM

The present study focused on the effects of environmental exposure on long-term performance of various reinforced concrete structural elements (concrete type - reinforcement combinations) for use in hog manure tank applications. Such exposure included chemicals in liquid manure solutions, temperature and moisture, over the duration of contact of structural elements with hog manure. The effects considered include changes in physical and mechanical properties. In order to investigate the effects on long-term performance of composite materials, the experimental program considered methodologies (test conditions) to accelerate the degradation phenomena in reinforced concrete. The experimental procedures are described systematically below:

5.1 Test Variables

Three variables were considered in the experiment: 1) reinforcement types, 2) type of protective isolation of reinforced concrete, and 3) duration of exposure of the reinforced concrete to manure (Table 5.1).

The concrete types currently used in the construction of hog manure tanks include: ordinary concrete with compression strength of about 25 MPa used in cast-in-place designs, and high strength concrete of about 50 MPa compression strength used in pre-cast reinforced concrete design.

In reinforced concrete construction, a significant segment of the concrete cross-section in the structural element is used solely for locating and protecting the reinforcement component. However, this protection diminishes as aggressive species from the environment ingress the concrete via pore solution. Once the aggressive species reach the reinforcement, a chain of degradation processes starts. Consequently, the reinforcement cross section diminishes, the concrete cracks and the reinforcement/concrete bonds weaken.

Table 5.1. Test variables considered in the Experiment Program

Concrete type	Reinforcement type	Confinement (concrete isolation from manure)	Exposure time to manure
1-OrdinaryConcrete ($f_c \sim$ 25-40 MPa)	1- Steel rebar (ϕ =6.35 mm) 2- GFRP C-BAR (ϕ =10.0 mm) 3- GFRP ISOROD (ϕ =10.0 mm) 4- GFRP spray (Coating ~ 3.5 mm)	1- No isolation 2- GFRP spray 3- PVC (Octaform)	t_x= 0 (control) t_1= 4 - months t_2= 8 - months t_3= 12 - months t_4= 18 - months

With regard to reinforcement materials, attention was focused on steel rebar (the reinforcement currently used in all designs for manure tanks) and on three types of glass fiber reinforcement polymers that are the least expensive and have a great potential to improve significantly the service life of these tanks. The GFRP materials under investigation were the GFRP C-BAR^{TM}, GFRP ISOROD and GFRP spray composite. MARSHAL COMPOSITES INC. U.S.A., using the hybrid pultrusion process, manufactures the GFRP C-BAR^{TM}. The ISOROD glass fiber rod is manufactured by Pultral Inc. Quebec, Canada. It is made of continuous longitudinal glass-fiber strands bound together with a thermosetting polymer resin, using the pultrusion process. Summaries of the physical and mechanical properties of both GFRP reinforcing materials have been shown in Table 4.2 and Table 4.3.

Little is known about the protective capacity of PVC used in the Octaform system design. However, the PVC and GFRP spray wrap of the tank wall should protect the reinforced concrete or the concrete against early failure, although mechanical cracking may damage the concrete

A variety of structural elements can be designed, combining the concrete, reinforcement and confining materials presented in Table 5.1. Four such structural elements have been investigated in this study (Table 5.2).

Table 5.2. Structural elements included in the Experimental Program

Element No.	Concrete / Reinforcement / Confining Material Combination
I	Ordinary Concrete (OC) / Steel rebar
II	Ordinary Concrete / GFRP ISOROD
III	GFRP spray / Ordinary Concrete / Steel rebar
IV	PVC / Ordinary Concrete / Steel rebar

5.2 Test Specimens

Two different types of specimens have been used in this study. These include:

1. Straight reinforcement bars,
2. Reinforced concrete beams.

5.2.1 Straight Reinforcement Bars

The reinforcement bars under investigation were steel (the reinforcement currently used in all designs for manure tanks). The GFRP C-BAR™ and GFRP ISOROD (Figure 5.1) have been supplied by Cowin Steel Co. Winnipeg Manitoba, MARSHAL COMPOSITES INC. U.S.A.

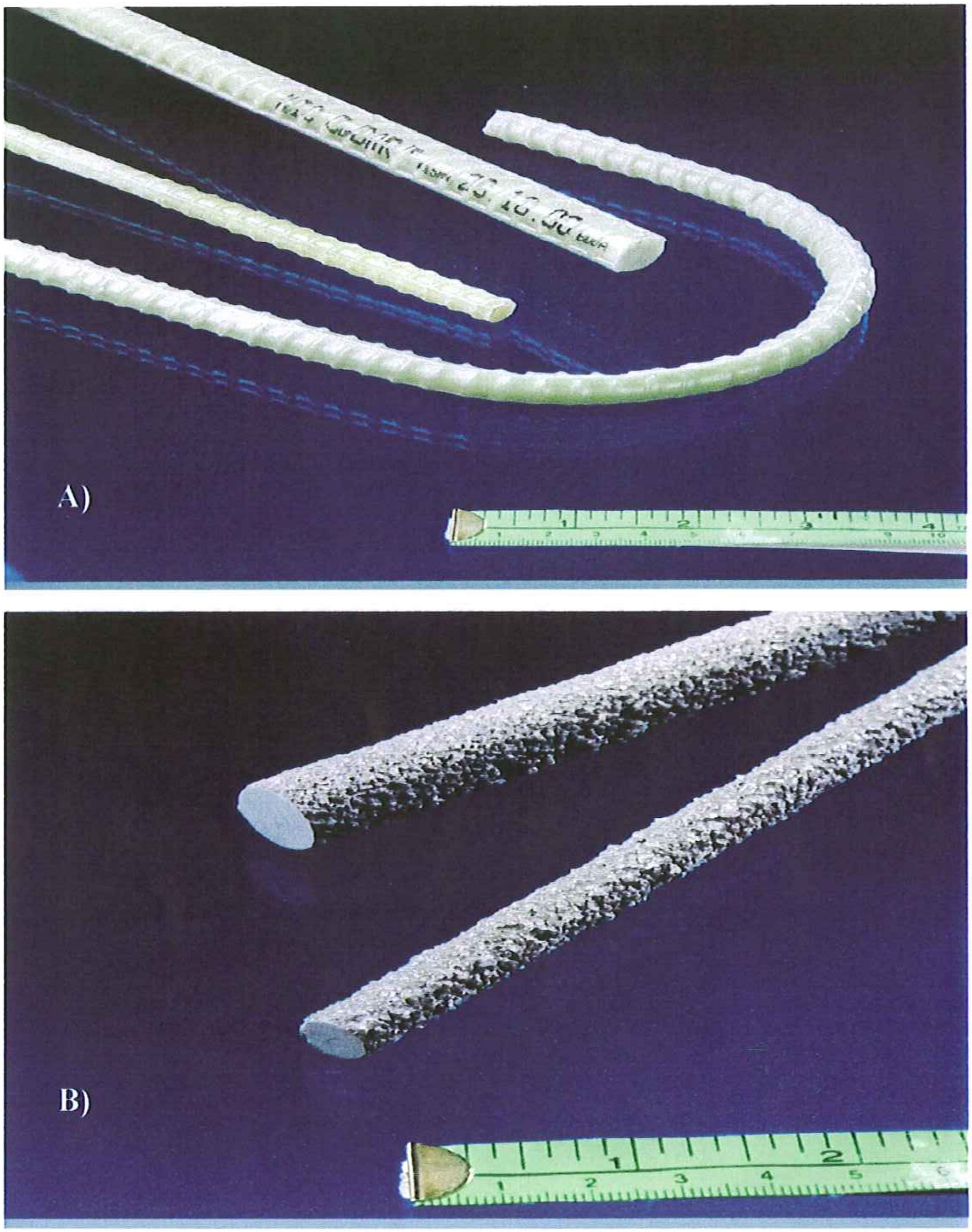


Figure 5.1. GFRP materials under investigation. A) GFRP C-BAR^{TM} and B) GFRP ISOROD

and Pultral Inc. Quebec, Canada, respectively. Fifty-two reinforcement bars 1010 mm long have been included in the experiment program and tested. Changes in the durability of the reinforced bars with exposure time under the manure environmental conditions have been assessed through changes in the tensile strength.

5.2.2 Reinforced Concrete Beams

The tests were designed to avoid premature failure in compression or shear when the flexural test was performed on reinforced concrete beams exposed to manure. In order to obtain information on the behavior of both components (i.e. reinforcement and concrete) a small diameter of steel reinforcement was mandatory (Figure 5.2)

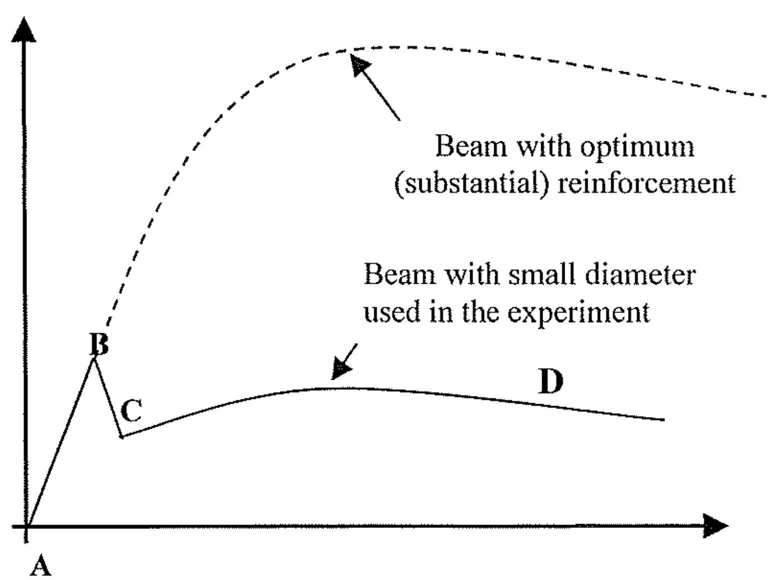


Figure 5.2. Schematic representation of load-defection behavior for beams with different reinforcement type.

The first part of the curve (A-B) provides information about the strength of the concrete (concrete acts alone with little help from the steel). The C-D portion of the curve provides information about the strength of steel reinforcement (in point C the steel strength is mobilized).

However, identical small diameters of steel reinforcement bars and GFRP bars were not available. From the common reinforcement bars currently used in reinforced concrete the most appropriate bars select were steel (ϕ ~6.5 mm) and GFRP (ϕ ~ 10.0 mm).

The test beams have been designed according to Canadian Code standards. They were designed to ensure that the only mode of failure of the reinforced concrete beam is flexural failure, the rupture of the reinforcement. The reinforced concrete beams have headed a nominal cross-section of 170 mm x 170 mm and a total length of 1000 mm. The beam designs are presented in Figures 5.3. Sixty-two reinforced concrete beams have been tested. Three different cover schemes were implemented: 1) concrete beams not covered with any protective materials to simulate the actual real - life conditions where the reinforced concrete is in direct contact with manure 2) beams sprayed on the four vertical sides and the bottom face of the beam with GFRP and 3) beams covered on the four vertical sides and the bottom face of the reinforced concrete beam with PVC plates. Changes in the structural component properties with time under hog manure environmental conditions have been assessed through changes in flexural behavior.

5.3 Test Conditions

To determine the performance of each structural element considered in this study in the particular environment, the test specimens have been exposed to hog manure in specially design experimental containers. In order to perform the exposure test in a reasonable time, special test conditions have been ensured to "accelerate" the degradation processes. Adequate long-term durability testing requires many years of exposure, specially when attempting to assess the performance of reinforced concrete intended to remain in service for very long times. Alternatives to long-term testing were required.

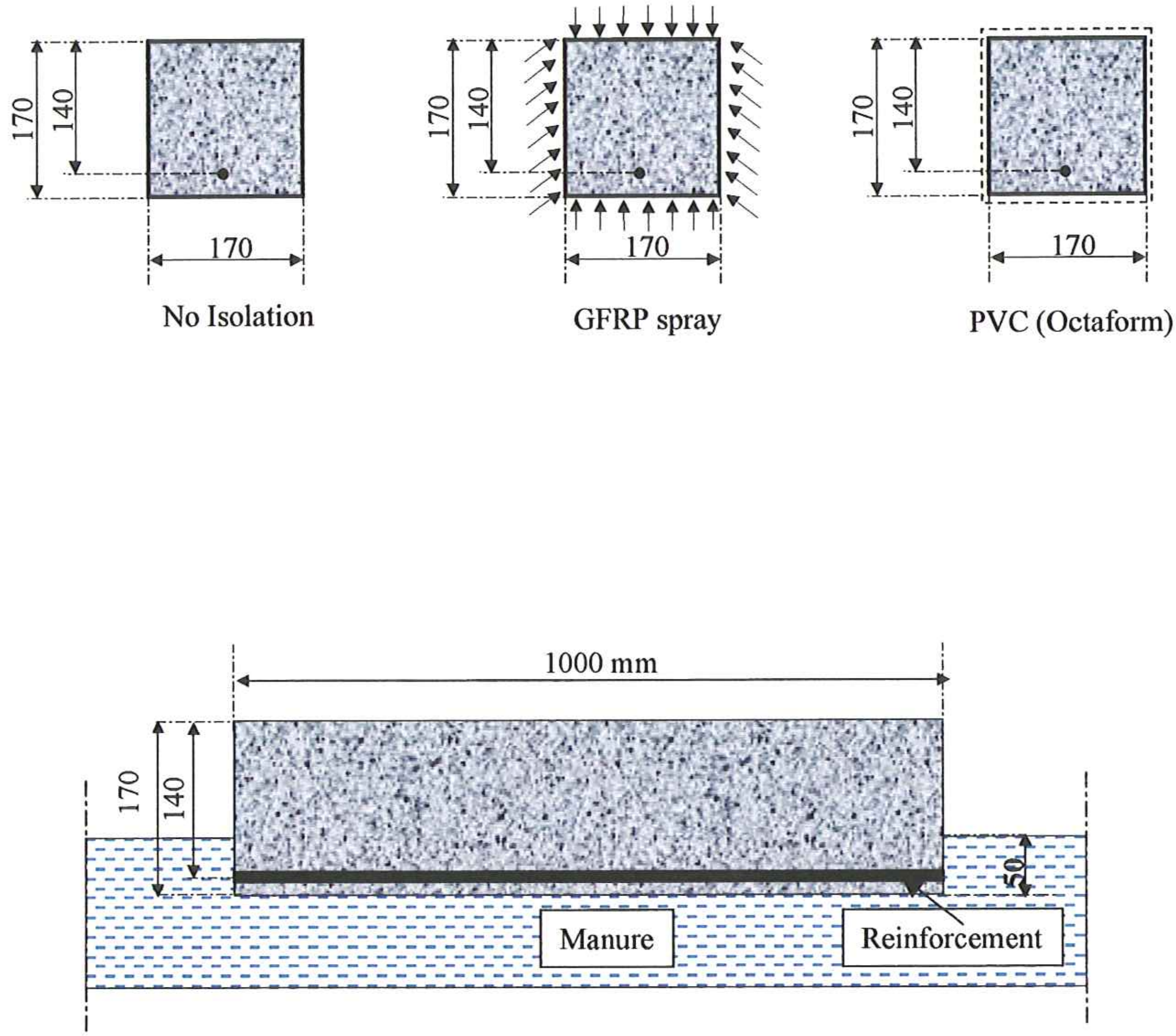


Figure 5.3. Beam geometry (dimension in mm) and cover material type.

However, for the accelerated tests to give meaningful results, degradation mechanisms occurring under accelerated conditions should be the same as those occurring under normal conditions. Acceleration in the experiment have been achieved by using elevated temperature and exposing the structural elements to wet/dry cycles. The test conditions have been precisely controlled: the manure in the experimental tanks have been replaced every two weeks. The manure has been also maintained at a constant temperature of 22 °C (±3 °C) during the entire duration of the experiment (winter / summer). The reinforced bars have been totally immersed in manure and exposed to wet/dry cycles. The standard reinforced concrete structural elements, (concrete / rebar combinations) as well as the confined (isolated from contact with manure) reinforced concrete structural elements (GERP spray or PVC / concrete / rebar combinations) have been partially immersed in the manure (Figure 5.2) and exposed to a wet/dry cycle. The wet and dry cycle involved 15 days exposure to hog manure and 15 days exposure to air at room temperature, i.e., ~ 22 °C (±3 °C). The test conditions in the test program are presented in Table 5.3.

To investigate the degradation rate dynamically, test specimens have been kept in contact with manure for four, eight, twelve and eighteen months. Significance of the environmental exposure effects on long-term performance of the different reinforced concrete structural elements have been benchmarked against control samples.

5.4 Mechanical Test Set-up and Procedures

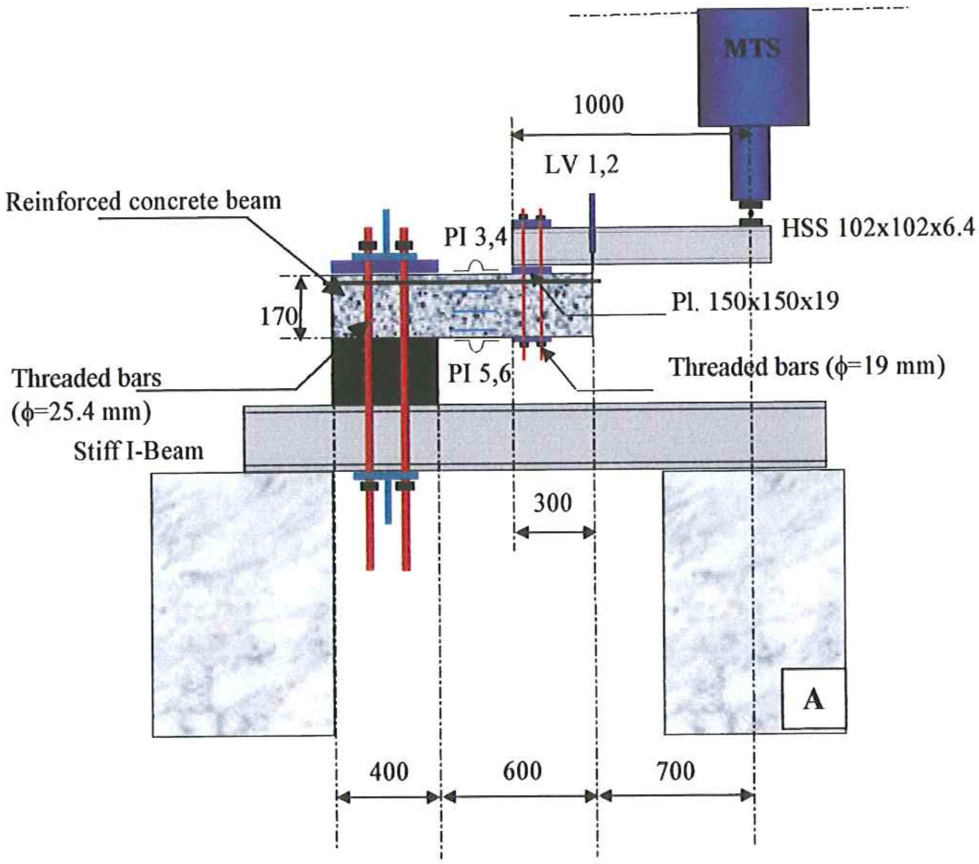
Mechanical testing was conducted at W.R. McQuade Structural Laboratories at the University of Manitoba. A 1000 kN capacity closed loop MTS machine was used to apply the load. The beams were tested in a cantilever mode as shown schematically in Figure 5.4A. The test set-up provide the most critical state of stress that a reinforcement would experience, where maximum bending and shear stresses were induced at the same section. To enhance flexural failure in the reinforced concrete beams and impede shear failure, the length of the beam was increased during testing using a hollow structural steel section (HSS, Figure 5.4A). The HSS was 1020 mm long and had a cross section area of 102x102 mm.

Table 5.3. Test Conditions (treatments)

Structural element	Conditions (contact of the elements with manure ; duration)
Steel bars	Submerged, wet/dray cycle; t = 0, 4, 8, 12, 18 months
GFRP C-Bar	Submerged, wet/dray cycle; t = 0, 4, 8, 12, 18 months
GFRP ISOROD	Submerged, wet/dray cycle; t = 0, 4, 8, 12, 18 months
OC* / Steel rebar	Partially submerged, wet/dray cycle; t = 0, 4, 8, 12, 18 months
OC / GFRP ISOROD	Partially submerged, wet/dray cycle; t = 0, 4, 8, 12, 18 months
GFRP spray / OC/ Steel rebar	Partially submerged, wet/dray cycle; t = 0, 4, 8, 12, 18 months
PVC / OC/ Steel rebar	Partially submerged, wet/dray cycle; t = 0, 4, 8, 12, 18 months

A steel plate with a 400 mm width was used to impede slippage of the reinforcement. Monotonic static loading was applied at a distance of 20 mm from the edge of the HSS to increase the applied moment while keeping the shearing force constant. The load was applied under stroke control with a rate of 2 mm/min up to failure. The laboratory setup is shown in Figure 5.4B. Displacements and strains were recorded for the duration of the tests.

The reinforcement bars were tested in axial tension using steel pipe of 19 mm nominal diameter filled with epoxy resin as anchorage. The steel pipes used were 300 mm long as shown in Figure 5.5A. The anchorages were gripped using the gripping system of the testing machine as shown in Figure 5.5B.



