THE REHABILITATION OF A CONCRETE STRUCTURE
USING FIBRE REINFORCED PLASTICS

by

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A thesis submitted in conformity with the requirements for the degree of Master of Applied Science

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ABSTRACT

The Rehabilitation of a Concrete Structure Using Fibre Reinforced Plastics

Master of Applied Science, 1997

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Cracks in concrete may become significant if they begin to compromise the strength, durability, or aesthetics of a structure. Prior to the undertaking of any repair procedure, the cause of cracking must be established. Typical steps that should be included in a condition survey are outlined to assist in the diagnosis process.

As our nation's infrastructure continues to deteriorate, the need for more economical and more durable rehabilitation techniques has emerged. The use of Fibre Reinforced Plastics provides an attractive alternative to traditional rehabilitation techniques. The installation of these light-weight materials is less labour- and equipment-intensive. Fibre composites can also enhance durability as they prevent the passage of chlorides and other harmful chemicals. Finally, composites can considerably reduce the time that structures are taken out of service. Experimental results from short-term tests conducted on walls and beams indicate the effectiveness of carbon and glass composites for rehabilitating damaged structures.
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Severe cracking in concrete is usually a symptom of an underlying cause. Although most cracks do not result in structural failure, they can lead to accelerated deterioration rendering a structure unserviceable. Cracks are often classified in many ways. For instance, cracks classified by their direction may include: longitudinal, transverse, vertical, diagonal, and random. Furthermore, certain cracks can be distinguished since they form in maps or patterns in concrete. In addition, single continuous cracks may form that run in parallel at definite intervals. These cracks may result from restraint in the direction perpendicular to them.

Cracks can also be categorized by the movement they display. For instance, cracks that continue to increase in width because their original reason for occurrence persists are called growing cracks. Furthermore, cracks that open and close with loading or exhibit cyclical movement due to thermal effects are considered active cracks. Finally, dormant cracks are those that were caused by factors that are not expected to occur again and, as a
result, do not display any movement. Cracks resulting from initial drying shrinkage, or temporary overloading in service or construction may fit into this category.

Cracks may also be described by their widths. They may be categorized as hairline, narrow, medium, or large. One source distinguishes these cracks in the following manner. Hairline cracks are those less than 0.1 mm in width while narrow cracks fall into the range measuring from 0.1 to 0.3 mm\(^1\). Medium cracks measure between 0.3 and 0.7 mm while large cracks are usually those measuring over 0.7 mm\(^1\). Acceptable crack widths usually depend on the function of the structure and on the exposure to which the structure is subjected.

In general, cracks may become significant if they begin to compromise the strength, durability, or aesthetics of a structure. As a result, repair may be necessary. Prior to the undertaking of any repair procedure, the cause of cracking must be established. This is because the reason for cracking must often be eliminated to prevent cracking from re-occurring. There are many causes of cracking in concrete. Some of the more common ones may be of a physical, thermal, chemical, or structural nature as shown in Figure 1.1.

![Figure 1.1 - Causes of cracks in concrete\(^1\).](image-url)
In most cases, a condition survey may be necessary to properly diagnose the cause of distress or cracking. This may involve collection of background information and scheduling of several site visits. In addition, field testing, laboratory testing, and monitoring may be required to confirm or eliminate probable causes of deterioration. The final evaluation is often based on all of the accumulated information and often requires the experience or knowledge of various individuals.

Once a cause for cracking is established, the selection of an appropriate repair procedure may occur. The selection of a repair material and technique may be based on factors such as: durability, cost, appearance, substrate compatibility, time required for application, and ease of application. The repair technique selected must address the cause of cracking, the possibility of future movement, and the environment surrounding the crack.

These principles will be demonstrated on an existing structure. The distress in an apartment complex will be examined. As a result, an investigation is required to make certain decisions on the cause and extent of damage, the safety and serviceability of the structure, and the implementation of remedial measures.

As reinforced concrete structures continue to deteriorate, the need for more economical and more durable rehabilitation techniques has emerged. A remedial measure growing in popularity is the use of Fibre Reinforced Plastics. The use of these fibre composites can provide a number of advantages over traditional rehabilitation techniques. For instance, these light-weight materials are easy to install and, in turn, are less labour- and equipment-intensive. In most cases, repairs can proceed without taking the structure
out of service or halting industrial processes. In addition, these materials enhance durability as they prevent the passage of chlorides and other harmful chemicals. Finally, glass, carbon, and aramid composites possess high strength-to-weight ratios and can significantly enhance the strength of reinforced concrete sections.

In order to recommend the use of Fibre Reinforced Plastics in the field, it was decided to conduct appropriate tests on large specimens. Although tests were designed to simulate the specific structure in this study, the results can be extended to include the rehabilitation of other structures.
CHAPTER 2 - CAUSES OF CRACKS

2.1 GENERAL

In order to properly diagnose the causes of specific cracks, it is essential to understand the various types of cracks that may occur and their distinctive characteristics. Although there are many causes of cracks, this chapter includes some of the more common ones that can be found in concrete.

2.2 PHYSICAL CRACKS

Drying shrinkage is a common cause of cracking in concrete and excessive shrinkage is usually the result of a poorly designed concrete mix. It occurs through the loss of moisture in the cement paste. After concrete is cast, moisture can be either chemically combined in hydration products or otherwise removed through evaporation. As a result, the unrestrained cement paste can irreversibly shrink by as much as 1% per unit length$^2$. Fortunately, the amount of shrinkage in the cement paste is reduced by the presence of aggregate to about 0.05% per unit length$^2$. 
Concrete may continue to lose moisture and shrink with time unless kept under water or in air at 100% relative humidity. However, it must be noted that subsequent shrinking may not be as significant as initial drying shrinkage. Consequently, concrete will also expand on wetting since its porous nature offers a large internal surface area on which water may become absorbed. It is these moisture induced volume changes that are characteristic of concrete. If these volume changes occur without any restraint, the concrete will not crack. However, if another part of the structure or the subgrade offers restraint to volume changes, then tensile stresses are developed in the concrete. If these tensile stresses exceed the tensile strength of the concrete, cracking will occur. Crack propagation may then occur at stresses less than those required for crack initiation.

The time of appearance for these cracks is usually several weeks or several months after casting of the concrete. These cracks often run in parallel at definite intervals and indicate restraint in the direction perpendicular to them. Figure 2.1 shows an illustration of various crack types that can occur in a typical structure. The cracks denoted by the letter A represent the characteristic shape of shrinkage cracks.

In certain concrete elements, differential shrinkage may occur in the concrete. For instance, various parts of a wall in an underground structure may be exposed to different conditions. The outer surface may be kept moist by the soil while the inner surface may be exposed to dryer conditions. As a result, the larger shrinkage at the inner surface may cause cracks to develop that do not penetrate the entire wall.

The amount of drying shrinkage that may occur is often controlled by the amount and type of aggregate and the water content of the mix. In general, stiffer and larger
amounts of aggregate contribute to less shrinkage. In addition, the higher the water content of the concrete mix, the greater the amount of drying shrinkage\(^2\). Higher water contents may often result from the practice of adding water to the concrete on site to improve workability. As a result, the effects of drying shrinkage can be controlled in many ways. For instance, the maximum practical amount of aggregate and the lowest usable water content should be used in the concrete mix. Water contents can be reduced by using superplasticizers to improve workability. In addition, proper curing for an appropriate amount of time can also substantially reduce the amount of shrinkage. Finally, contraction joints and proper steel detailing should be provided to control shrinkage cracking.