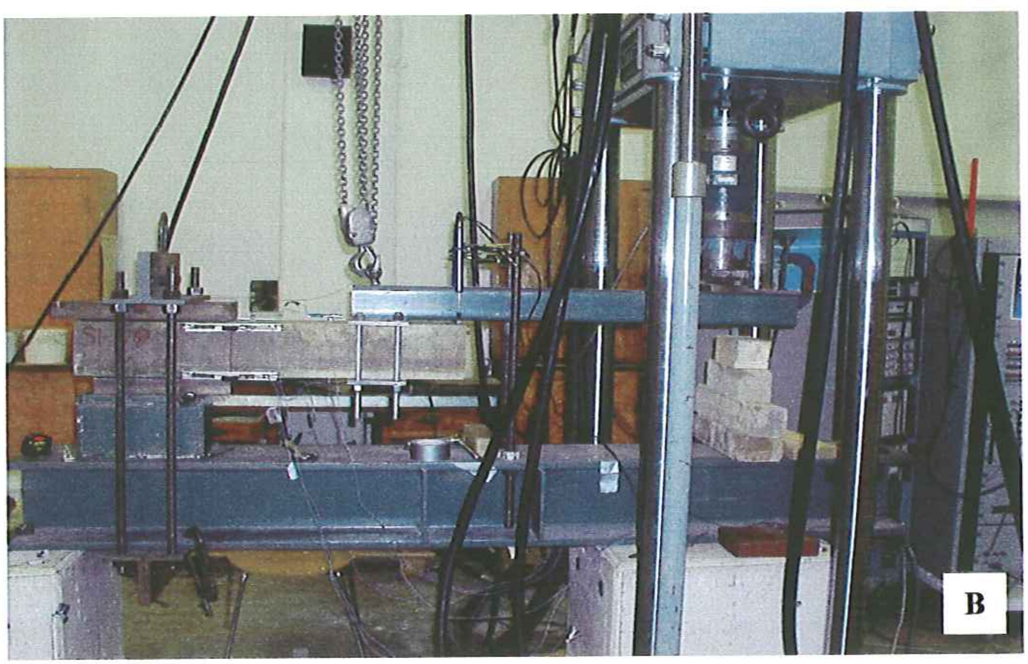


Figure 5.4. Testing scheme. (A) Test setup diagram and (B) Laboratory test setup.

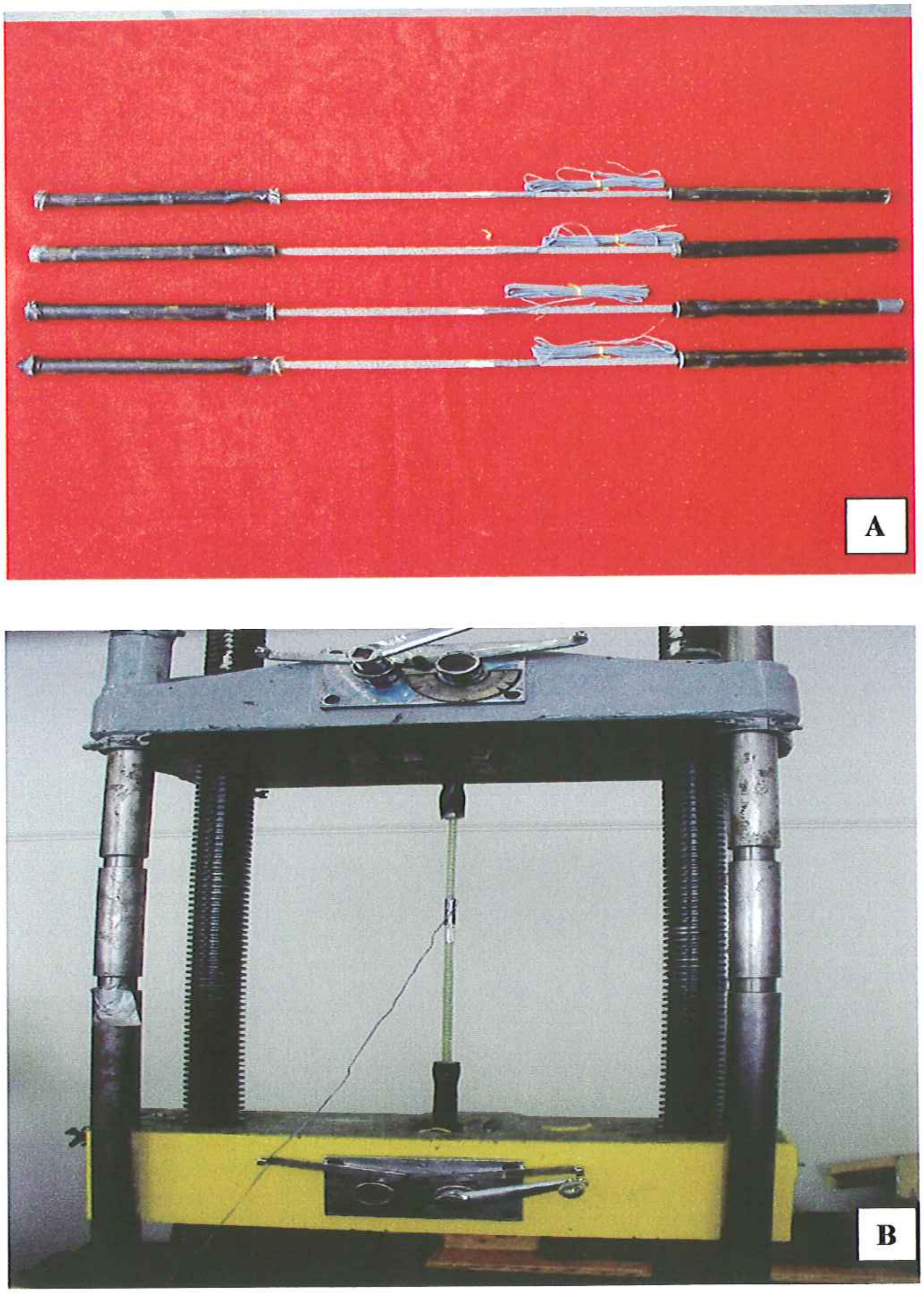


Figure 5.5. Tension test setup showing: A) The location of steel pipes and B) System for the C-BAR reinforcement bar specimen being tested in tension.

5.5 Instrumentation

The instrumentation used to monitor the behavior of the reinforced concrete beams and reinforced bars during the test consisted of a combination of linear variable differential transducers (LVDTs), PI gauges, and strain gauges. Both the LVDT and the PI gauges were calibrated before the test.

The maximum deflection at the free end of the cantilever was measured, from the top surface of the reinforced concrete beam, using two LVDTs. Two LVDTs were attached to the ends of the reinforced concrete beams to measure the slip of the exposed reinforcement bar.

The PI gauges located on the top and side of the reinforced concrete beams were used to measure the tensile strain and, consequently, the crack width. Placing the PI gauges at the bottom of the beam also monitored the compression strain behavior of the beams. The location of the instrumentation during the tests is shown in Figure 5.4A.

Strain gauges were mounted on each of the 80 reinforcement bars tested at the center of the bar to measure the strain in the longitudinal direction as shown in Figures 5.4. The strain gauges had a 5-mm length and 120 ohms electrical resistance.

The LVDTs, PI gauges and the strain gauges were connected to a computer-controlled 32-channel data acquisition system to monitor and record the loading deflection and strain in the reinforced concrete beams and stress-strain behavior of reinforcement bars.

6 CASTING OF SPECIMENS

The concrete mix used in the preparation of the specimens was design for a nominal strength of 25-35 MPa at 28 days. The mix proportions are given in Table 6.1.

Table 6.1. Concrete Mix Proportions

Material	Quantity
Type 10 cement	260.0 kg
Sand	930.2 kg
Coarse aggregates	1025.0 kg
Water	142.0 kg
Air	4.0 %
Water reducing admixture	832 mL
Superplasticizer	65 mL
Slump	110 mm
Water/cement ratio	0.55

The target compression strength of the concrete used in the experiment was about 25-35 MPa. However, this mix prepared produced concrete of a higher strength, i.e., 42 MPa.

6.1 Concrete Specimens

The concrete specimens were fabricated at the Lafarge Canada Inc. Construction Materials Group, Precast Division in Winnipeg. Special forms were constructed from wooden or PVC sections to account for the required shape and the size of the concrete specimens. After casting, the concrete specimens were cured at Lafarge Canada Inc. Quality Control Department in a conditioned room at (temperature 22 °C ± 3, and relative humidity 100%) until the specimens were placed in contact with manure.

The beams were cast according to the designed establish in the experimental program; a nominal cross-section of 170 mm x 170 mm and a total length of 1000 mm.

Four series of beams have been cast: A) Ordinary Concrete (OC) / Steel rebar, B) Ordinary Concrete / GFRP ISOROD rebar, C) GFRP spray / Ordinary Concrete / Steel rebar and D) PVC / Ordinary Concrete / Steel rebar.

Casting details for the reinforced concrete beam for series A, B and C are shown in Figure 6.1. After casting, the beams were covered and left place for 24 hours after which they were demoulded and transferred to the curing room. The remaining 20 reinforced concrete beams (Series D) were covered with PVC plates on the four vertical sides and the bottom face. Casting and reinforcement details for the concrete beam cover with PVC are shown in Figure 6.2.

To prevent the PVC forms from bloating during casting a wood frame was build around the PVC forms. After curing for 48 hours in air, the reinforced concrete beams with the PVC form left in place were transferred to the curing room.

All reinforced concrete beams, were kept in curing room until they have been placed in contact with manure with exception of the 20 beams that have shipped to University of British Columbia for spraying with GFRP after 60 days of curing.

The uncovered beams (Scheme A) have been used to evaluate the effectiveness of steel and GFRP as the internal reinforcing element in a hog manure environment.

The covered specimens (Scheme B and Scheme C) have been used to determine the behavior and the protective capacity of the PVC cover and GFRP spray cover as external materials to protect the reinforced concrete of manure storage tanks. Changes in the structural component properties with time under hog manure environmental conditions have been assessed through changes in flexural behavior as well as in the physico-chemical composition.

Figure 6.1. Casting the reinforced concrete beams for A, B, and C series

Figure 6.2. Casting the reinforced concrete beams covered with PVC, D series.

6.2 Spray Concrete Specimens with GFRP

The spray was done in collaboration with Department of Civil Engineering, University of British Columbia. The GFRP spray composite used in coating the specimens was developed at the University of British Columbia under the auspices of Network of Centers of Excellence – ISIS program.

A set of 20 reinforced concrete beams have been sprayed with GFRP composite (Figure 6.3). The composite consists of short, randomly distributed E-glass fibers embedded in a polyester matrix. The GFRP composite is sprayed on the surface of the concrete at a high speed and compacted pneumatically on the application surface. The sprayed concrete specimens have been used to assess the protective capabilities of sprayed GFRP with time under hog manure environmental conditions.

Figure 6.3. Reinforced concrete beams sprayed with GFRP composite.

6.3 Site Experiment and Containment Units

Twelve specially design containment units in which the concrete specimens and the reinforcement bars are being kept in contact with the liquid hog manure have been constructed. The containment units were design to accommodate 212 specimens; they consist of 2.5m x 1.7m x 0.5m wooden boxes, lined with a heavy-duty plastic liner resistant to the corrosive effect of the hog manure (Figure 6.4). The liner will effectively contain the hog manure and will prevent any leakage. The containment units accommodate ~1200 L of manure per each experimental cycle.

The large scale experimental tests included in the program have been carried out at the experimental site established at Glenlea Research Station Manitoba in a restored barn (Figure 6.5).

The barn accommodates twelve specially designed containment units (Figure 6.6) in which the concrete specimens and the reinforcement bars have been exposed to the liquid hog manure. In ten of these containment units, reinforced concrete structural elements (concrete – reinforcement combinations) were placed in contact with liquid hog manure for predetermined lengths of time. The remaining two containment units accommodated the reinforcement bars.

The test conditions as described in Chapter 5.3 were carefully monitored and maintained over the entire duration of the project.

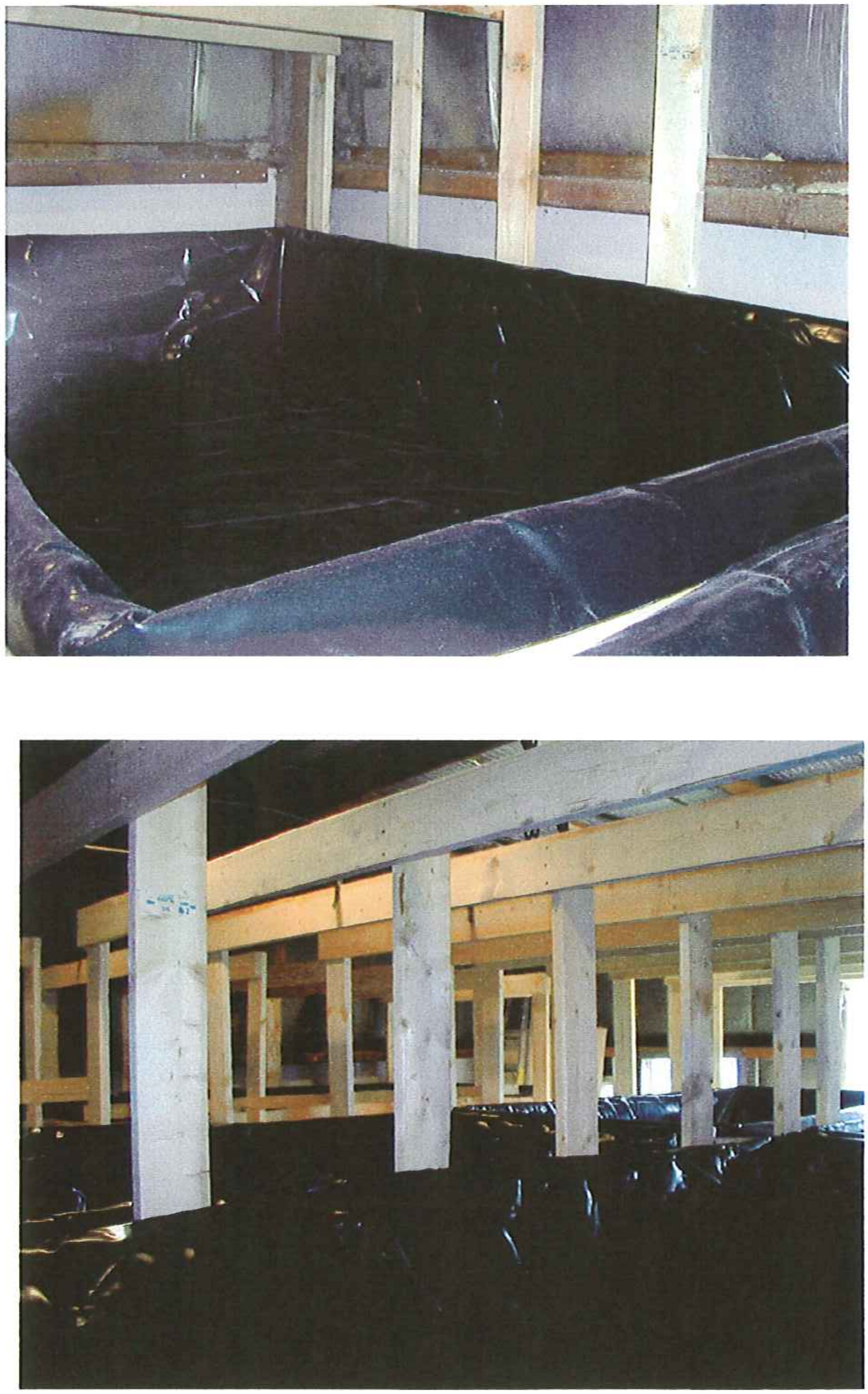


Figure 6.4. Containment units lined with heavy-duty plastic liner

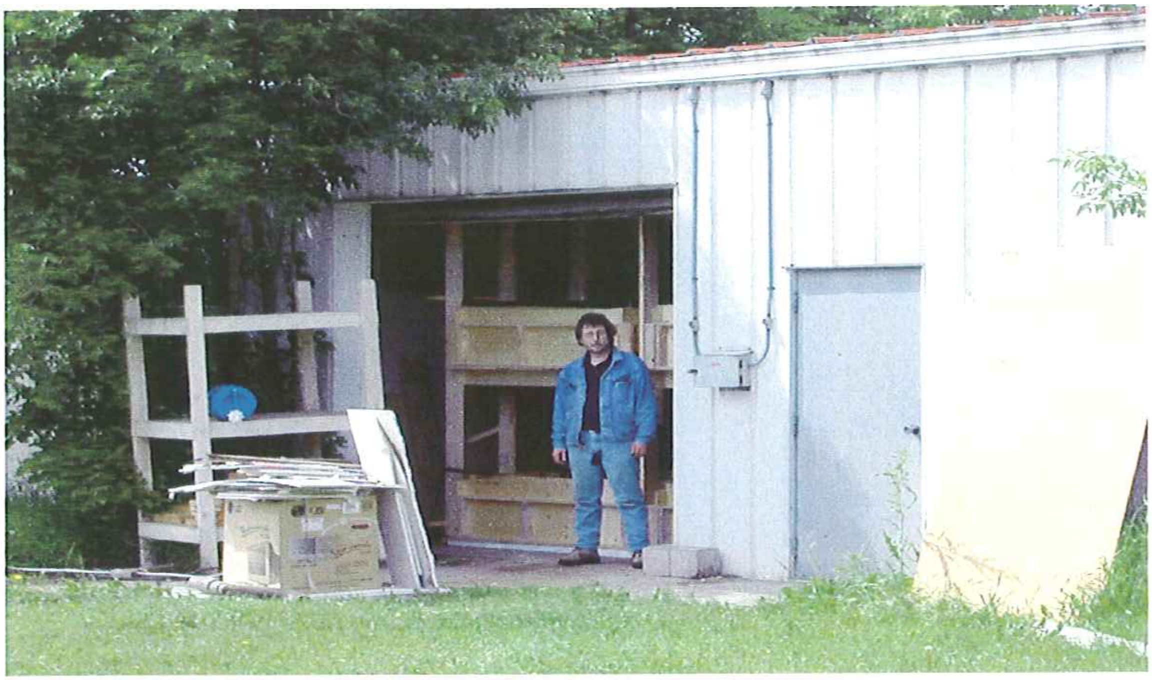


Figure 6.5. The experimental site at Glenlea Research Station Manitoba.

7 RESULTS AND DISCUSION

Fifty-two reinforcement bars and 62 reinforced concrete beams were tested after they had been exposed to a manure environment for 0 (control), 4, 8, 12 and 18 months to determine the changes in their mechanical properties with exposure time. In addition, systematic microstructure analyses of the reinforced concrete were made to examine the results of chemical attack and physical degradation due to the environmental exposure conditions; factors controlling the initial microstructure development are strongly related to reinforced-concrete durability. Frequent quantitative analyses of the manure were also made to identify changes resulting from contact with the concrete structural elements.

Mechanical testing was conducted at W.R. McQuade Structural Laboratories at the University of Manitoba.. A 1000 kN capacity closed loop MTS machine was used to apply the load. The beams were tested in a cantilever mode as shown schematically in Figure 5.3A, Chapter 5.

7.1 Monitoring the Experimental Conditions

The environment of manure storage tanks presents certain hazards to reinforced concrete. Many constituents of manure react with the components of its structural elements (i.e., concrete and reinforcement). The rates of these reactions (i.e., leaching, carbonation, and corrosion) result in significantly reduced service lifetimes for such structural elements.

Over the duration of the experiment, the experimental conditions were precisely controlled. The manure in contact with the reinforced concrete structural elements was changed every two weeks. The effect of cementitious materials in the concrete on the chemistry of manure in its immediate vicinity could be significant under static conditions. These frequent changes of the manure in contact with the concrete structural elements prevented the rise in pH and the increased concentration of concrete decay products, maintaining the relatively high reactivity of the manure with respect to concrete. The pH was measured and the changes in the chemical composition of the manure were quantitatively analyzed. Manure analyses were performed at Norwest Laboratories in Winnipeg. Analyses were performed on as-received new manure as well as on the manure that had been in contact with the structural elements. The manure samples were quantitatively analyzed using ICP atomic emission spectrometry Method 3120B and Titration Method 2320 B. Examples of results of the analyses performed during the first 4 months exposure (0 - 4 months) and the last 6-months exposure (12-18 months) on the manure as-received and after two weeks in contact with the concrete structural elements are presented in Table 7.1 and 7.2. Attention was paid to chemical species that could affect the performance of the structural elements, such as: CO_3^{2-}, HCO_3^-, Cl^-, Mg^{2+} Ca^{2+}, Si^{4+}, K^+ and Na^+. Particular attention was given to Ca^{2+} and Si^{4+} . These elements were chosen because they reflect the leaching characteristics of two major phases in cement paste: 1) calcium hydroxide [$Ca(OH)_2$]; and 2) calcium silicate hydrates (C-H-S).

Overall, the results show a relatively large variation in the concentrations of the elements in the manure supplied at two-week intervals. This variation could be attributed to factors such as the animal diet, productivity and management (i.e., dilution). In real system operation, variation of manure element concentrations should be expected. These variations would affect the aggressiveness of the manure, the rate of concrete degradation and corrosion of reinforcement as well as the type of reaction that will take place between concrete and manure.

The results of the analyses indicated that the concentration of calcium in received manure ranged between 1.25 $x10^{-3}$ and 5.50 $x10^{-3}$ mol L^{-1}. The calcium solubility limit in water in equilibrium with both $Ca(OH)_2$ and C-S-H is approximately 0.02 mol L^{-1} at 25 (C. At a value of 10^{-3} mol L^{-1}, the bulk solution concentration of Ca^{2+} measured in received manure is below the equilibrium value.