![Diagram of potential crack types in a hypothetical structure](image)

Figure 2.1 - Potential crack types in a hypothetical structure\(^1\)\(^3\)\(^6\).
2.3 THERMAL CRACKS

Most materials will expand and contract with temperature changes and concrete is no exception. If concrete elements are restrained against this movement, serious stresses can result which may also lead to cracking. Once again, concrete will crack if induced stresses exceed the tensile strength of the concrete.

Concrete has a coefficient of thermal expansion that may range from 7 to $11 \times 10^{-6}$ per degree Celsius with an average of about $10 \times 10^{-6}$ per degree Celsius$^2$. As a result, differential movement may exist where there are large temperature gradients in concrete. For instance, some portions of a structure may be exposed to external conditions while other portions of a structure may be protected from these external conditions. In addition, solar radiation may increase temperature gradients even further$^4$. Consequently, this differential movement may lead to cracking in concrete elements.

The time of appearance for these cracks is usually several months or years after casting of the concrete. These cracks are similar to shrinkage cracks and often run in parallel at definite intervals. They tend to indicate restraint in the direction perpendicular to them. Cracks resulting from thermal effects may take on the appearance of those labeled by the letter B in Figure 2.1. These problems may be alleviated by allowing for movement by using contraction joints and proper detailing.

Cracking in concrete may also occur due to freezing of water in the cement paste or in the aggregate. This damage is caused by the movement of water to freezing sites and by hydraulic pressure generated by the growth of ice crystals$^2$. Saturated aggregates may be surrounded by cement paste that prevents the rapid escape of water. The expansion of
the absorbed water during freezing may crack the surrounding cement paste. Further damage may result upon subsequent cycles of freezing and thawing. Often cracking is accompanied by surface scaling or popouts.

The time of appearance of these cracks will usually depend on the length of time between freezing and thawing cycles. For instance, certain portions of a structure that are exposed to the sun may show the first signs of damage. This is due to the fact that the sun may promote thawing even in cold weather. Freezing cycles may occur at night while thawing cycles may occur during the day. As a result, the time between freezing and thawing cycles would be very short. In turn, damage would become evident much sooner.

The amount of damage due to freezing and thawing is often minimized through the selection of quality aggregate and the use of air-entrainment in the concrete. A proper period of curing prior to exposure to freezing conditions is also advisable. Finally, allowing the concrete to dry after adequate curing will reduce the likelihood of damage due to freezing and thawing.

2.4 CHEMICAL CRACKS

Cracking of concrete may also result from a number of chemical reactions. These reactions may be due to materials used in the concrete mix or elements that the concrete is exposed to after hardening. Some of the more serious chemical reactions that may occur in concrete are the corrosion of reinforcement and alkali-silica reaction (ASR).

Chemical reactions may occur in reinforced concrete that is subjected to the ingress of external agents. Concrete protects embedded steel from corrosion since
Concrete is high in alkalinity. This high pH environment causes a passive and noncorroding protective oxide film to form on steel. However, chloride ions from de-icing salts or seawater can approach the steel and destroy or penetrate this film. Once the film is penetrated, an electric cell is formed along the steel and the electrochemical process of corrosion begins.

Corrosion is a process that requires an oxidizing agent, moisture, and electron flow within the metal. As part of the process, a number of reactions take place on and near the surface of the metal. This is due to regions of different electrochemical potential known as anodes and cathodes that develop on the surface of a metal. At the anodes, metal atoms lose electrons forming ions that go into solution. At the cathodes, oxygen and water combine with free electrons to form hydroxyl ions. These hydroxyl ions move towards the anodes and combine with the metal ions to form hydrous metal oxides. In the case of steel, iron oxides and hydroxides form as a deposit at the anodes.

The volume occupied by the corrosion products is greater than that of the original steel. This expansion puts the surrounding concrete in tension and initially leads to cracking. These cracks will usually appear several months or years after the start of the corrosion process. They can propagate along reinforcing bars resulting in longitudinal cracks parallel to corroding bars as shown in Figure 2.1 near letter C. Initially, these cracks may have small widths. However, they provide an easy route for oxygen, moisture and chlorides. As a result, further deterioration can occur. Rust staining may become visible in wet conditions and crack widths may increase with time. Eventually, delamination or spalling of the concrete cover will occur.
Protection of metal can result by stopping or reversing these chemical reactions. This can be achieved by eliminating the oxygen and moisture required for the reaction or by reversing the electron flow. Some options include sealers or overlays applied to the concrete surface, coated reinforcement, and cathodic protection.

ASR is a concrete deterioration process that involves the reaction between active silica constituents of certain aggregates and the alkalis in cement. The reaction begins when alkaline sodium or potassium hydroxides derived from the cement paste attack the siliceous minerals in the aggregate. This produces an alkali-silicate gel that absorbs large amounts of water resulting in an increase in gel volume. The expanding gel may fill pores and voids in neighbouring aggregate particles and cement paste but, most often, the expanding gel causes large internal pressure and localized micro-cracking occurs.

The inside of a concrete element will expand more than the surface since alkalis are often leached from the surface. As a result, a surface pattern of random or "map" cracking occurs as a result of differential movement. Typically, these cracks will be similar to the ones labeled by the letter D in Figure 2.1. These cracks usually appear beyond five years of service and are usually concentrated in the parts of a structure exposed to moisture.

There are many options currently available to avoid the destructive expansion and cracking of concrete due to ASR. These include the use of nonreactive aggregates, the control of alkali content in cement, and the use of supplementary cementing materials.
2.5 STRUCTURAL CRACKS

Load-induced tensile stresses cause cracking if the tensile capacity of the concrete is exceeded. To eliminate this cause of cracking entirely would be very difficult. As a result, the use of reinforcing steel is not only to carry tensile forces but to obtain an adequate crack distribution and a reasonable limit on crack width. Internal stresses may also arise from poor detailing or improper foundation design.

The loads that concrete elements may experience during construction may exceed those experienced in service. Furthermore, this loading occurs at a time when these concrete elements have not attained their full strength. As a result, permanent damage may occur in the form of cracking. This damage may occur to cast-in-place concrete that must support subsequent floors at an early age or pre-cast members that are not adequately supported during transport or erection. This damage can only be prevented if designers specify load limitations and if contractors follow these instructions.

Service loading can also lead to cracking. The time of appearance for these cracks will tend to depend on the usage of the structure. A crack will form in concrete when the principal tensile stress reaches the cracking strength of the concrete. The crack will form normal to the direction of the principal tensile stress. As a result, principal tensile stresses are parallel with the longitudinal axis for concrete members subjected to pure axial tension or pure flexure. In turn, resulting cracks would form perpendicular to the member axis. For members subjected to shear stresses, the principal tensile stress directions are inclined to the longitudinal axis of the member. Accordingly, shear cracks would be inclined to the member axis and are sometimes referred to as diagonal cracks\(^7\).
Figure 2.2 shows the various types of cracks that may form according to specific loading configurations. In addition to flexure, tension, and shear cracks, other cracks may also result due to service loading. Torsion cracks may occur if twisting action is present. Furthermore, cracks parallel to reinforcement may initiate where bond stresses are high between the steel reinforcement and the concrete or where concentrated loads are present.

If the design for strength is satisfactory, the preceding crack widths will usually be small. In general, they will be less than 0.4 mm². The following general conclusions can also be made. Crack width increases with increasing steel stress, cover thickness, and area of concrete surrounding each reinforcing bar. Furthermore, flexural and tensile crack widths can be expected to increase with time for members subjected to either sustained or repetitive loading.

![Diagram of various types of load-induced structural cracks](image)

**Figure 2.2 - Various types of load-induced structural cracks.**
For load-induced cracks, well-distributed reinforcement offers protection. A larger amount of smaller bars will tend to reduce stress concentration and, in turn, reduce the amount of cracking. Although a reduced cover will act to decrease crack widths, it must be kept in mind that this may reduce the corrosion protection of the steel.