Table 7.1. Chemical composition of manure used in the experimental program (0-4 months)

Compound	Units	As received manure	After two Weeks in contact with structural elements
Moisture	%	99.1 – 99.6	99 – 99.4
Total –Alkalinity	mg/L	4880 -7000	5800 –7444
Bicarbonate	mg/L	5940 - 8530	7070 – 9070
Carbonate	mg/L	< 5	<5
Hydroxide	%	< 5	<5
Calcium	mg/L	170 - 220	140 –180
Chloride	mg/L	600 -800	1000 –1400
Iron	ppm	<8 - 12	<10 - <8
Magnesium	mg/L	0.005 - 0.008	10 – 30
Phosphorous	mg/L	120 -180	70 – 80
Potassium	mg/L	850 - 1170	0.146 - 0.162 – 0.198
Silicon	ppm	135 - 153	145 - 278
Sodium	mg/L	380 - 470	550 – 740
Sulphate-S	mg/L	70 - 110	100 - 130
PH		7.3 –7.5	7.7 – 8.0

Calcium was in an under-saturated condition. Analyses of the manure in contact with the reinforced concrete specimens indicate relatively small variation in concentrations of most of the elements existing in manure. A slight decrease in the concentration of calcium in the manure in contact with reinforced concrete was observed over the entire duration of the experiment. The variation of calcium concentration after two weeks in contact with structural elements during the first 4 and last 6 months of exposure ranged from 3.50×10^{-3} to 5.50×10^{-3} mol L^{-1}, and from 2.25×10^{-3} to 3×10^{-3} mol L^{-1}, respectively. The decrease in Ca^{2+} was attributed to precipitation.

Table 7.2. Chemical composition of manure used in the experimental program (12-18 months)

Compound	Units	As received manure	After two Weeks in contact with structural elements
Moisture	%	99.5 – 99.6	99.5
Total –Alkalinity	mg/L	1410 – 4940	3450 –7990
Bicarbonate	mg/L	4240 - 4390	4210 - 4970
Carbonate	mg/L	< 6	<6
Hydroxide	%	< 5	<5
Calcium	mg/L	50 - 140	90 -120
Chloride	mg/L	400 - 500	400 – 600
Iron	ppm	< 6 - 12	< 6 - < 10
Magnesium	mg/L	60 - 100	30 - 120
Phosphorous	mg/L	30 - 160	30 - 160
Potassium	mg/L	720 - 770	720 - 910
Silicon	ppm	81 - 106	70 - 154
Sodium	mg/L	340 - 450	340 - 430
Sulphate-S	mg/L	20 – 80	20 - 70
PH		7.5 - 8.5	7.3 – 8.1

Calcium in the manure more likely reacted with elements present in manure and in the concrete (i.e., bicarbonates, carbonates, silica, and sulfates) forming new phases such as calcium carbonate, calcium hydroxide, and calcium silicate hydrates. Large decreases in concentrations of elements present in manure due to precipitation could induce changes in the reactions of the concrete with the manure. However, the concentration of chloride was observed to increase substantially during the first 8 months. Its concentration increased from between 600 and 800 mg L^{-1} to between 1000 and 1400 mg L^{-1}. Chloride may be introduced in concrete as an accelerating agent for hydration of Portland cement (i.e., calcium chloride) or through water-reducing agents which may contain small amounts of calcium chloride

($CaCl_2$) to offset the set-retarding effect of water reducer. Because of the increased awareness of the deleterious effects of chloride, the occurrence of additional $CaCl_2$ has decreased significantly in current concrete mixes. The increase in concentration of chloride in the manure in contact with concrete specimens is most likely due to evaporation.

Analysis results indicated that the pH in as-received manure ranged between 7.3 and 8.5. No significant changes in the pH were observed; after two weeks in contact with the concrete structural elements, the pH of the manure ranged from 7.3 to 8.1. Typically, the pH of concrete is dominated by two factors: the solubility of $Ca(OH)_2$ and calcium-rich C-S-H, and soluble alkali content. The first factor not only generates a high pH, circa 12.4 at 20 °C, but also provides buffering in the long term. However, alkalis such as NaOH (sodium hydroxide) and KOH (potassium hydroxide) will raise the pH above the buffer limit, in excess of 13.5. Both NaOH and KOH are not well bound into solid hydrates, are the most soluble components and unlike portlandite ($Ca(OH)_2$), the dissolution of these phases is not limited by their solubility but by their physical access to the water. The way in which the pH in manure changes with time is determined by the way in which the water-soluble species (i.e., $Ca(OH)_2$, NaOH and KOH) from the concrete become available for leaching.

The data presented in Tables 7.1 and 7.2 suggest that changing the manure regularly prevents the accumulation in the manure of species leached out from the concrete. In these conditions, in the experiments, saturation with respect to the major concrete components such as Ca^{2+} and Si^{4+} is not achieved and the pH remains close to its initial value. The accumulated amount of species leached from the concrete seems to be too small to alter the manure's reactivity with respect to concrete. This is because the concrete is in contact with a given batch of manure for only a short period of time (two weeks).

7.2 Reinforcement Bars

Three types of reinforcement bar have been investigated: (1) steel, (2) GFRP ISOROD and (3) GFRP C-BAR. Fifty-two reinforcement bars were tested in axial tension after exposure to a manure environment for 0 (control), 4, 8, 12 and 18 months. Changes in the mechanical properties such as: yield strength, ultimate strength, and modulus of elasticity of the reinforcement bars due to exposure was investigated. The stress-strain behavior for each type of reinforcement bar is discussed below. The experimental results were benchmarked against control specimens.

7.2.1 Steel Reinforcement Bars

The tensile test results for the steel reinforcement bars after 0 (control), 4, 8, 12, and 18 months exposure to manure environmental conditions are summarized in Table 7.3.

Table 7.3. Summary of experimental results of mechanical tests on steel bars after 0 (control) 4, 8, 12 and 18 months exposed to manure environment.

Time Exposure (months)	Yield Strength (MPa)	Ultimate Strength (MPa)
0 (control)	505 – 521	607 - 631
4	470 - 495	560 - 587
8	415 -457	455 – 522
12	420 / *	430 - 465
18	*	353 - 392

* bars did not reached yielding strain.

The experimental results indicate that both the yield strength and ultimate strength of steel reinforcement bars in contact with manure decreased continuously with exposure time. The yield strength after 12 months exposure decreased by about 18% compared with control specimens. Starting with 12 months exposure, the bars failed before reaching the yield strain. The yield strength decreased at an accelerated rate during the second four-month period, about 9%, compared with the first four-month period, about 6%. However, it should be mentioned that three out of 4 bars exposed to manure for 12 months and all specimens exposed to manure for 18 months failed before reaching the yield strain.

A significant reduction was also observed in ultimate tensile strength. The average ultimate tensile strength of bars in contact with manure for 18 months was 373 MPa, a decrease of 252 MPa, about 40 % compared with the control specimens. The decrease in the ultimate strength during the first four months was about 8%. In the next four months (4- 8 months) the decline increased to 14 %. In the next four months (8 - 12 months), the decrease in the ultimate strength was reduced compared with the previous period but remained high at about 9%. In the last six months of the experiment (12 - 18months), the decrease in the ultimate strength was 17%. It appears that the corrosion rate was high at all times, but in particular during the 4-8 months period. The decrease in the corrosion rate after 8 months exposure may be due to the decrease in the diffusion rate of both oxygen and moisture (two main controlling factors of corrosion) through the corrosion products formed at the corrosion site.

Figure 7.1 illustrates typical changes in ultimate strength, yield strength and elastic modulus with exposure time. The stress-strain in tension results after 18 months exposure are compared to the stress-strain curves in tension for the control specimen and after 4, 8 and 12 months exposure. The experimental results indicate that both the yield strength and ultimate strength of steel reinforcement bars decreased significantly after 18 months of contact with manure. All specimens exposed to manure for 18 months failed before reaching the yield strain. It should be mentioned that after 18 months of contact with manure, on average the steel bars lost about 40% of their ultimate strength.

The observed decreases in yield strength and ultimate tensile strength of the steel bars are due to advanced corrosion (i.e. localized corrosion and/or general corrosion). This led to a substantial decrease in diameter of the steel bars in various places and consequently to a significant mechanical weakening. Figure 7.2 shows the appearance of the steel bars after exposure to manure.

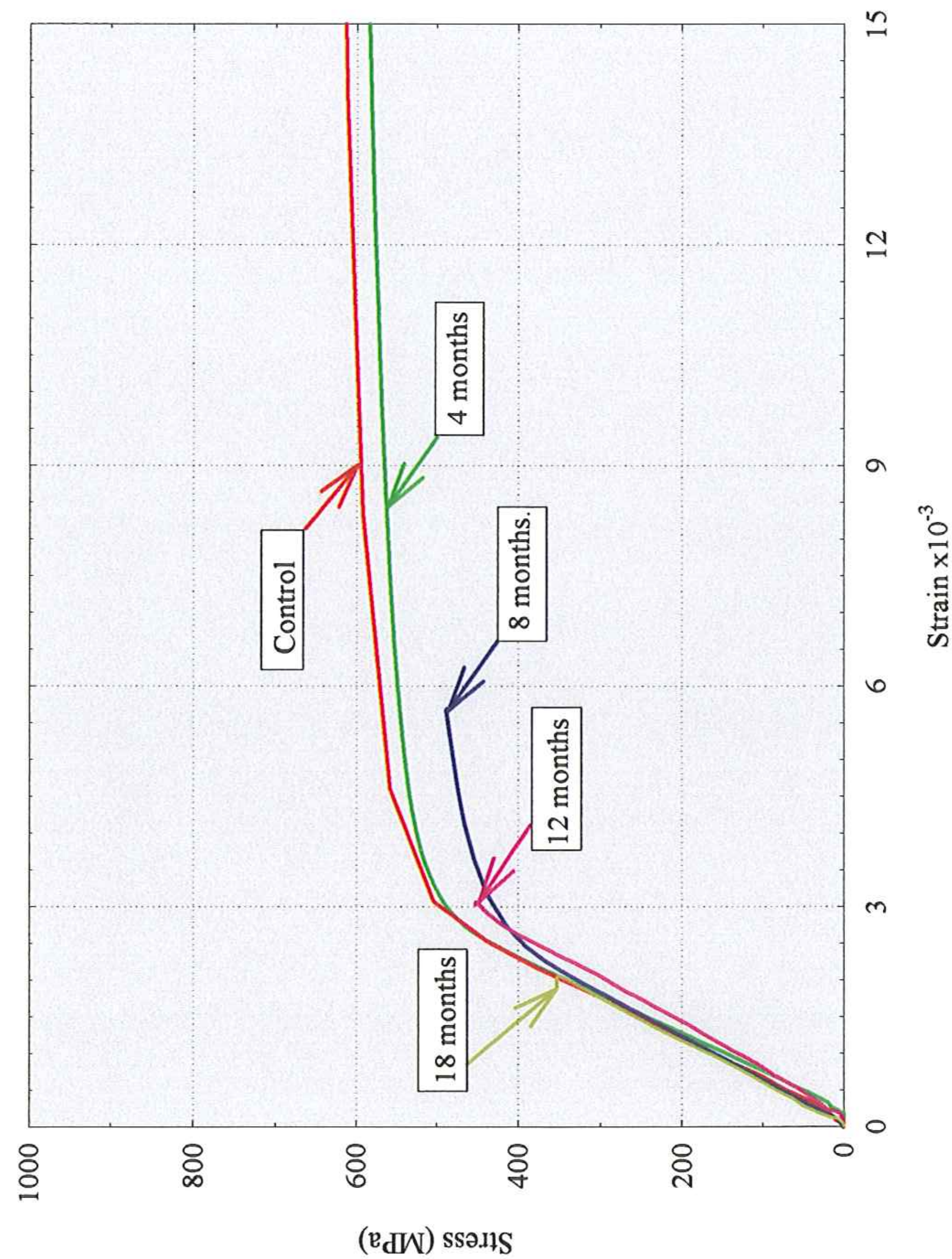
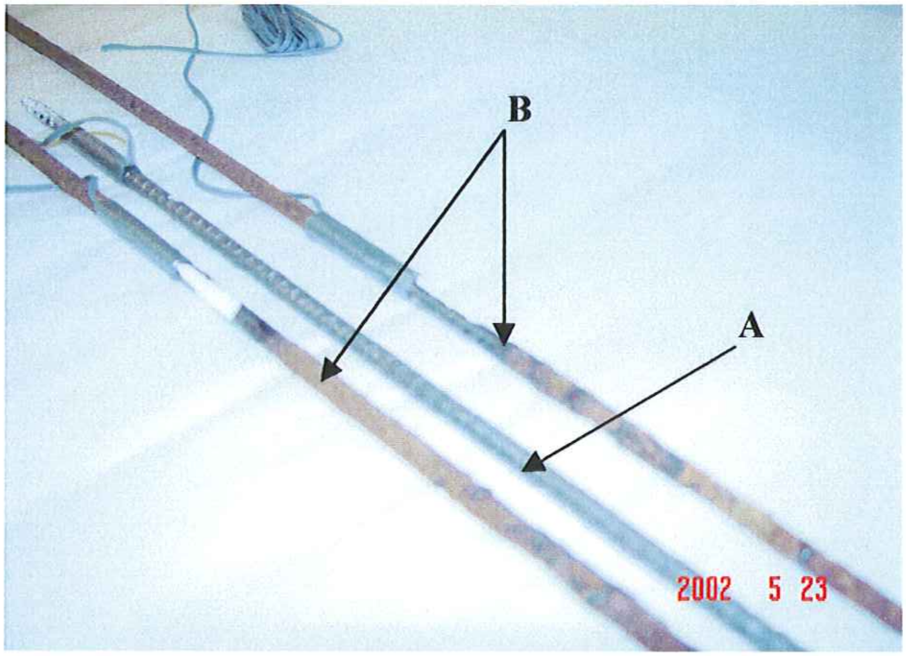


Figure 7.1 The effect of exposure time on stress-strain behavior of reinforced steel bars.



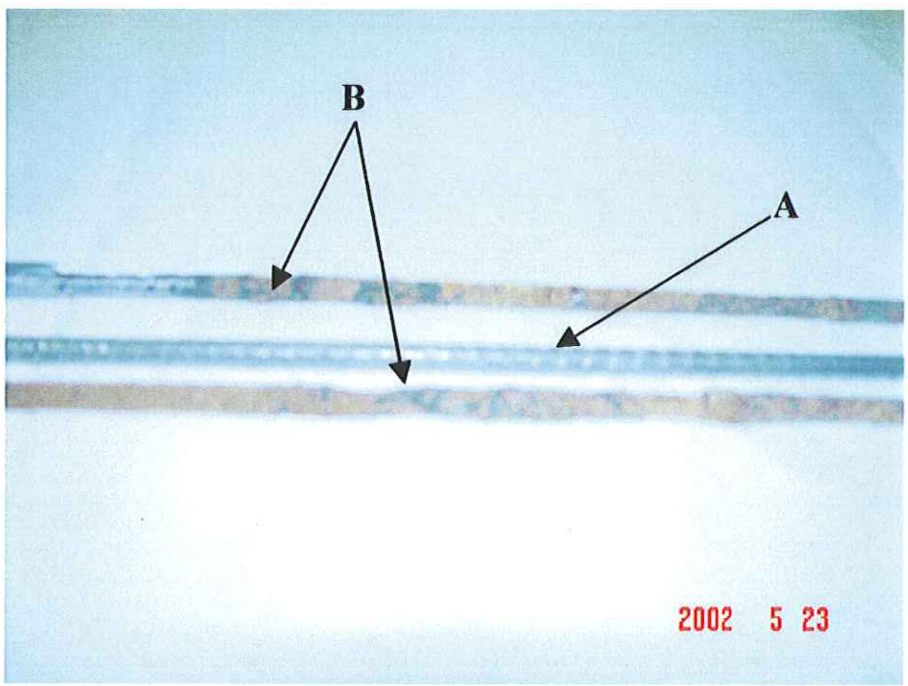


Figure 7.2. Reinforcement steel bars appearance A) control and B) after 18 month in contact with manure.

The manure environmental conditions (i.e., pH, wet/dry cycles and high chloride concentration) favored the continuation of corrosion of the steel bars observed after 4, 8 and 12 months of exposure. Microscopic examination of the selected specimens revealed that the corroded areas were covered with a very thick crust (Figure 7.3) immediately after they had been exposed to the manure environment. Analysis of this crust using energy dispersive X-ray analysis (EDXA) revealed that the crust consists mainly of corrosion products such as FeO $(H_2O)x$ (Figure 7.3A) and a series of elements present in the manure such as K^+ and Cl^- (Figure 7.3B). The elements present in the manure were adsorbed on the corrosion product on the surface of the bars. Examination also revealed that the uniform corrosion (general corrosion) after the first 4 months exposure becomes localized corrosion (pitting corrosion) thereafter. The presence of the crust on the surface of the steel bars induced local variations in the electrochemical potential and consequently the development of localized corrosion. The pits formed in the steel bar surface act as sites of stress concentration. Localized corrosion, generally characterized by much more rapid corrosion rates compared with general corrosion, will shorten significantly the service life of these structural elements.

Corrosion of steel is an electrochemical process. Electrochemical potentials which form the corrosion cells were generated by changes in the surface characteristics of the steel reinforcement due to crust formation and/or due to differences in concentration of dissolved ions (oxygen, chloride and alkalis) in the manure environment.

Differences in concentration of dissolved ions always will be present in a tank wall. For example, during the filling of the tank, part of the inner surface of the tank is in contact with manure and another part is in contact with air. The outer surface is permanently in contact with air. The concentrations of alkalis, chlorides and oxygen, obviously, are different at the portions of the wall surface in contact with liquid manure, solids and air. In the present experiments the variation of concentration of alkalis, chlorides and oxygen were accentuated by the two-week cycle of exposure to manure and two-week cycle exposed to air, as well as by changing the manure periodically.

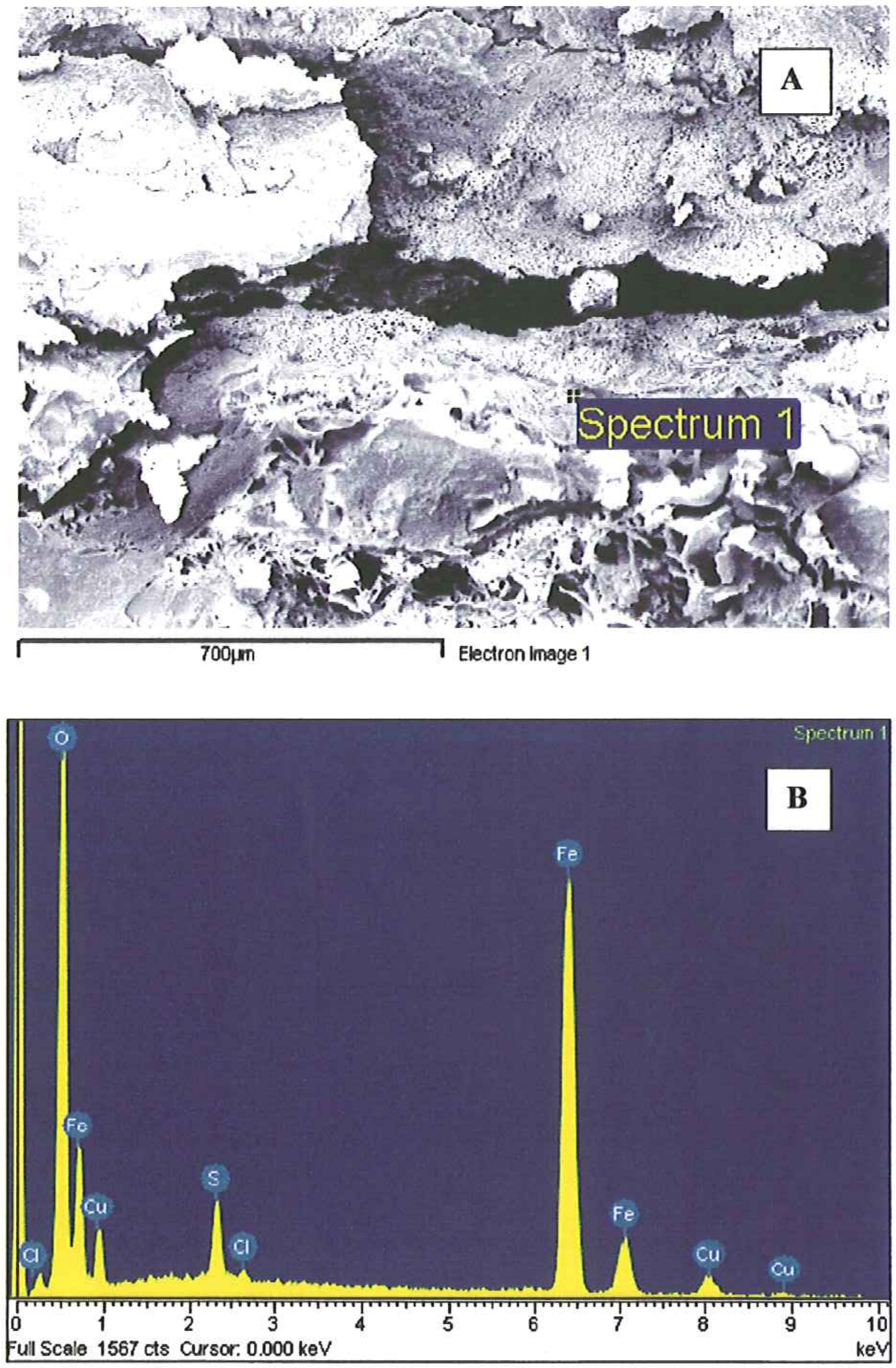


Figure 7.3. Corrosion products formed on the surface of the steel bars in contact with manure: A) morphology of corrosion products and B) chemical composition of corrosion products.

The results suggest that the mechanism of corrosion of steel in manure is characterized by a gradual eroding away or alteration by chemical and/or electrochemical oxidizing processes. In general, the corrosion modes can be classified into five groups: 1) general corrosion, 2) localized corrosion (pitting and crevice corrosion), 3) environmentally induced cracking (stress corrosion cracking), 4) metallurgically influenced corrosion, and 5) mechanically assisted degradation (erosion and corrosion fatigue). All five modes are important and they always operate simultaneously. In a given circumstance, any mode of corrosion can be the dominant deterioration process. Microscopic examination of selected steel bar specimens after exposure to the manure environment revealed that a combination of two potentially dominant corrosion modes, general corrosion (at the early exposure times) and localized corrosion (pitting corrosion) contributed to the deterioration of the steel reinforcement (Figure 7.2). However, stress corrosion cracking may have some impact in real situations; in this case, the damage caused by stress and corrosion acting together may exceed that produced when they act separately.