Serious problems in structures may occur at monolithic beam-column interfaces or slab-column interfaces where poor design or reinforcement detailing has been specified. For instance, extensive radial and circumferential cracking may occur on the top surface of a flat plate deck slab situated over a column. As a result, a dangerous situation is set up where the observed cracking may eventually lead to a punching shear failure. This type of situation can be avoided with the use of a column capital to increase the area in which load is transferred between the column and the slab.

Differential movement through settlement may occur in a structure where an improper foundation design has been specified. Cracking will usually occur with small differential movements. Larger differential settlements may result in element failure if the structure is not able to redistribute resulting loads rapidly enough.

Finally, cracks can be caused by stress concentrations as shown in Figure 2.3. The tendency is for internal stresses to flow around openings causing splitting forces. Corners of doors, windows, and other openings are usually susceptible to the initiation of cracks if provisions are not made to accommodate the stresses that accumulate here. As a result, additional diagonal reinforcement may keep these cracks narrow and prevent them from propagating.
Figure 2.3 - Cracks resulting from stress concentrations at corners\textsuperscript{1,2}.
CHAPTER 3 - THE CONDITION SURVEY

3.1 GENERAL

The most difficult and important step in the repair process is to determine the cause of deterioration. If the cause is not understood, it is not possible to justify a need for repair or to select a repair procedure. Causes are difficult to identify in concrete since there may be insufficient data or because several deterioration processes may have occurred simultaneously. As a result, a well-planned field and laboratory test program is essential to obtain information on the extent of damage and to establish the cause of deterioration.

For this study, information was gathered from a review of design, construction, and service records. This review was then complemented by a field investigation that included nondestructive testing, sample collection for laboratory testing, and monitoring.
3.2 **TASK 1 - COLLATION OF INFORMATION**

The structural components of the entire complex are predominantly made of reinforced concrete. The parking structure contains two interior suspended levels, two interior suspended ramps, and two slabs on grade. The approximate dimensions of the parking structure are approximately 80 m in length and 38 m in width. A typical cross-section is shown in Figure 3.1. The suspended parking levels and one level on grade are waterproofed and protected with a mastic asphalt wearing course. It is also worthy to note that there are no expansion joints in the parking structure.

![Figure 3.1 - Typical building cross-section.](image)

Several reports\textsuperscript{3,8-18} have been written since 1994 on the distress experienced by the structure. Most of the damage is concentrated in the parking levels and at a number of reinforced concrete beams near the main entrance. The following is a brief overview describing some of this damage. A layout of parking levels P4 and P3 complete with Grid Lines has been included in Appendix A to simplify the descriptions of the locations of distress.
3.2.1 PARKING LEVEL P4

There are approximately 30 vertical cracks on the interior face of the West foundation wall (Grid Line T - Appendix A). These cracks are easily noticeable and most run parallel to each other at almost regular intervals. Cracking is more severe on the Northern portion of this wall. In the most damaged zone (between Grid Lines 8 and 9), crack widths range from hairline to 0.75 mm and average approximately 0.3 mm in width. Certain cracks in this zone show the occasional leakage of water as is shown in Figure 3.2. This seems to suggest that these cracks penetrate the entire wall.

Figure 3.2 - The leakage of water through a vertical crack.
Many of these vertical cracks in the wall join other cracks that run in an East-West direction on the slab soffit (i.e., ceiling of P4). In total, there are approximately 50 cracks that run parallel to the East-West line. Most of these cracks are concentrated between Grid Lines T and V. Their widths generally vary from hairline to approximately 0.7 mm while their lengths vary from approximately 2 m to 6 m. Once again, the most damaged area is found in between Grid Lines 8 and 9. This zone does not contain the widest cracks but the largest number of them. There are approximately 10 cracks in this 6.5 m span. The average crack width is approximately 0.24 mm while the total width of these cracks is between 2.5 and 3 mm. It should also be noted that there are a large number of diagonal and random cracks on the slab soffit that are, for the most part, hairline to narrow.

![Diagram](image)

**Figure 3.3 - Typical crack pattern for a slab soffit.**

In the second span over from the West foundation wall (between Grid Lines U and V), the slab soffit contains 1 or 2 long North-South or longitudinal cracks. They are located close to the center of the span and run for approximately the entire 80 m length of the building. In certain areas, the widths of these cracks approach 0.5 mm. These North-
South cracks meet some East-West cracks in many areas. A typical pattern of larger cracks for the underside of a slab soffit is shown in Figure 3.3. It must be noted that the top portion of these slabs could not be inspected since they are covered with an asphalt topping.

Two horizontal cracks are also found on the West foundation wall. The lower crack is located at roughly 0.6 m from the slab-on-grade. It runs from Grid Line 10 to 15X and assumes a diagonal direction as it approaches the last few meters from the North end. The upper horizontal crack is located at 1 m from the slab-on-grade. It runs from Grid Line 11 to 15X. At most locations, these horizontal cracks are hairline but approach 0.3 mm in certain regions. Figure 3.4 is a photograph of the West foundation wall at the intersection of Grid Line 15 and Grid Line T. In this region, there are actually 3 horizontal cracks intersecting a vertical crack. The third crack is located at approximately 0.3 m from the ground and runs for about 1.5 m.

Severe cracking was also observed between Grid Lines U and V on Grid Line 15X. This location contains a wall that is situated at the bottom of a stairwell. This wall contains diagonal cracks that measure approximately 1.8 to 2 mm in width.

Finally, the slab on grade in Parking Level P4 has undergone an upward displacement in the northern region. As a result, the slab on grade features several severe cracks. In addition, there is a stepped edge at the junction between the slab on grade and the raft foundation. These defects have created an uneven driving surface over an area measuring 55 meters in length and 4.1 meters in width.
The distress in this parking level is similar to that found in parking level P4. For instance, there are approximately 28 vertical cracks on the interior face of the East
foundation wall (Grid Line Z). Crack widths range from hairline to 1.1 mm and average approximately 0.3 mm in width.

Some of these vertical cracks in the east wall join other cracks that run in an East-West direction on the slab soffit (i.e., ceiling of P3). In total, there are approximately 25 cracks that run parallel to the East-West line. Most of these cracks are concentrated between Grid Lines X and Z. Their widths generally vary from hairline to 0.65 mm while their lengths vary from approximately 2 m to 6 m. In addition, there are a large number of diagonal and random cracks on the slab soffit that may be classified as hairline or narrow. For instance, at the North East corner of P3, the underside of the slab shows a diagonal crack indicating corner separation.

In the second span over from the East foundation wall (between Grid Lines X and Y), the slab soffit contains 1 or 2 main North-South cracks. They are located close to the center of the span and run for approximately the entire 80 m length of the building. The widths of these cracks range from hairline to approximately 0.3 mm. These North-South cracks meet some East-West cracks in many areas. The typical pattern of large cracks for the underside of a slab soffit can, once again, be seen in Figure 3.3.

Two horizontal cracks are also found on the East foundation wall. The lower crack is located at roughly 0.7 m from the slab-on-grade while the upper horizontal crack is located at 1.1 m from the slab-on-grade. For the most part, these cracks run for the entire length of the building. However, at certain locations only one of the horizontal cracks is present. At most locations, these horizontal cracks are hairline to narrow but approach 0.4 mm in certain regions.
3.2.3 PARKING LEVELS P2 AND P1

These levels exhibit cracking patterns similar to those found in levels P4 and P3. For instance, levels P2 and P1 also contain vertical wall cracks, East-West slab soffit cracks, North-South slab soffit cracks, and diagonal slab soffit cracks. The main difference is that these cracks are not as wide or severe as the ones found in levels P4 and P3. In addition, levels P2 and P1 do not exhibit any horizontal wall cracks. Finally, many of the wall and slab soffit cracks in levels P2 and P1 reveal water leakage especially during the thawing season.