The presence of relatively low pH (7.3 - 8.5), high concentration of chloride (400 – 800 mg L^{-1}), the rapid wet/dry cycle and high RH (60 % -85%) during the experiment promoted and maintained the observed advanced corrosion in the steel bars in a relatively short time (18 months exposure).

The Pourbaix's diagram suggests that iron is in a passivation state at a pH value in the range of 8 - 12. This passive layer can be disturbed and localized corrosion can be initiated. The pH threshold value of about 9.5 for initiation of pitting corrosion varies, decreasing with rising temperature and increasing with rising Cl^- concentration. At the manure's pH in the range 7.3 - 8.5 and in the presence of chloride the oxide film (i.e., hydrate ferric oxide, FeOOH and/or ferric oxide, Fe_2O_3) that protect (passivate) the steel from corrosion, loses its protection capabilities and corrosion is initiated.

In summary, corrosion of steel reinforcement bars can be initiated and maintained in a manure storage tank under two broad sets of conditions; 1) high pH (alkaline) conditions in the presence of chloride ions, or 2) low pH (acidic) conditions in the absence of chloride ions. In either condition, iron, oxygen and water must be present in order for corrosion to occur. Although the presence of water is necessary, excess water can limit corrosion severely (insufficient oxygen present). However, the tank wall is exposed cyclically to wet/dry conditions and significant localized corrosion can occur.

7.2.2 GFRP Reinforcement Bars

Summary results of mechanical tests performed on GFRP ISOROD bars are presented in Table 7.4.

Table 7.4 Summary of experimental results of mechanical tests on ISOROD bars after 0 (control), 4, 8, 12 and 18 months exposed to manure environment.

Time Exposure (months)	Elastic Modulus (GPa)	Ultimate Strength (MPa)
0 (control)	42 – 43	540 - 646
4	37 - 39	552 - 615
8	38 - 39	503 – 536
12	37 – 40	465 - 548
18	33 - 37	492 - 505

A decrease in both modulus of elasticity and the ultimate tensile strength was observed during each interval (Table 7.4). On average, the modulus of elasticity and ultimate tensile strength of the ISOROD bars after 18 months exposure to manure decreased by about 17 % and about 19 %, respectively, compared with the control specimens. It appears that decreases in elastic modulus took place mainly during the first four months of the experiment (0 - 4 months) and in the last 6 months of the experiment (12 - 18 months). Very little change in modulus of elasticity took place in the second and third period (4 - 8 months and 8 - 12 months). The ultimate tensile strength decrease rates also diminished with time. The overall decrease in the ultimate strength in 18 months of the experiment was 19%. About three quarters of this decrease took place in the first two periods of the experiment, i.e., 0 - 4 months and 4 - 8 months, half of this decrease took place in the second period (4 - 8 months). In the last six months of exposure to manure the ultimate tensile strength decreased by only 3%.

The effect of exposure time on the stress-strain relationship for ISOROD bars is shown in Figure 7.4. The results at 18 months exposure are compared with the results at 4, 8 and 12 months as well as with the results from the control specimens. The results show that the typical stress-strain behavior of ISOROD bars maintained linearity at all stress levels up to the point of failure (Figure 7.4), the curves exhibiting no yielding.

The elastic modulus and ultimate strength values from the tests performed on GFRP C-BAR bars are presented in Table 7.5.

Table 7.5. Summary of experimental results of mechanical tests on GFRP C-BAR bars after 0 (control), 4, 8, 12 and 18 months exposed to manure environment.

Time Exposure (months)	Elastic Modulus (GPa)	Ultimate Strength (MPa)
0 (control)	42	829 – 924
4	40 - 41	763 – 786
8	38 - 40	702– 749
12	36 – 39	669 – 769
18	35	760

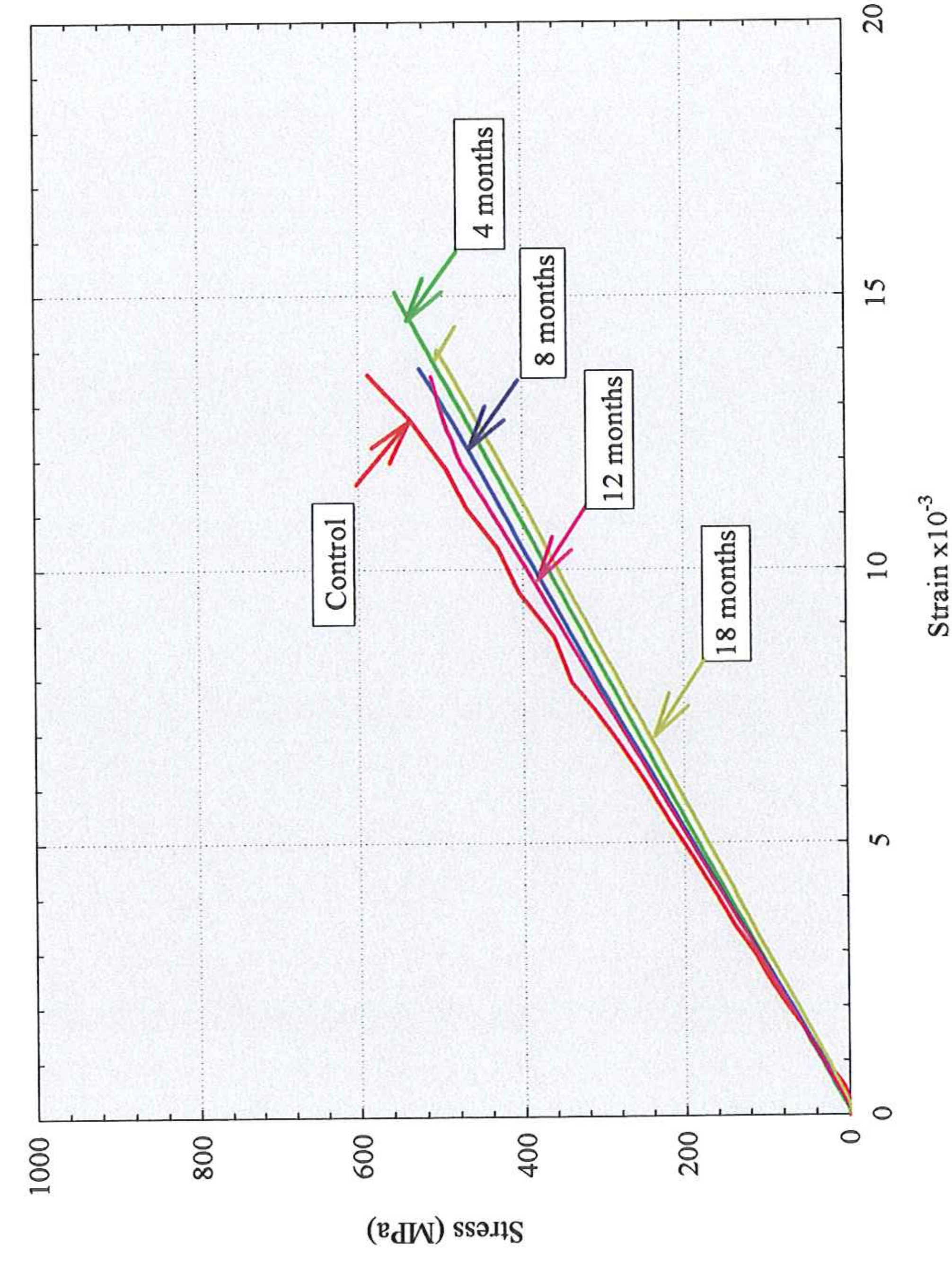


Figure 7.4. The effect of exposure time on the stress-strain behavior of reinforced ISOROD bars.

The results for the GFRP C-BAR reinforcement bars follow a similar trend as the GFRP ISIOROD bar results. However, the values for the ultimate strength of the C-BAR reinforcement (Table 7.5) are much higher than for the ISOROD (Table 7.4). For C-BAR a decrease in both modulus of elasticity and ultimate tensile strength was observed after 18 months exposure to the manure environment although the ultimate strength changed very little in the last 10 months. The average decrease in modulus of elasticity after 18 months exposure was 17%. The average decrease of ultimate strength at 18 months exposure was 18 %. However, most of the decreases in ultimate tensile strength took place during the first 8 months of exposure, a decrease by about 16%, compared with the values for the control specimens. In the last six months of the experiment, no decrease took place in ultimate tensile strength.

The effect of exposure time on stress-strain behavior of C-BAR is presented in Figure 7.5. The changes in the tensile strength and modulus of elasticity of the C-BAR reinforcement bars with exposure time (Figure 7.5) took place in much the same manner as in ISOROD bars. However, some difference between the ultimate strength values of ISOROD and C-BAR bars have been observed. The average ultimate strength at 18 months exposure for C-BAR was 711 MPa whereas for ISOROD it was 498 MPa. The difference between the strain-stress behavior after exposure (i.e., strength characteristics) of the ISOROD and C-BAR bars may be attributed to differences in their production method, as well as in the glass and polymer chemical composition.

Typical failure patterns for both GFRP bar types (ISOROD and C-BAR) are presented in Figure 7.6. The failure behavior of both GFRP bars was characterized by longitudinal surface fiber splitting/ breakage either at the center (Figure 7.6A) or close to the lower grips (Figure 7.6B), producing the typical "broom-like" fracture. The failure of the specimens occurred quite fast over a very long distance along the bar with appreciable noise.

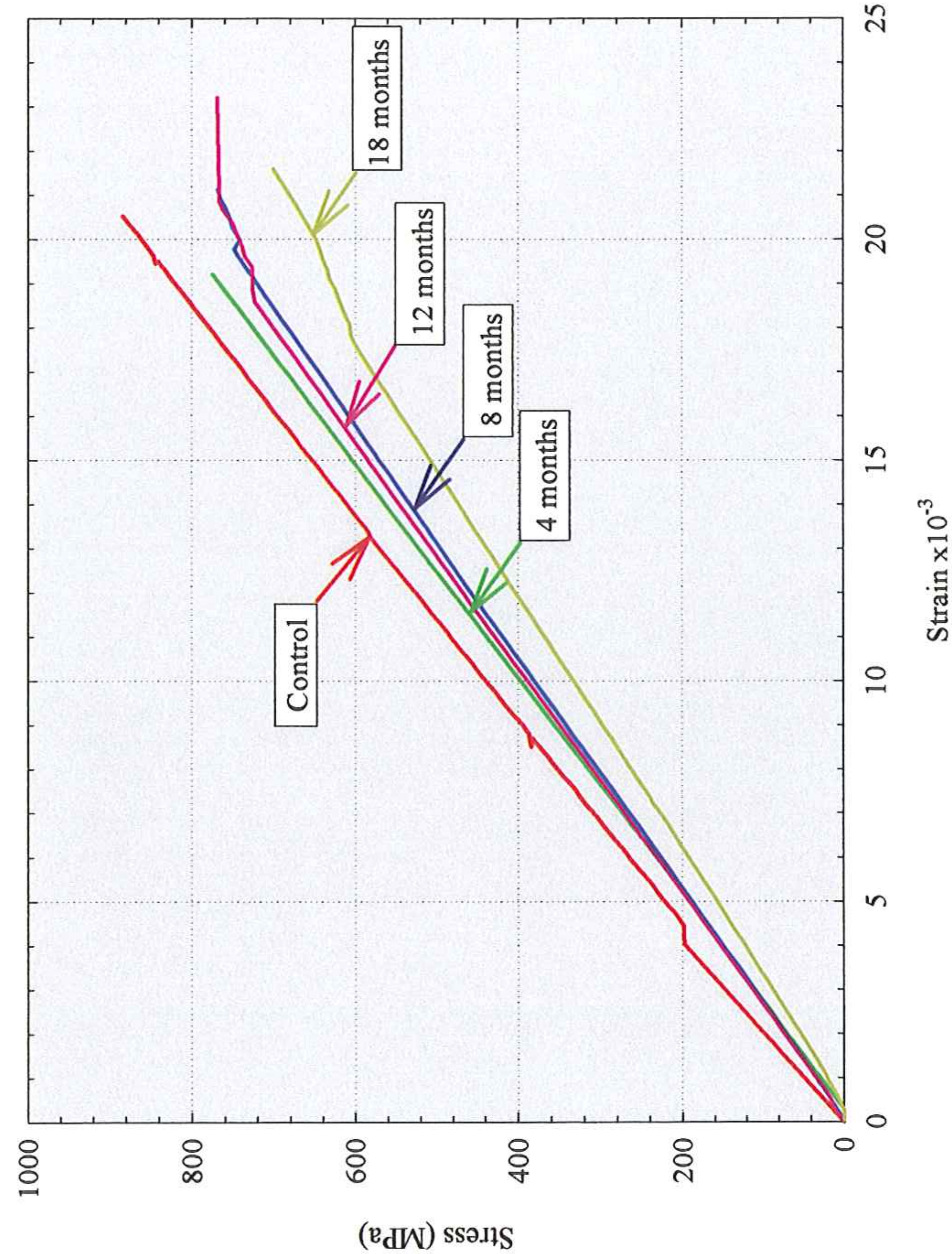


Figure 7.5 The effect of exposure time on the stress-strain behavior of GFRP C-BAR bars.

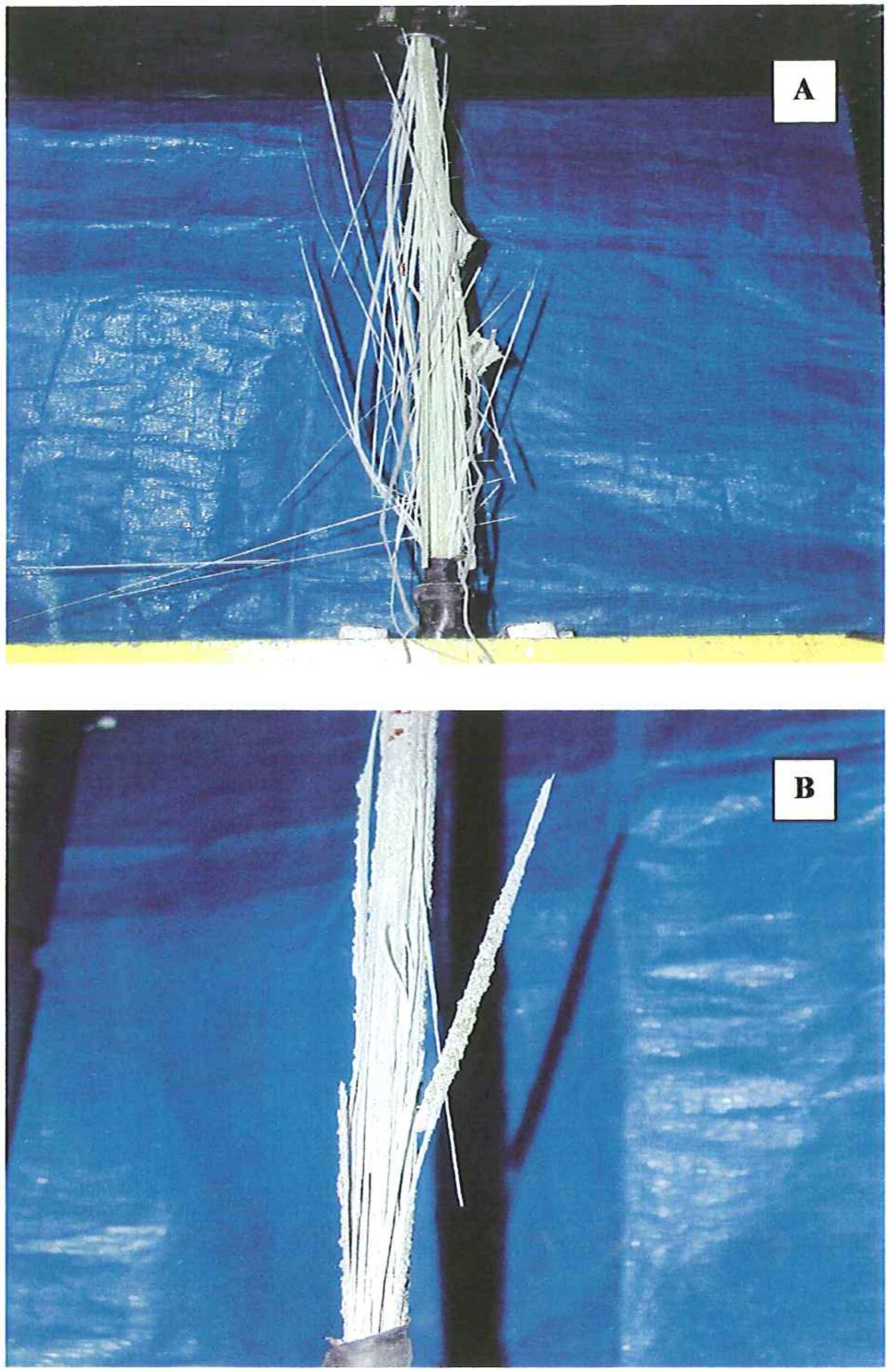


Figure 7.6. Typical failure patterns of unidirectional GFRP bars under longitudinal tension;
A) breakage at the center (C-BAR) and
B) breakage close to the lower grip (ISOROD).

The observed decreases in the ultimate tensile strength and the modulus of elasticity with time of exposure to a manure environment for both ISOROD and C-BAR reinforcement bars may be attributed to absorption of moisture by the polymer; the diffused moisture weakened the glass fiber/polymer interfacial bond strength. The mode of failure of the exposed bars supports this supposition.

Microscopic examination carried out to evaluate the morphological changes and the degree of degradation due to interaction with the manure environment on the randomly selected specimens exposed for 12 and 18 months revealed no visible changes in the specimens exposed for 12 months (Figures 7.7 and Figure 7.8). The photomicrograph of the cross section of selected GFRP - ISOROD bar and C-BAR specimens after 12 months exposure shows that the thermosetting polymer resin that binds and covers the glass-fiber strands is intact and well bonded to the glass-fiber strands. No degradation (i.e., formation of gel-like material, swelling, dissolution or blisters) of the outer surface of the bars containing polymer resin was observed. However, microscopic examination of selected specimens revealed the presence of manufacture imperfections such as polymer resin discontinuities and large amounts of debris of different sizes embedded in the outer-polymer layer as well as between glass-fiber strands. Similar features were observed in the control specimens. The debris was found to consist of the same elements that are present in the glass fiber strands (i.e., Al, Ca and Si), however in different concentration (Figure 7.8B). The debris is richer in Al and deficient in Ca. Examination of the 18 months-exposed specimens suggests some GFRP degradation only on the GFRP C-BAR bar specimens. However, the degradation area was confined only to the lugs (protrusions) which are to prevent longitudinal movement of the bars relative to the concrete surrounding the bar. This may be related to the imperfections / physical damage of the coating polymer in this area during handling. It should be noted that the imperfections /damage were not due to chemical degradation. However, they could provide a pathway for the ingress of moisture and chemicals present in manure into the C-BAR composite, leading to the initiation of degradation of the fiber-matrix interface and the fibers in these areas (Figure 7.9).

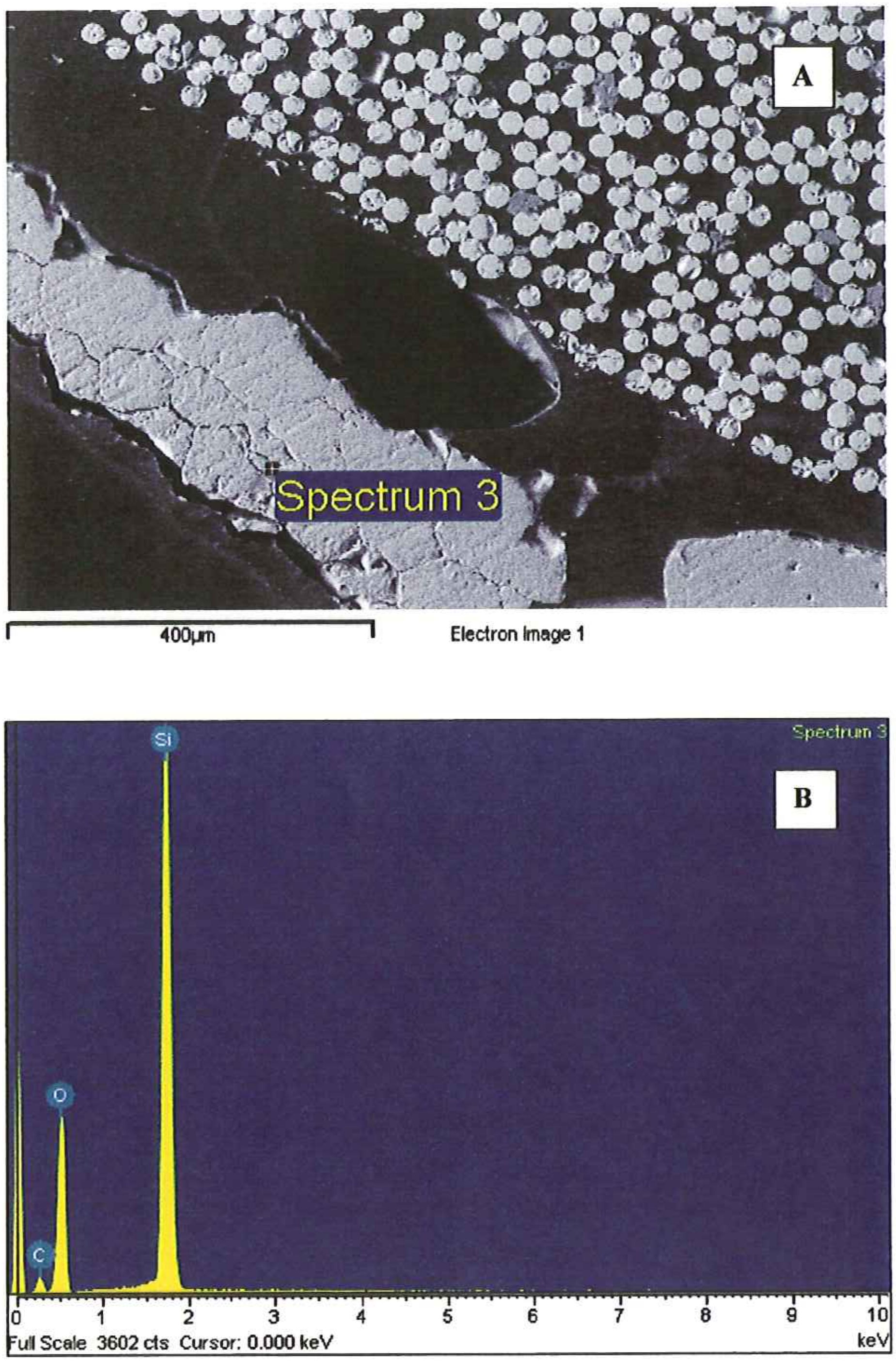


Figure 7.7 Scanning electron microscope micrograph of: A) cross section of the ISOROD bar after 12 months exposure to manure environment and B) chemical composition of sand grain.

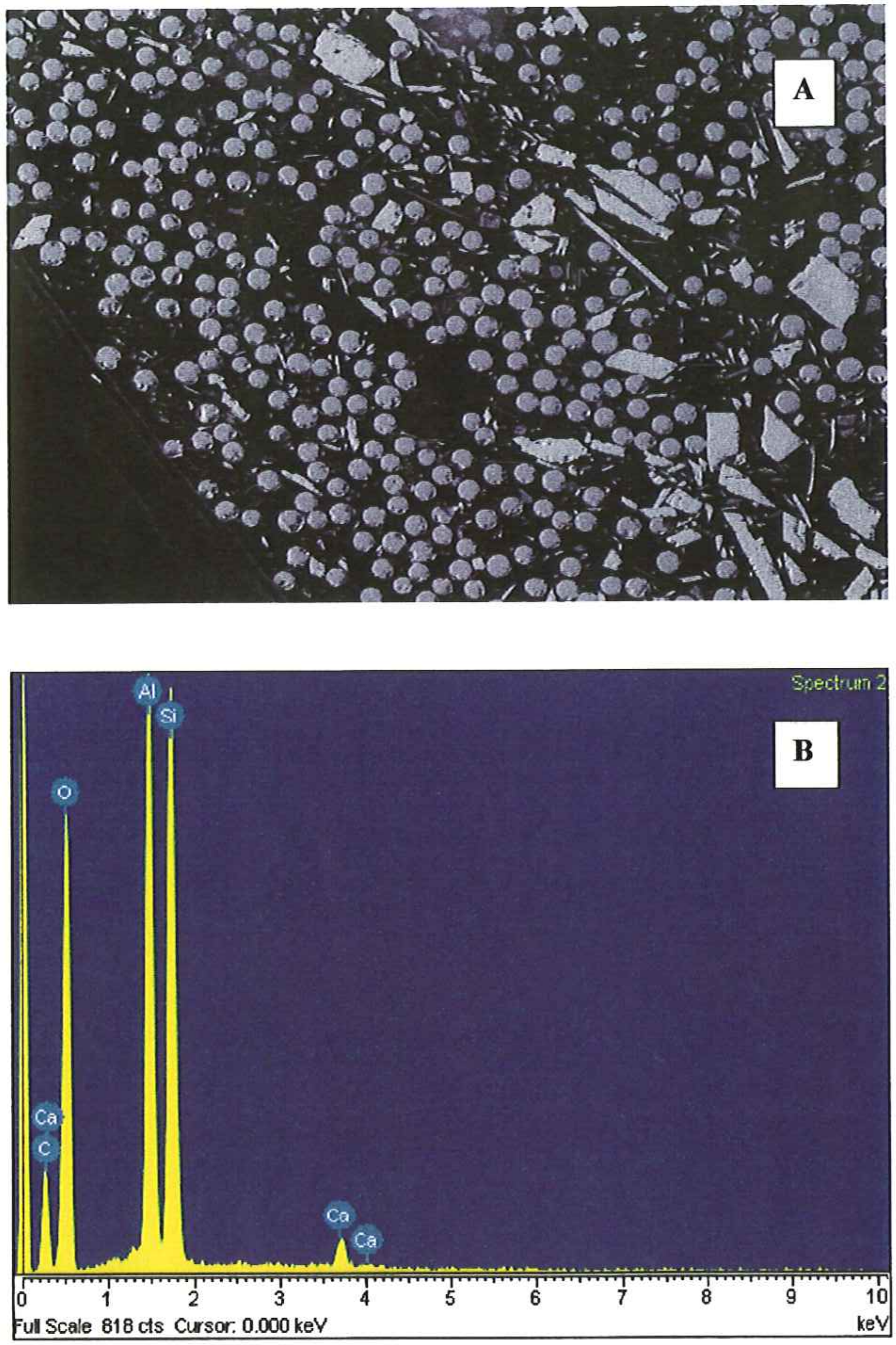


Figure 7.8 Scanning electron microscope micrograph of: A) cross section of the C-BAR bar after 12 months exposure to manure environment and B) chemical composition of the glass.

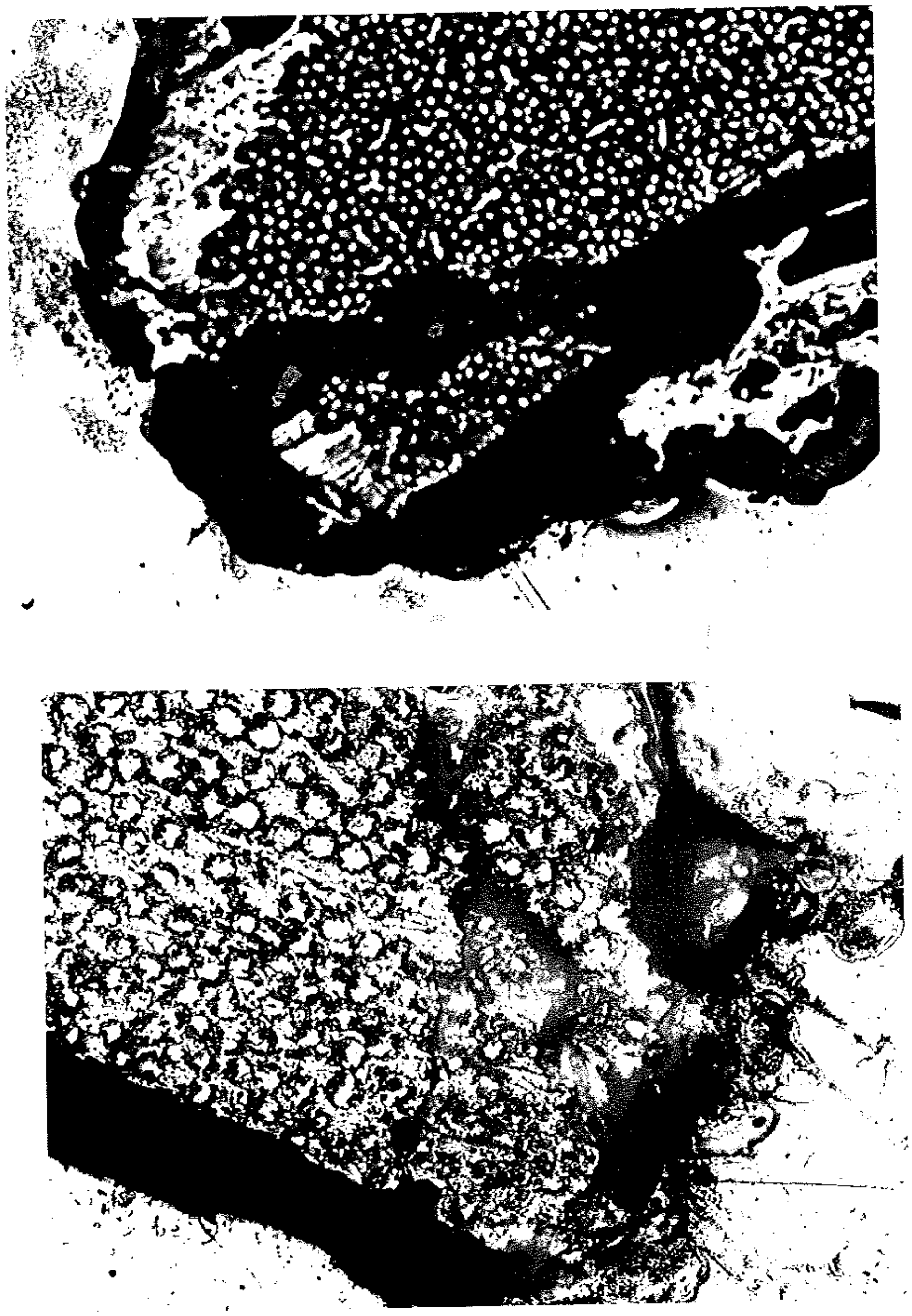


Figure 7.9 Light Microscopy photo of two cross section in the lugs (protrusions) areas of the C-BAR bar after 18 months exposure to manure environment.

Detailed examination is required to determine if the observed degradations are due to chemical reaction with the manure.

The mechanism of glass degradation is a dissolution process. Sometimes, the degradation of fiberglass is termed corrosion. If a corrosion mechanism is considered to be driven by the electromotive forces of thermodynamically reactive metals, then the glass dissolution is not a corrosion process. However, the result of the dissolution process and corrosion processes are similar, i.e., the reduction of the section of the structural element.

Nonetheless, even when the glass dissolution will start in a manure storage tank, conditions are such that the dissolution rate should be very low. In analyzing the kinetics of glass dissolution, it is important to take into account the configuration of the leaching system at the storage tank site.

Experimental and modeling work on the alteration of borosilicate glass indicates that the most important parameters for glass dissolution rates are: a) exposed surface area of glass to dissolved cations and anions in the liquid phase (flow), b) pH of the concrete pore solution, c) dissolved silica concentration in solution, and d) temperature. Since the operative leaching mechanism and the degradation rate of glass are highly dependent on service conditions, two types of time-dependence have to be considered. The total exposure time of the glass controls the transformation of its surface from its dry original state to a form, which is in steady state with respect to interaction with the aqueous environment. During this time, the degradation rate changes, reflecting for instance the build-up of the hydrated layer. This is a transient effect and in the long term, the rate of material loss is likely to become constant. The value of this rate will depend on the contact time between the glass and the solution. This contact time is inversely proportional to the flow rate. The contact time, together with the ratio of the surface area of the glass to the volume of solution in contact with it, determine the extent to which the composition and reactivity of the solution are affected by the dissolution of the glass (i.e., the extent of pH rise due to alkali extraction and the extent of approach to saturation with respect to Si, Al, etc).

For the case of slow flow, the appropriate scenario for concrete reinforced with fiberglass, the glass dissolution rate can be described by:

$$R = k\left(1 - \frac{C_{Si}}{K_{Si}}\right) + k_{long} \tag{7.1}$$

where k is the rate coefficient that is dependent on temperature and pH, C_{Si} / K_{Si} is an affinity term consisting of silica concentration in aqueous phase divided by the solubility of silica in aqueous phase and k_{long} is the long term rate at constant temperature.

Obviously, the value of all coefficients in Equations 7.1 should be determined from experimental data. Use of fiberglass in civil engineering is relatively new. This material is still expensive and not fully tested for many applications. However, based on generalization of a few values reported in the literature, the glass dissolution will probably range between 10^{-1} to 10^{-2} (g m^{-2} d^{-1}). Assuming a density of fiberglass of 1.7 Mg m^{-3} the dissolution rates will probably be in the range of 0.0215 to 0.0021 mm y^{-1}. That is, it will take about 100 years for fiberglass to loss between 0.21 mm and 2.15mm. Although dissolution is the main degradation mechanism of glass, stress cracking may have some impact in a real life storage tank. In this case, the damage caused by stress and dissolution acting together may exceed that produced when they act separately.

7.3 Reinforced Concrete Beams

Sixty-two reinforced concrete beams were tested after they had been exposed to manure environment for 0 (control), 4, 8, 12 and 18 months. Four series (A, B, C and D) of reinforced beams consisting of different concrete / reinforcement / confining material combinations were tested at every time interval (Table 7.6).

Table 7.6. Reinforced concrete beams tested

Element No.	Concrete / Reinforcement / Confining Material Combination
A	Ordinary Concrete (OC) / Steel rebar
B	Ordinary Concrete / GFRP ISOROD rebar
C	GFRP spray / Ordinary Concrete / Steel rebar
D	PVC / Ordinary Concrete / Steel rebar

In series A and B the concrete was in direct contact with the manure, simulating conditions in the common manure tank designs. In series C and D the GFRP spray, composite and PVC were used as cover materials, to prevent exposure of the concrete to the manure. In series A, C and D the longitudinal reinforcement consisted of one single 6.35 mm diameter steel rod. In series B the 10 mm diameter GFRP ISOROD was used as longitudinal reinforcement.

Analysis of the test results for the beams is discussed in terms of load-deflection behavior, failure mode, and cracking pattern.

7.3.1 Concrete Beams Reinforced with Steel Bar (Series A)

The effect of exposure time on the load-deflection behavior of the steel-reinforced concrete beams is shown in Figure 7.10. The typical load-deflection curves of the exposed beams to manure for 4, 8, 12 and 18 months and a control beam are compared. The results show that yielding started before cracking, indicating that the concrete is still strong in tension. The yielding of steel reinforcement in the control specimens started at a load level of 2 kN. Flexural cracking was initiated at a load level of 3.16 kN for the control specimen (12 months). A slight increase in cracking load in the first 12 months of exposure was observed; the average cracking loads were: 3.0 kN after 4 months, 3.9 kN after 8 months, 3.5 kN after12 months and 3.0 kN after 18 months exposure.

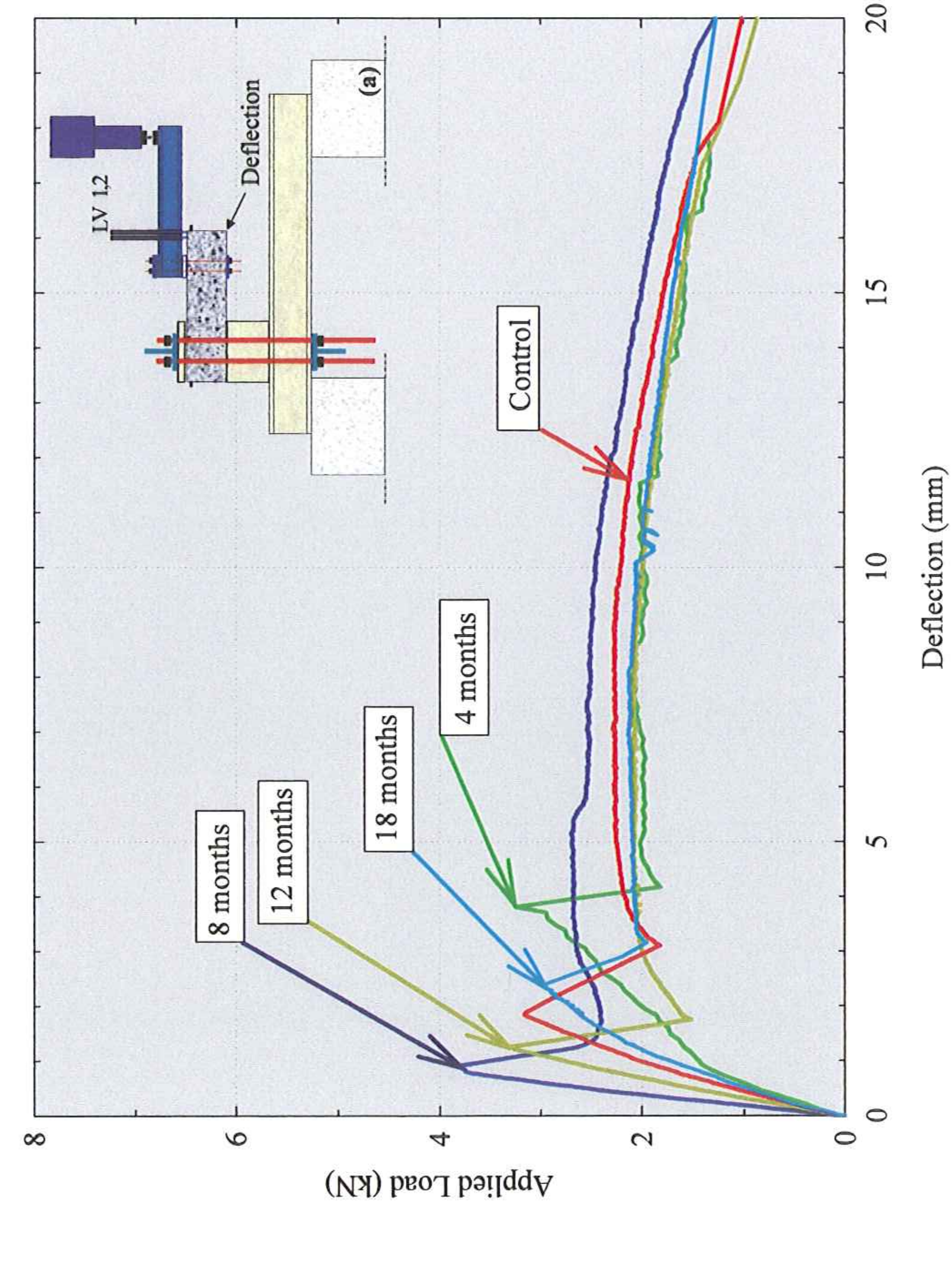


Figure 7.10. The effect of exposure time on the load-deflection behavior of concrete reinforced with steel.(a) Test setup and location of instrumentation.

In most cases, this increase was larger in the specimens exposed to manure for 4 and 8 months than for 12 and 18 months exposure specimens. This increase was attributed to the increase in compressive strength of the concrete and hence its tensile strength. This may reflect changes in the strength of the concrete due to changes in its microstructure as the result of continued hydration. The hydration reaction processes in most cement systems are rarely completed. However, with the increase in curing time, the degree of hydration becomes well advanced and the reaction slows considerably.

Mercury intrusion porosimetry (MIP) analysis on selected concrete specimens indicated that microstructural characteristics (i.e., pore volume, pore radius and pore size distribution) change with exposure time. The pore structure of the reinforced concrete governs to a large extent two of the most important engineering properties of the hardened concrete: 1) mechanical strength, and 2) permeability.

The total pore volume and pore diameter decreased during the 4 and 8 months exposure and slightly increased during the10 months exposure (Figure 7.11). The observed changes were attributed to the continued hydration and therefore progressive densification of the concrete structure as manure progressively penetrated the specimen. The lower value for cracking load after 12 and 18 months exposure compared with the cracking load of 4 and 8 months specimens may be attributed to the observed increase in the total pore volume and pore size distribution (Figure 7.11). Furthermore, microscopic examination of the concrete surface exposed to manure revealed that a distinctive feature of the reinforce concrete/manure interaction was the formation of a surface precipitate layer consisting mainly of Ca phases (i.e., portlandite, calcite), ettringite and new calcium silicate hydrate (CHS) within the first months of exposure. The chemistry of precipitates formed earlier is controlled by the concrete, as shown in Figures 7.12 and 7.13. At the later stage, the precipitate composition is controlled by the chemistry of manure, as shown in Figure 7.14. Precipitation of these phases inside the open pores could also lead to a decrease in pore volume and pore size distribution and therefore progressive densification of the concrete structure.

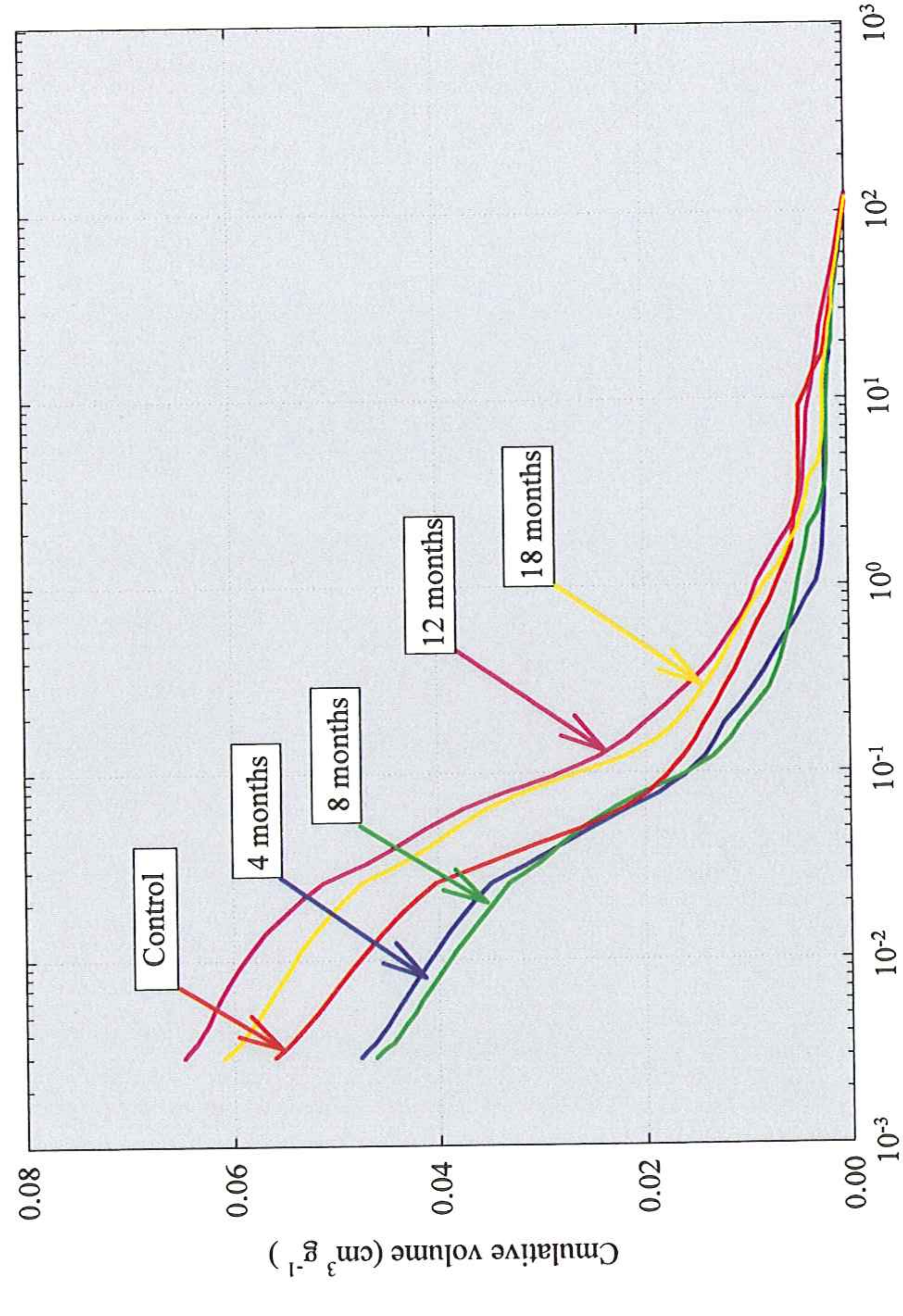
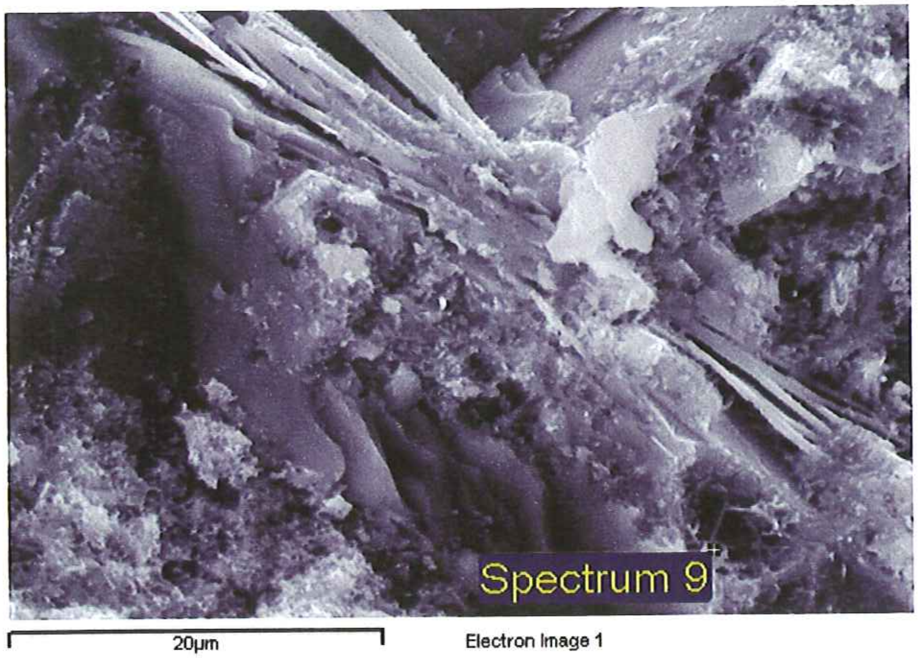


Figure 7.11. Cumulative pore volume for concrete specimens in contact with manure environmemnt for 4, 8, 12 and 18 months.



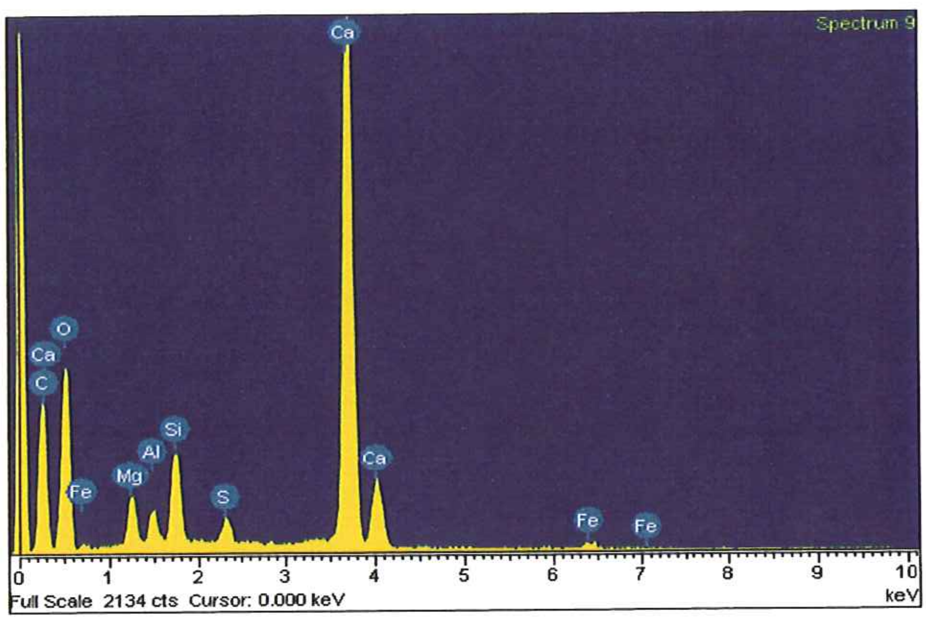


Figure 7.12. Scanning electron micrograph of: A) the concrete surface exposed to manure for 4 months and B) the chemical composition of the precipitated layer.

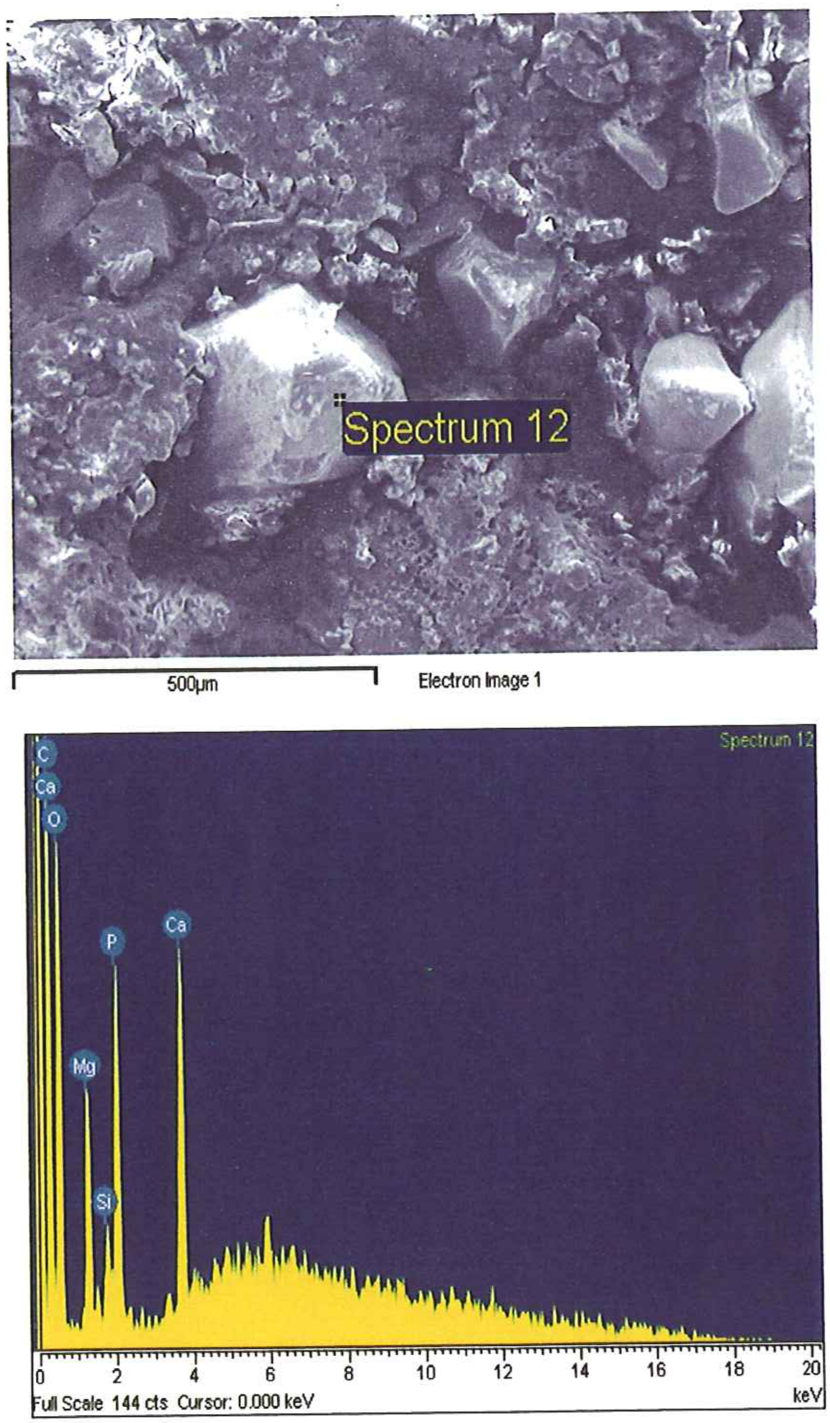


Figure 7.13. Scanning electron micrograph of A) the concrete surface exposed to manure for 12 months and B) the chemical composition of the precipitated layer.

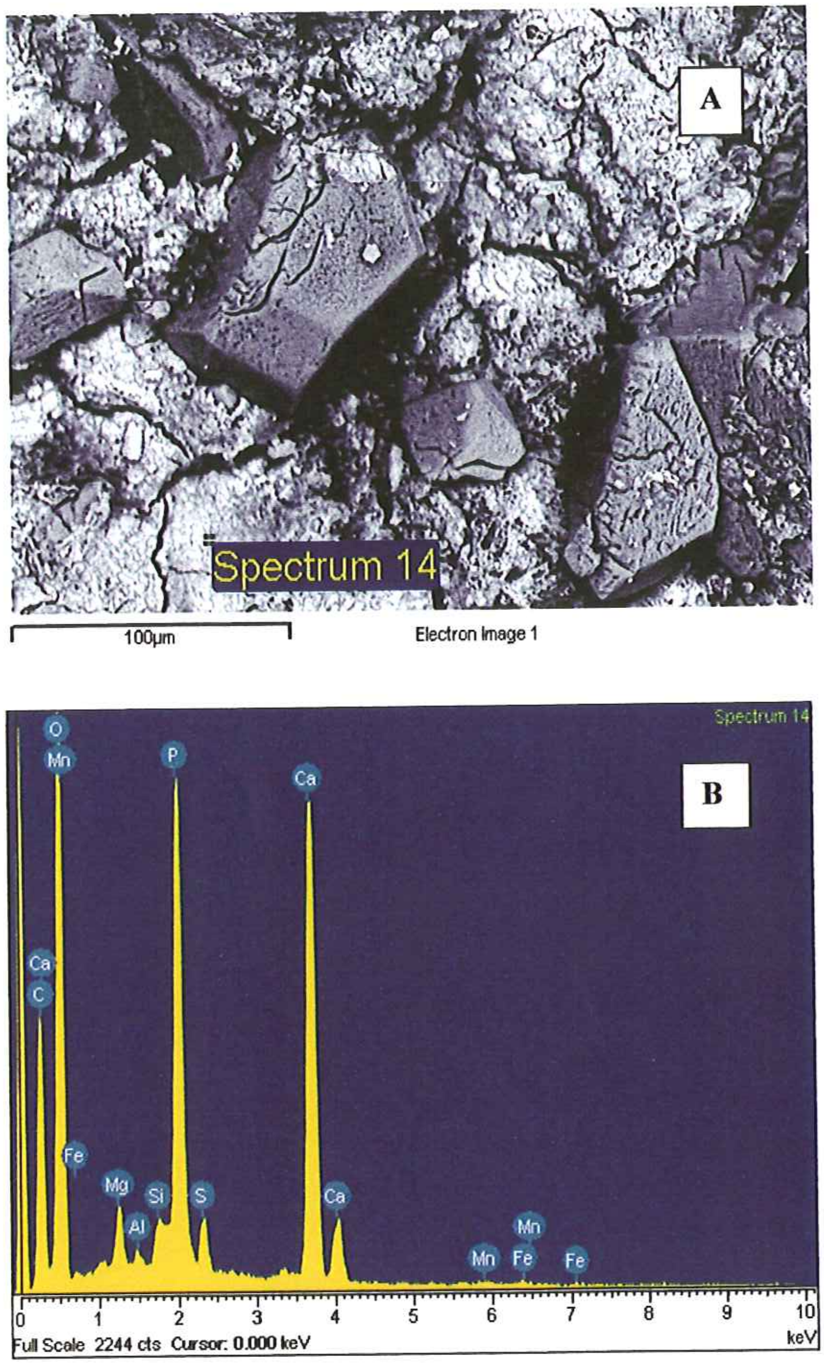


Figure 7.14. Scanning electron micrograph of A) the concrete surface exposed to manure for 18 months and B) the chemical composition of the precipitated layer.

These changes in microstructural characteristics may be attributed to deterioration of the concrete by chemical processes such as leaching of cement phases (i.e., portlandite, calcium silicate hydrates) by the liquid manure.

Before failure, the damage was dominated by two or three typical flexural cracks. However, only one crack led to failure. As was the case with the 4, 8 and 12 months exposed specimens, the initial flexural crack for 18 months exposure specimens always occurred near the support point position.

The initial load-deflection behavior was linear up to cracking, followed by nonlinear behavior after cracking. The discontinuities in the load-deflection curves and the associated decrease in the applied load indicate the formation of new cracks in the reinforced concrete beams. The beams exhibited deformation after yielding of the steel without a significant increase of the applied load until failure occurred. The failure load in the control specimen (12 months) occurred at 3.16 kN. An increase in the ultimate load was observed after eight months exposure and this was attributed to the increase in strength of the concrete. The failure loads of beams in contact with manure for 12 months ranged between 3.2 kN -3.9 kN and for 18 months was 3.0 kN (Table 7.7).

The dominant failure mode was flexural failure (rupture of the steel bar) on the tension face. The failure in the beams was always seen to occur near the support point position, where the initial flexural cracks had been observed. Before failure, the damage was dominated by two or three typical flexural cracks (vertical cracks); However, only one crack led to failure (Figure 7.15). At the cracked section, slippage of the bars due to debonding was not observed.

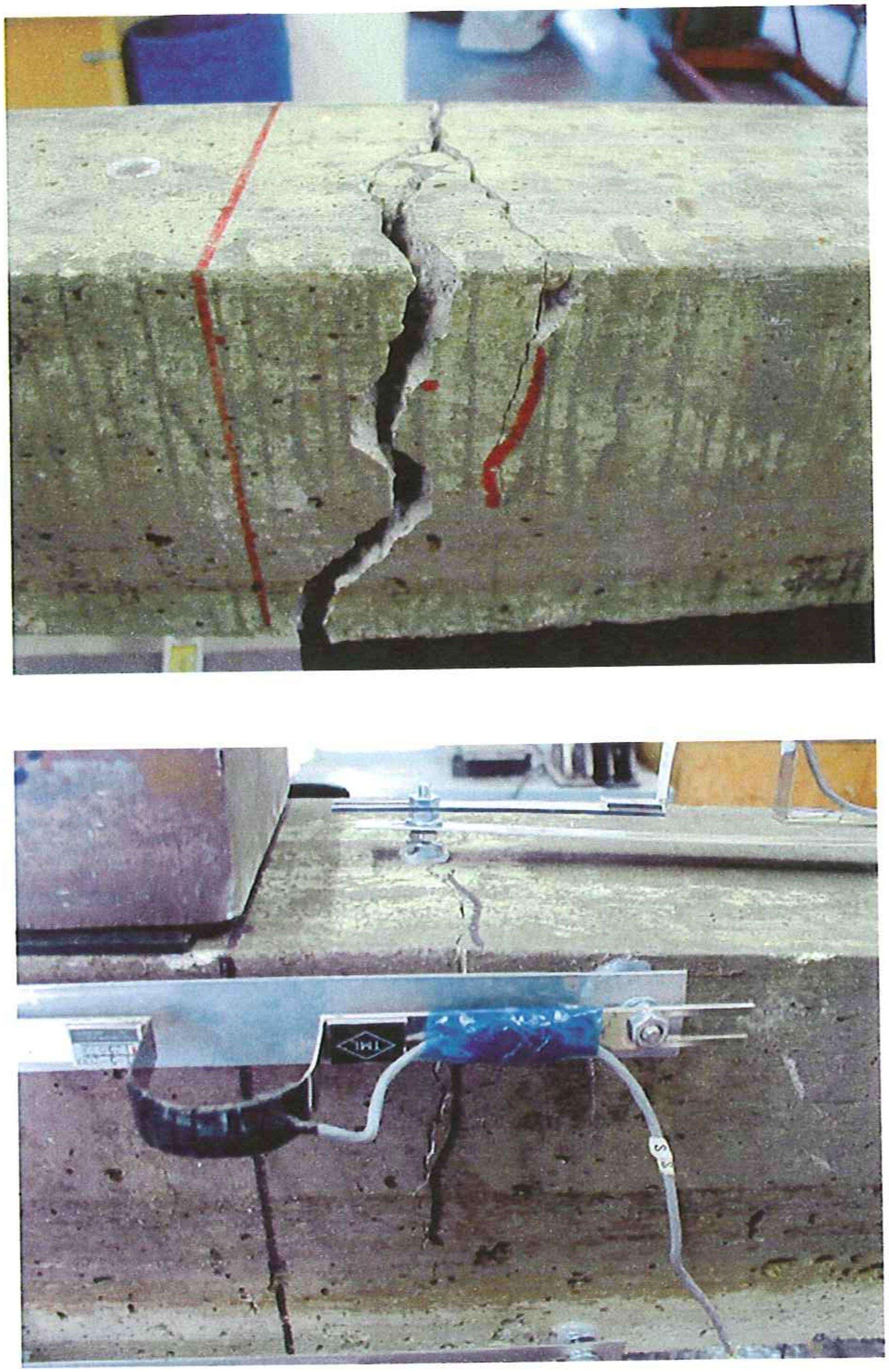


Figure 7.15. Typical failure of the concrete beams reinforced with steel.

Table 7.7 Summary of experimental results for the steel reinforced concrete beams after 0 (control), 4, 8, 12 and 18 months exposure to manure environment.

Exposure time (month)	Cracking load (kN)	Deflection at cracking (mm)	Ultimate load (kN)	Ultimate deflection (mm)	Mode of failure
0 (control)	1.9 - 3.2	1.5 – 1.9	2.2 – 2.6	16.2 – 20.1	Flexural failure – Rupture of steel bar
4	2.9 - 3.1	1.9 - 3.9	2.0 - 3.1	18.0 – 21.8	Flexural failure – Rupture of steel bar
8	2.7 - 4.0	0.6 – 1.0	2.1 – 4.0	15.0 - 19.4	Flexural failure – Rupture of steel bar
12	3.2 - 3.9	1.3 – 1.66	3.2 – 4.9	19.0 - 20.9	Flexural failure – Rupture of steel bar
18	2.97	2.5	2.97	23.3	Flexural failure – Rupture of steel bar

Although some changes did take place in the beams reinforced with steel, no significant deterioration of these beams took place for the duration of the experiment (Table 7.7.). However, stress induced creaking has always developed in the actual storage tanks. They are either developed due to loading forces or due to the transformation of metallic iron to rust that is accompanied by an increase in volume. Depending on the state of oxidation, the oxidation products may be as large as 600% of the original metal. Once the concrete cracks expand, the reinforcement is no longer protected by the concrete, the reinforcement will be in contact with the manure and the corrosion of the steel bars will progress rapidly as observed on steel reinforcement bars described in chapter 7.2.1.

7.3.2 Concrete Beams Reinforced with ISOROD Bar (Series B)

The typical load-deflection curves for the concrete reinforced with ISOROD bar after 0 (control), 4, 8, 12 and 18 months exposure to a manure environment are presented in Figure 7.16. The experimental results suggest a small, insignificant, increase in the flexural cracking load with exposure time. At 18 months, the beams showed linear behavior up to the first crack at a load level that ranged between 3.0 kN and 3.1 kN (Table 7.8). Cracking consisted predominantly of flexural cracks. The first crack was initiated near the steel plate used to prevent bond failure. As loading progressed, new flexural cracks formed, as reflected in the curve discontinuities, (Figure 7.16) although only one crack led to failure. Cracking patterns as well as the widths of the cracks were found to be different from those in steel-reinforced concrete beams. This is due to the linear stress-strain behavior, different bond characteristics, and lower modulus of elasticity of GFRP ISOROD bars relative to steel reinforcement.

A slight decrease in the ultimate load was observed after 18 months exposure. However, this decrease appears insignificant. The control specimens failed at an average load of 4.1 kN whereas the average ultimate load for beams exposed for 4, 8, 12, an 18 months to manure were 3.7, 5.3, 3.7 and 3.7 kN, respectively. The observed changes in strength characteristics were attributed to changes in concrete microstructure as discussed in Section 7.3.1. Longer time (12 and 18 months) exposure to the manure had no effect on the mode of failure; flexural failure and rupture of the ISOROD bar. Examination of the fracture revealed that failure of the ISOROD bar took place by both glass-fiber strand fracture and fiber pullout (Figure 7.17). No slippage of the ISOROD bars was observed at the cracked sections.

The summary of mechanical test results on ISOROD reinforced concrete beams at 0 (control), 4, 8, 12 and 18 months exposure to manure environment is presented in Table 7.8.

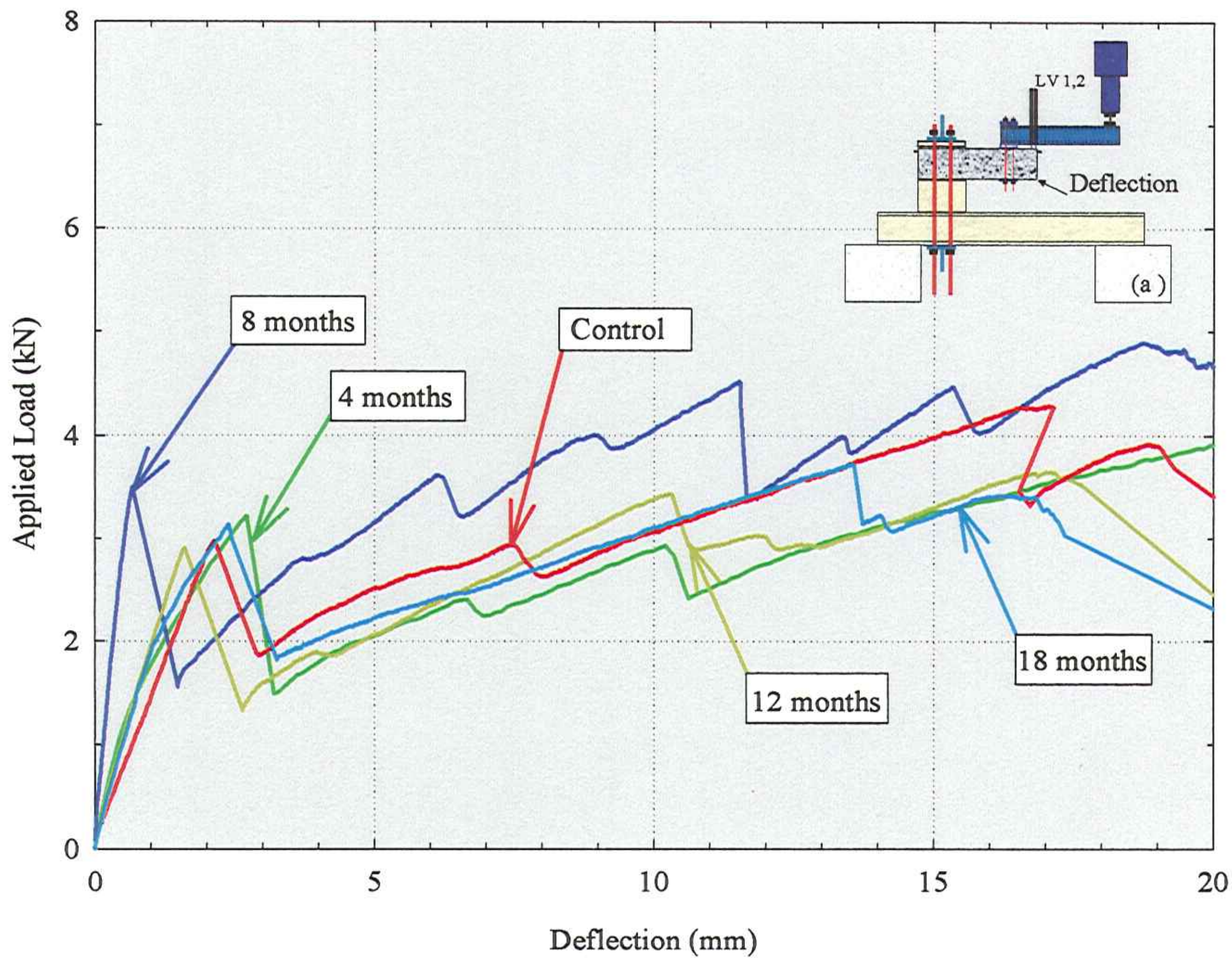
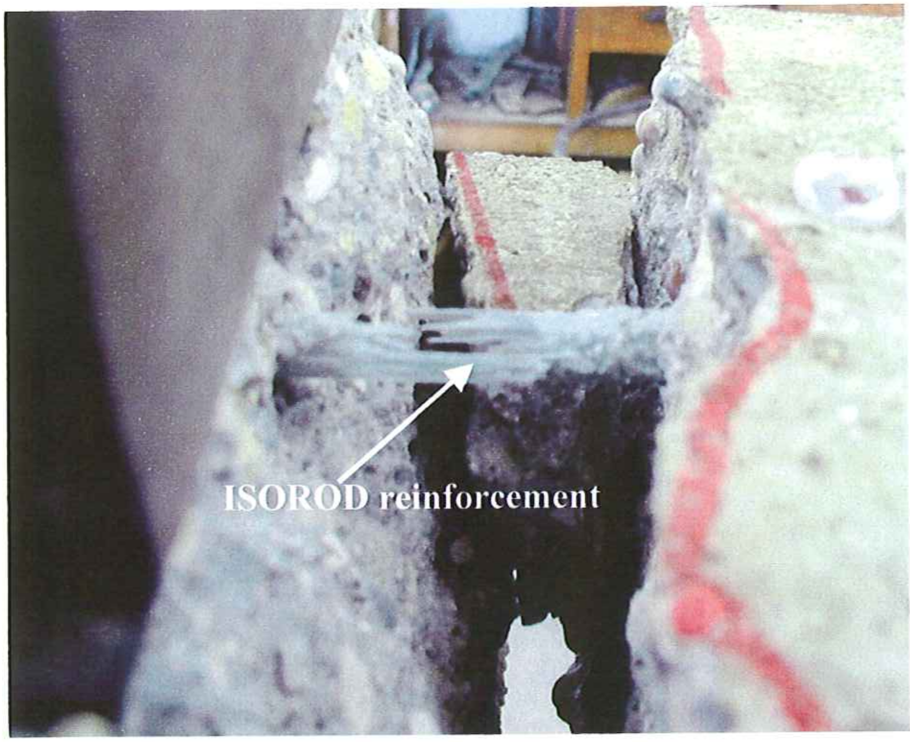


Figure 7.16. The effect of exposure time on the load-deflection behavior of concrete beams reinforced with GFRP ISOROD bar. (a) Test setup and location of instrumentation.



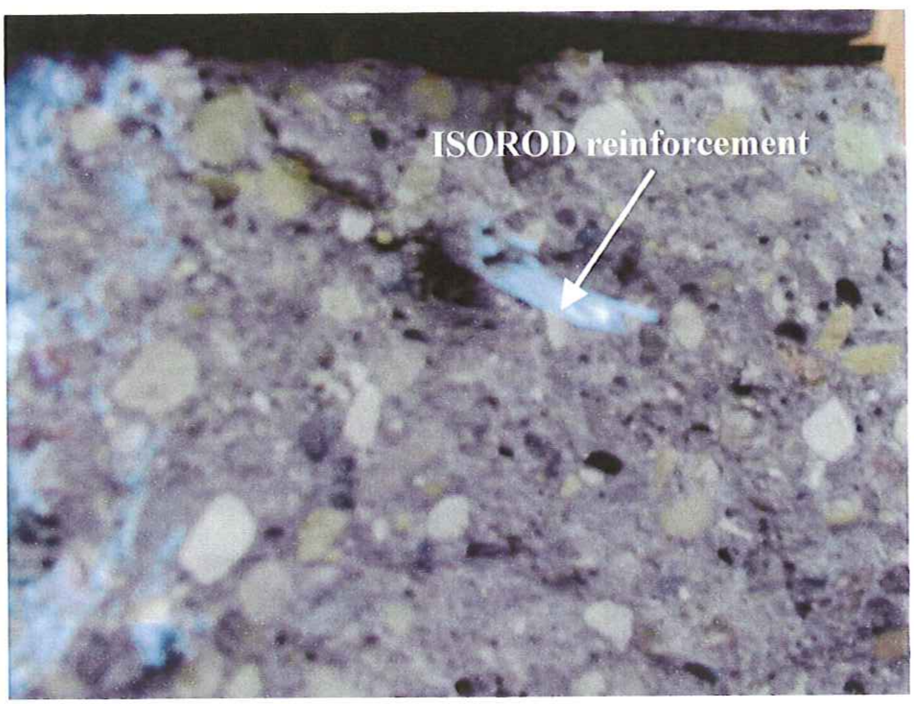


Figure 7.17. Typical failure pattern of the ISOROD reinforcement bar in concrete beams.

Table 7.8 Summary of experimental results for ISOROD reinforced concrete beams after 0 (control), 4, 8, 12 and 18 months exposure to manure environment.

Exposure time (month)	Cracking load (kN)	Deflection at cracking (mm)	Ultimate load (kN)	Ultimate deflection (mm)	Mode of failure
0 (control)	2.0 - 3.0	0.7 – 2.2	3.7 – 4.3	17.1– 21.4	Flexural failure – Rupture of ISOROD bar
4	2.1 - 3.4	1.3 - 3.0	3.0 - 4.3	20.0 – 27.5	Flexural failure – Rupture of ISOROD bar
8	3.5 - 4.7	0.8 – 3.0	4.9 – 5.9	11.4 - 18.5	Flexural failure – Rupture of ISOROD bar
12	2.9 - 3.6	1.2 - 1.6	3.7 – 3.9	16.3 - 20.8	Flexural failure – Rupture of ISOROD bar
18	3.0 - 3.1	2.4 – 2.8	3.6 – 3.7	16.3 - 21.2	Flexural failure – Rupture of ISOROD bar

Overall, no significant changes have been observed in the mechanical properties with exposure time for the reinforced concrete beams with ISOROD bar. The measured value for some parameters such as deflection at cracking and ultimate deflection showed a large variability (Table 7.8). This may be attributable to manufacture inconsistency.

One of the reasons for considering replacing the steel with GFRP for concrete reinforcement is that steel corrodes. However, use of the GFRP has its own problems; the highly alkaline environment of concrete pore water may create a problem for GFRP reinforcement. The chemical processes involved during the curing of concrete create an environment (i.e., high pH) in which GFRP reinforcements are vulnerable to chemical attack. The increase in pH of the concrete is due to cement hydration. When combined with water, the hydration reactions produce calcium silicate hydrates (CSH) and calcium hydroxides ($Ca(OH)_2$. The free lime as well as the alkali oxides react with water to increase the pH to values between 11.5 and 13.7. However, the final pH depends on the concrete mix design, the type of cement used, and the addition of pozzolanic materials (i.e., silica fume, fly ash). Furthermore, the pH in concrete decreases with maturation and carbonation. In general, the alkaline solution produces embrittlement of the matrix and damage at the fiber-polymer interface. Greater resistance to the high pH can conceivably be achieved by using either an alkali-resistant polymer or alkali-resistant fibers, or a combination of both in the manufacture of GFRP. In addition, the use of polymer with very low permeability (i.e., very small porosity) can aid alkali resistance. Since the GFRP bars in almost all applications are under tension, the polymer must be resistant to micro-cracking. Any defects in the polymer facilitate deleterious solutions reaching the glass. Since the glass is the primary component of the composite affected by alkalinity, use of alkali resistant glass in the manufacture of GFRP should be mandatory.

The results produced by the present studies suggest that GFRP reinforcement bars exhibited a high resistance to degradation in a manure environment. Consequently, penetration of the manure into the concrete through diffusion or through the inherent cracks that occur in concrete will have little influence on the degradation of the reinforcement. Furthermore, the manure - pore water exchange that will take place during tank operation will not sustain the high alkalinity in concrete over a long period of time.

7.3.3 Concrete Beams Reinforced with Steel Bar and Sprayed with GFRP (Series C)

The load-deflection behavior of the steel-reinforced concrete beams sprayed with GFRP after 0 (control), 4, 8, 12 and 18 months exposure is shown in Figure 7.18. As can be seen, the load-deflection behavior was non-linear until failure. This was attributed to the random nature of the fiber distribution and the short length of the fibers in the GFRP spray coat.

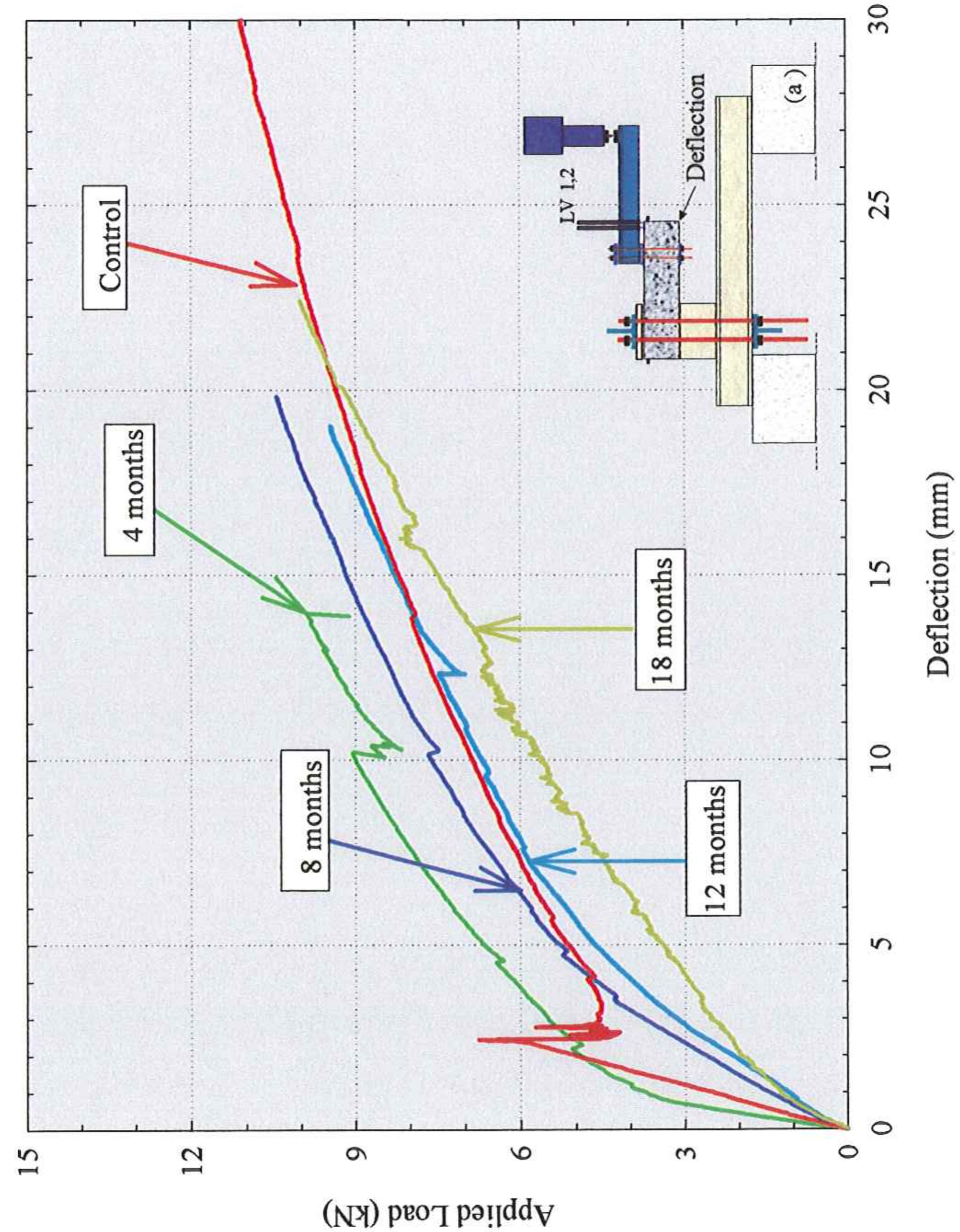


Figure 7.18. Effect of exposure time on load-deflection behavior of steel reinforced concrete beams sprayed with GFRP. (a) Test setup and location of instrumentation.

A slight decrease in the ultimate failure load with exposure time was observed. The average ultimate failure loads at 4, 8, 12 and 18 months exposure were 9.6 kN, 9.9 kN, 10.4 kN, 9.9 kN. For control specimens, the ultimate failure load was 11 kN. However, it should be noted that the ultimate failure load of the reinforced concrete beams sprayed with GFRP was about three times higher than the ultimate failure load of plain (uncovered) beams reinforced with steel (Series A) and about two times higher than the ultimate failure load for beams reinforced with ISOROD (Series B). A significant improvement in the toughness of the beams due to the sprayed coating was observed. The overall response of the beams sprayed with GFRP after 18 months exposure was found to be similar to the overall response of the beams exposed for 4, 8 and 12 months to manure. The average cracking loads at 12, 8 and 4 months were very similar, i.e. 3.9, 4.2 and 3.9 kN, respectively. A slightly lower average value (2.5 kN) was observed at 18 months exposure. The observed decreases in both cracking load and ultimate load may be attributed to moisture accumulation at the concrete /sprayed GFRP cover. The mode of failure seems to have been unaffected by the longer exposure of the beams to manure. The reinforced concrete beams sprayed with GFRP continue to fail in flexural mode with rupture of the GFRP coat. A typical flexural failure exhibited by the reinforced concrete beams sprayed with GFRP after 18 months exposure is shown in Figure 7.19. The sprayed glass-fiber reinforced composite failed by a combination of fiber fracture and pull-out of fibers from the matrix. Failure patterns indicated that the bond between the sprayed GFRP and concrete in all reinforced concrete was strong enough and no debonding was seen to occur.

Summary of the test results at 0 (control), 4, 8, 12 and 18 months exposure to the manure environment is presented in Table 7.9. No significant changes have been observed in the steel-reinforced beams sprayed with GFRP. A large variation of parameters measured on these beams was attributed to non-uniformity of the GFRP sprayed coating.

In addition to the increases of the beam strength, the sprayed GFRP acted as a secondary protection layer for the reinforcement; the concrete cover of the reinforcement is well protected from contact with manure.

Figure 7.19. Typical flexural failure of the concrete beams reinforced with steel and sprayed with GFRP after 18 months exposure to manure environment.

Table 7.9 Summary of experimental results for steel-reinforced concrete beams sprayed with GFRP after 0 (control), 4, 8, 12 and 18 months exposure to the manure environment.

Exposure time (month)	Cracking load (kN)	Deflection at cracking (mm)	Ultimate load (kN)	Ultimate deflection (mm)	Mode of failure
0 (control)	6.8 - 7.9	2.4 - 3.4	11.1 – 11.8	24.0- 30.0	Flexural failure – Rupture of steel bar
4	3.4 - 4.6	1.0 - 1.7	8.9 - 9.9	13.1 - 14.3	Flexural failure – Rupture of steel bar
8	3.5 - 4.5	2.9 - 5.5	8.8 - 10.8	17.9 - 23.2	Flexural failure – Rupture of steel bar
12	3.6 - 4.1	2.5 - 3.5	9.4 – 11.5	15.3 - 19.0	Flexural failure – Rupture of steel bar
18	2.3 - 2.7	1.7 - 2.7	9.7 - 10.0	17.8 - 22.8	Flexural failure – Rupture of steel bar

7.3.4 Concrete Beams Reinforced with Steel Bar and Covered with PVC (Series D)

The load-deflection behavior of the steel-reinforced concrete beams and covered with PVC panels after 0 (control), 4, 8, 12 and 18 months exposure is shown in Figure 7.20. There are no notable differences in the load-deflection behavior between the control beam and those exposed for 18 months. The control specimen failed at an average load of 4.0 kN whereas the average ultimate load value for the exposed beam for 4, 8, 12 and 18 months were 4.3 kN, 3.8 kN, 4.1 kN and 4.2 kN, respectively. Longer exposure time (18 months) to the manure environment did not appear to further affect the ultimate failure load or the cracking load. Table 7.10 presents a summary of the results for the concrete beams reinforced with steel bar and covered with PVC panels after 0 (control), 4, 8, 12 and 18 months exposure.

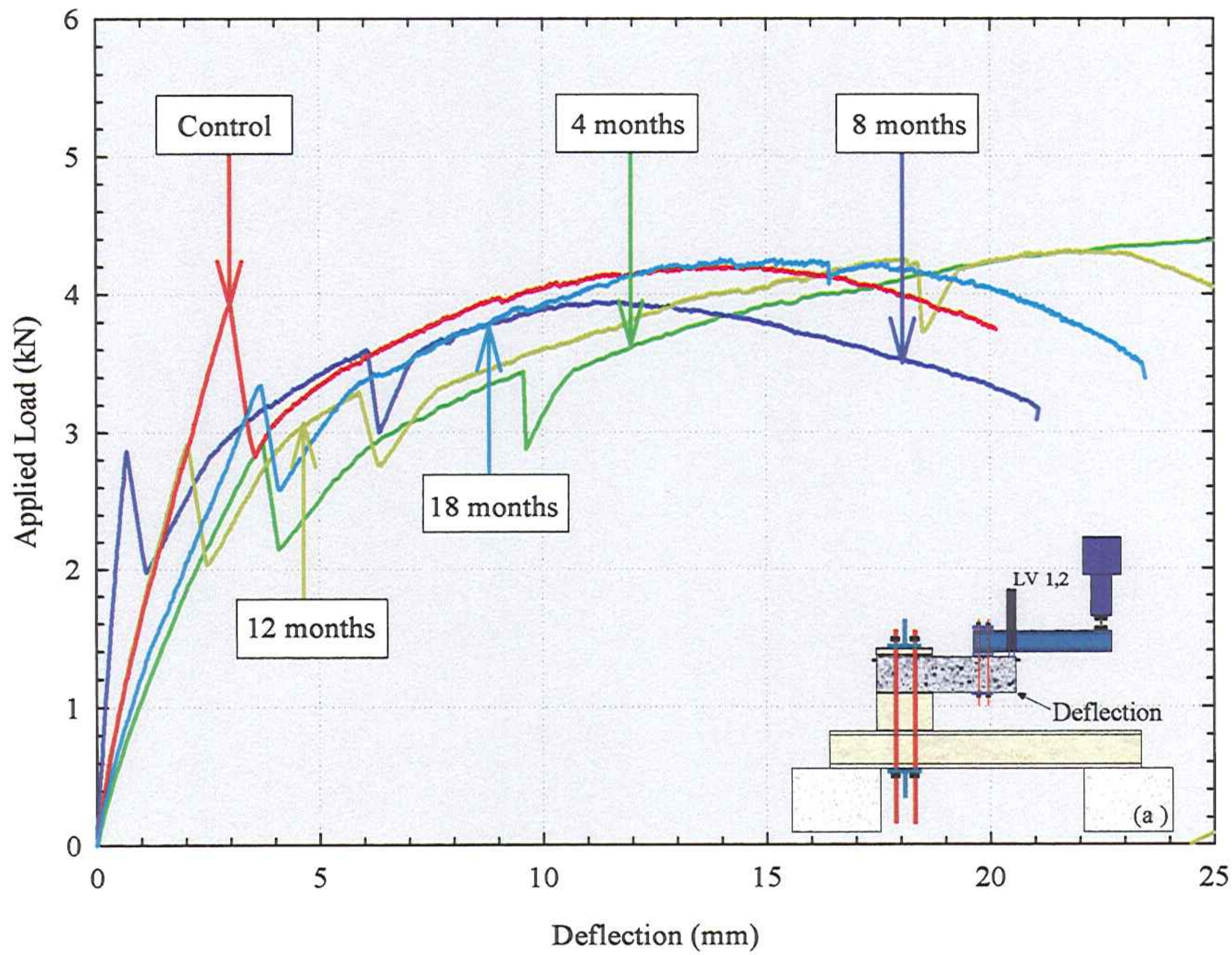


Figure 7.20. Effect of exposure time on the load-deflection behavior of steel reinforced concrete beams cover with PVC panels.(a) Test setup and location of instrumentation.

The steel reinforced beams covered with PVC panels (series D) have been found to have a higher load carrying capacity than the steel reinforced beams without cover (series A). The increase in ultimate load of steel reinforced beams covered with PVC panels are attributed to the special profile (strip) embedded in the concrete during casting. The results suggested that the PVC panel strip when well embedded in the concrete acts as an additional reinforcement for the concrete.

Table 7.10 Summary of experimental results for steel-reinforced concrete beams covered with PVC panels after 0 (control), 4, 8, 12 and 18 months exposure to manure

Exposure time (month)	Cracking load (kN)	Deflection at cracking (mm)	Ultimate load (kN)	Ultimate deflection (mm)	Mode of failure
0 (control)	2.2 - 3.9	1.4 – 6.2	3.7 – 4.2	20.3- 35.0	Flexural failure – Rupture of steel bar
4	2.8 - 3.2	3.9 - 7.0	4.2 – 4.4	27.6 - 35.0	Flexural failure – Rupture of steel bar
8	2.2 - 2.9	0.7 - 2.0	3.7 - 3.9	30.3 – 37.0	Flexural failure – Rupture of steel bar
12	2.7 - 3.5	1.6 - 3.0	3.3 – 4.4	31.5 - 37.0	Flexural failure – Rupture of steel bar
18	3.3 - 3.4	3.8 - 4.6	4.2 - 4.6	23.7 - 29.7	Flexural failure – Rupture of steel bar

The average cracking load values at 0 (control), 4, 8, 12 and 18 months were similar, i.e., 3.0 kN, 3.0 kN, 2.6 kN, 3.0 kN and 3.4 kN, respectively. Not much change in the concrete strength due to curing was observed. The concrete being confined by the PVC panels prevents the moisture necessary for continued hydration to come in contact with the concrete.

In general, the ultimate failure load of the steel-reinforced beams covered with PVC was higher than those for the bare steel-reinforced beams and lower than for the steel-reinforced beams sprayed with GFRP.

Failure of all beams was due to rupture of the steel bar as shown in Figure 7.21. Peeling of the PVC cover on most beams was initiated at the flexural crack. However, the special PVC panels profile (strip) embedded in the concrete during casting remained attached to the concrete and kept the concrete pieces together.

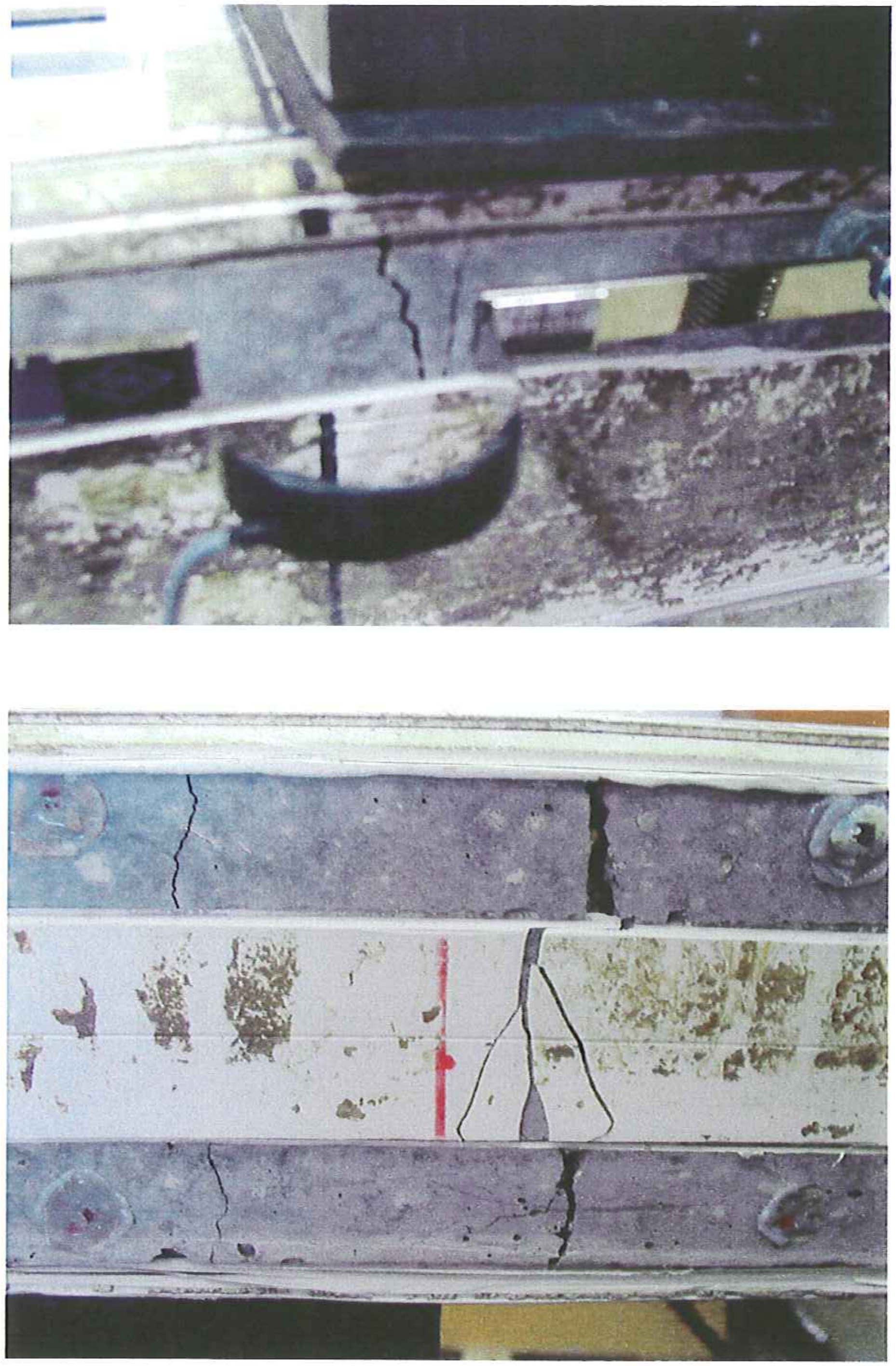


Figure 7.21. Typical flexural failure of the concrete beams reinforced with steel and covered with PVC panels.

8 CONCLUSIONS

Environmental concerns about the integrity of manure storage facilities have been raised in many regions across the country. Hog manure storage in steel-reinforced concrete tanks is a major constraint on a confined animal production system.

The challenge of reinforced concrete is its long-term stability, which controls the so-called durability or service life of concrete structures. Due to the hostile service environment associated with manure storage, corrosion rates of the steel reinforcement are potentially high. These deterioration rates could lead to relatively short service life of the tanks. The advanced composite materials, FRP's, have an outstanding strength/weight ratio and a high degree of chemical inertness to most civil engineering environments, strongly suggesting their consideration as reinforcement for concrete manure storage tanks.

The primary objective of the research in this study was to investigate the suitability of innovative design procedures for reinforced concrete manure storage tanks using advanced composite materials (FRP) as the internal or external reinforcing element.

This study focused on the effects of environmental exposure on long-term performance of various reinforcement bars and reinforced concrete structural elements (concrete type – reinforcement combinations) for use in hog manure tank applications. The relevant conditions included the chemicals in liquid manure solutions, temperature, moisture, and the duration of contact of structural elements with hog manure. The effects considered include changes in physical and mechanical properties. In order to investigate the effects on long-term performance of composite materials, we considered methodologies (test conditions) to accelerate the degradation phenomena in reinforced concrete. Acceleration in the experiment has been achieved by using elevated temperature and exposing the structural elements to wet/dry cycles. In addition, the manure in contact with the reinforced concrete structural elements was changed every two weeks to maintain the reactivity of the manure with respect to concrete. Two different types of specimens have been used in this study: 1) straight reinforcement bars; and 2) reinforced concrete beams.

With regard to reinforcement materials, attention was focused on steel rebar (the reinforcement currently used in all designs for manure tanks) and on three types of glass fiber reinforcement polymers that are the least expensive and have a great potential to improve significantly the service life of these tanks. The GFRP materials under investigation were the GFRP ISOROD, GFRP C-BAR and GFRP spray composite.

Test beams have been designed according to Canadian Code standards. They were designed to ensure that the only mode of failure of the reinforced concrete beam is flexural failure, with rupture of the reinforcement. Three different cover schemes were implemented: 1) concrete beams not covered with any protective material, to simulate real-life conditions where the reinforced concrete is in direct contact with manure; 2) beams sprayed on the four vertical sides and the bottom face of the beam with GFRP; and 3) PVC plates covering the four vertical sides and the bottom face of the reinforced concrete beam.

Large-scale experimental tests included in the program have been carried out at the experimental site established at Glenlea Research Station, Manitoba in a restored barn. The reinforcement bars and reinforced concrete beams were kept in contact with manure in specially design containment units.

Mechanical testing was conducted at W.R. McQuade Structural Laboratories at the University of Manitoba. Reinforcement bars were tested in axial tension and the reinforced concrete beams were tested in cantilever mode. Analysis of the test results on the bars is discussed in terms of yield strength, elastic modulus and ultimate strength. Changes in the performance of the beams are discussed in terms of load-deflection behavior, failure mode and cracking patterns.

Fifty-two reinforcement bars and 62 reinforced concrete beams were tested after they had been exposed to a manure environment for 0 (control), 4, 8, 12 and 18 months to determine the changes in their mechanical properties with exposure time. In addition, systematic microstructure analyses of the reinforced concrete were made to examine the results of chemical attack and physical degradation due to the environmental exposure conditions;

factors controlling the initial microstructure development are strongly related to reinforced-concrete durability. Frequent quantitative analyses of the manure were also made to identify changes resulting from contact with the concrete structural elements.

The objective of the study has been achieved. The experimental results of this study suggested the following conclusions that are of particular relevance to the application of innovative design procedures for reinforced concrete manure storage tanks using advanced composite materials as the internal or external reinforcing element.

The tensile test results on the reinforcement bars investigated, i.e., steel, ISOROD and C-BAR, indicated that their strength characteristics (yielding, tensile strength and modulus of elasticity) decline with exposure time to manure.

Under the experimental conditions of this study, the mechanical characteristics of the steel reinforcement bars were the most affected. For example, the ultimate tensile strength decreased about 40% compared with controlled specimens after 18 months exposure. It appears that the corrosion rate was high at all times, but in particular during the 4-8 months period. All steel specimens failed before reaching the yield strain. The observed decrease of mechanical characteristics of steel reinforcement bars was due to advanced corrosion (i.e. localized corrosion and/or general corrosion). This led to a substantial decrease in diameter of the steel bars in various places and consequently to a significant mechanical weakening. The relatively low pH (7.3 - 8.5) of the manure and the presence of chloride, (500 – 800 mgL^{-1}) destroyed the protective iron oxide film, which makes the steel resistant to corrosion. Microscopic examination of the steel bars revealed that the uniform corrosion (general corrosion) after the first 4 months exposure becomes localized corrosion (pitting corrosion) thereafter; the presence of the corrosion crust on the surface of the steel bars induced local variations in the electrochemical potential and consequently the development of localized corrosion. The pits formed in the steel bar surface acted as sites of stress concentration. Localized corrosion is characterized by much more rapid corrosion rates compared with general corrosion. Corrosion of steel reinforcement bars can be initiated and maintained in a manure storage tank under two broad sets of conditions; 1) high pH conditions in the presence of chloride ions; or 2) low pH conditions in the absence of chloride ions. In real life operation, the tank wall is exposed cyclically to wet/dry conditions and significant localized corrosion is expected to occur.

The experimental results show a decrease in both the modulus of elasticity and the ultimate tensile strength during each exposure time interval for ISOROD and C-BAR bars. On average, the modulus of elasticity and ultimate tensile strength of the ISOROD bars after 18 months exposure decreased by about 17 % and about 19 %, respectively, compared with the control specimens. It appears that decreases in elastic modulus took place mainly during the first four months of the experiment (0 - 4 months). Very little change in modulus of elasticity took place in the second and third period (4 - 8 months and 8 - 12 months). In the last six months of exposure to manure the ultimate tensile strength decreased by only 3%. The results for the GFRP C-BAR reinforcement bars follow a similar trend as the GFRP ISOROD bar results. However, the values for the ultimate strength of the C-BAR reinforcement are much higher (average control 862 MPa and after 18 months in contact with manure about 760 MPa) than for the ISOROD (average control 612 MPa and after 18 months in contact with manure about 498 MPa). The difference between the strength characteristics of the ISOROD and C-BAR bars may be attributed to differences in their production method. The observed decreases in the ultimate tensile strength and the modulus of elasticity with time of exposure for both ISOROD and C-BAR reinforcement bars were attributed to absorption of moisture by the polymer; the diffused moisture weakened the glass fiber/polymer interfacial bond strength. The mode of failure of the exposed bars supports this supposition; The failure behavior of both GFRP's bars was characterized by surface fiber breakage either at the center or close to the lower grips. The main mechanism of glass degradation is a dissolution process. Glass dissolution was not observed. Nonetheless, even when the glass dissolution will start in a manure storage tank, conditions are such that the dissolution rate should be very low. The condition expected at the wall tank are: exposed surface area of glass to dissolved cations and anions in the liquid phase (flow) low; dissolved silica concentration in solution relatively high; and temperature relatively low.

Sixty-two reinforced concrete beams were tested for load-deflection behavior, failure mode, and cracking pattern after they had been exposed to a manure environment for 0 (control), 4, 8, 12 and 18 months. Four series (A, B, C and D) of reinforced beams consisting of different concrete / reinforcement / covering material combinations were tested at every time interval. The four series were: A) Ordinary Concrete (OC) / Steel rebar; B) Ordinary Concrete / GFRP ISOROD rebar; C) GFRP spray / Ordinary Concrete / Steel rebar; and D) PVC / Ordinary Concrete / Steel rebar.

In series A and B the concrete was in direct contact with the manure, simulating conditions in the common manure tank designs currently in use. In series C and D the GFRP spray composite and PVC were used as cover materials, to prevent exposure of the concrete to the manure.

The flexural test results on the reinforcement concrete beams showed an increase in the cracking load with exposure time indicating that the concrete is still strong in tension. The observed increases were attributed to the increase in compressive strength of the concrete and it's tensile strength. Changes in microstructure, in particular refinement of the pore structure resulting from increases in degree of hydration and/or precipitation of new phases have been observed to form on the surface of concrete in contact with manure; these could be the cause of the observed increase in cracking load. Higher increases in the cracking load were observed in the first eight months of exposure.

For the steel-reinforced concrete beams (Series A) the yielding of steel reinforcement in the control specimens started at a load level of 2 kN. Flexural cracking was initiated at a load level of 3.16 kN for the control specimen (12 months). A slight increase in cracking load in the first 12 months of exposure was observed; the average cracking loads were: 3.0 kN after 4 months; 3.9 kN after 8 months; 3.5 kN after12 months; and 3.0 kN after 18 months exposure.

The experimental results on concrete beams reinforced with ISOROD bar (Series B) suggest a small, insignificant, increase in the flexural cracking load with exposure time. At 18 months, the beams showed linear behavior up to the first crack at a load level that ranged between 3.0 kN and 3.1 kN.

A slight decrease in the ultimate failure load with exposure time was observed in the concrete reinforced with steel bar and sprayed with GFRP (Series C). The average ultimate failure loads for control specimens was 11 kN. The average ultimate failure loads at 4, 8, 12 and 18 months exposure were 9.6, 9.9, 10.4, and 9.9 kN, respectively. However, it should be noted that the ultimate failure load of the reinforced concrete beams sprayed with GFRP was about three times higher than the ultimate failure load of plain (uncovered) beams reinforced with steel (Series A) and about two times higher than the ultimate failure load for beams reinforced with ISOROD (Series B). In addition to the increases of the beam strength, the sprayed GFRP acted as a secondary protection layer for the reinforcement; the concrete cover of the reinforcement is well protected from contact with manure.

There were no notable differences in the load-deflection behavior of the steel-reinforced concrete beams covered with PVC panels between the control beam and those exposed for 18 months. The control specimen failed at an average load of 4.0 kN whereas the average ultimate load value for the exposed beam for 4, 8, 12 and 18 months exposure were 4.3, 3.8, 4.1 and 4.2 kN, respectively. Longer exposure time (18 months) to the manure environment did not appear to further affect the ultimate failure load or the cracking load. Steel-reinforced beams covered with PVC panels (series D) have been found to have a higher load carrying capacity than the steel-reinforced beams without cover (series A). The increase in ultimate load of those covered with PVC panels are attributed to the special profile (strip) embedded in the concrete during casting. The results suggested that the PVC panel strip when well embedded in the concrete acts as an additional reinforcement for the concrete. The ultimate failure load of the steel-reinforced beams covered with PVC was higher than those for the bare beams and lower than for those sprayed with GFRP.

Mercury intrusion porosimetry analysis on selected concrete specimens indicated that microstructural characteristics (i.e., pore volume, pore radius and pore size distribution) change with exposure time. The pore structure of the reinforced concrete governs to a large extent two of the most important engineering properties of the hardened concrete: 1) mechanical strength; and 2) permeability. The total pore volume and pore diameter decreased during the 4 and 8 months exposure and slightly increased during the10 months exposure. The observed changes were attributed to the continued hydration and therefore progressive densification of the concrete structure as manure progressively penetrated the specimen. Furthermore, microscopic examination of the concrete surface exposed to manure revealed that a distinctive feature of the reinforce concrete/manure interaction was the formation of a surface precipitate layer consisting mainly of Ca phases (i.e., portlandite, calcite), ettringite and new calcium silicate hydrate (CHS) within the first months of exposure. The chemistry of precipitates formed early on is controlled by the concrete and at the later stage, the precipitate composition is controlled by the chemistry of the manure. Under test conditions, GFRP reinforcement in the reinforced concrete showed no degradation. The examination of the cracked area showed that the GFRP bars are well bounded by the concrete, and that the bars failed by both fracture and pullout of the fiber. However, under stress conditions the polymer may ultimately lose its protection capability, possibly accelerating the degradation of the reinforcement.

One of the reasons for considering replacing the steel with GFRP for concrete reinforcement is that steel corrodes. However, use of the GFRP has its own problems; the alkaline pore solution may produce embrittlement of the matrix and damage at the fiber-polymer interface. The chemical processes involved during the curing of concrete create an environment (i.e., high pH) in which GFRP reinforcements are vulnerable to such attack. The increase in pH of the concrete is due to cement hydration which produces calcium silicate hydrates (CSH) and calcium hydroxides ($Ca(OH)_2$). The free lime as well as the alkali oxides react with water to increase the pH to values between 11.5 and 13.7. However, the final pH can be controlled in some extent by concrete mix design, the type of cement used, and the addition of pozzolanic materials (i.e., silica fume, fly ash). Furthermore, the pH in concrete decreases with maturation and carbonation. Greater resistance to the high pH can conceivably be achieved by using either an alkali-resistant polymer or alkali-resistant fibers, or a combination of both in the manufacture of GFRP.

Summing up the findings of this study, overall the GFRP reinforcement bars exhibited a high resistance to degradation in a manure environment. Consequently, penetration of the manure into the concrete through diffusion or through the inherent cracks that occur in concrete will have little influence on the degradation of the GFRP reinforcement bars. Furthermore, the manure-pore water exchange that will take place during tank operation will not sustain the high alkalinity in concrete over a long period. Reinforced concrete beams sprayed with GFRP performed the best among structural elements considered in the study. In addition to the increase in the beam strength, the sprayed GFRP acted as a secondary protection layer for the reinforcement. The use of GFRP materials as reinforcement (internal or external) in the construction of manure storage tanks is worthy of further consideration.

ACKNOWLEDGEMENT

Agri-Food Research and Development Initiative (ARDI), Manitoba Triple S Hog Manure Management Initiative, Manitoba Livestock Manure Management Initiative (MLMMI) and ISIS CANADA have jointly funded the work described in this report. Their financial support is greatly appreciated. The authors gratefully acknowledge support and assistance provided by Prof. Dr. Nemy Banthia of the University of British Columbia and Lafarge Canada Inc. in the preparation of the reinforced concrete specimens used in the test program. The authors would like to thank to Moray McVey from University of Manitoba for the technical assistance.

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