3.2.4 SECOND-FLOOR BEAMS

The structural components in this building complex include a series of 28 reinforced concrete beams. They are located at the second floor level in the East-West direction and extend beyond the building envelope as shown in Figure 3.1. These beams are supported by circular columns at their external end and on columns or walls at their internal end. Most of the beams support part of the second floor slab and walls, a concrete wall, and load from upper floors.

There are two beams on the East side of the building that show excessive diagonal cracking. These beams are located on both sides of the main entrance or on Grid Lines 8 and 9. The beam on Grid Line 8 has a number of diagonal cracks on both faces that join together on the bottom face. These cracks start from the column side at the bottom and travel diagonally upward to the wall above as shown in Figure 3.5. These cracks vary from hairline to 0.7 mm in width. The beam on Grid Line 9 has similar diagonal cracks.
but they vary in width from hairline to 0.8 mm. Other diagonal, hairline cracks have been observed in the beams on the East side of Grid Line 4 and on the West side of Grid Line 10.

![Image of diagonal cracking on a second-floor beam]

**Figure 3.5 - Diagonal cracking on a second-floor beam.**

Several columns below the external ends of the second floor beams display a number of horizontal cracks on their exterior faces. In general, these cracks start at about 200 mm from the underside of the beam and other cracks may go down to about 1300 mm. The columns situated on the sides of the main entrance show the most distress. The horizontal cracks on the column found on Grid Line 8 vary from hairline to 0.2 mm. However, the cracks on the column found on Grid Line 9 vary from hairline to 0.5 mm.
3.3 **TASK 2 - ESTABLISHMENT OF SERVICE AND EXPOSURE CONDITIONS**

3.3.1 **WALLS AND SLABS**

Spot checks on the structural drawings indicate that the design of walls and slabs generally meet the code requirements for thickness and the amount of steel for resisting the applied loads\(^3\). However, field tests were deemed to be necessary to determine the as-built conditions. For instance, material properties, geometric properties, and steel detailing, were all required in order to arrive at the real causes of distress.

It must be noted that a large opening exists in the North-West corner of parking level P4. During the winter, this area often becomes the coldest region in the entire garage. As a result, this area is exposed to large temperature fluctuations and should be designated as a problem zone.

A number of areas in this parking garage are in contact with moisture. Water leakage is sometimes seen through a number of cracks in the East and West end walls of P1, P2, P3 and P4, and in the slab soffits of P1 and P2. This may signal significant crack depth and inadequate water proofing. The main concerns in these areas are freeze-thaw damage in the concrete, and deterioration of the reinforcement and concrete due to possible corrosion in the future.
3.3.2 SECOND-FLOOR BEAMS

A further review of the structural drawings seems to suggest the potential for problems with the second-floor beams. To begin with, designs are inconsistent. It seems that the two beams symmetrically situated around the main entrance have totally different reinforcement detailing as shown in Figure 3.6. The beam on the North side of the entrance is denoted as 2B15 while the beam on the South side is denoted as 2B11. Beam 2B11 has different detailing along the span of the beam that is much more labour-intensive than Beam 2B15. The problem with this situation is that the simpler design may have been adopted for both beams in the field.

In addition, Beam 2B15 has an inadequate amount of transverse or shear reinforcement. According to clause 11.2.8.4 of A23.3-94[19], the minimum area of shear reinforcement that was supposed to be provided was Number 10 bars at 350 mm (for $f'c=35$ MPa). However, the design of Beam 2B15 prescribes a spacing of 400 mm.

Another area of concern has to do with the fact that parts of the beams extend beyond the building envelope into an unprotected environment. As a result, part of the span is exposed to moisture, wind, and temperature fluctuations while the remaining portion is sheltered. Certain portions of these beams also support concrete load-bearing walls that are rigidly connected. As a matter of fact, the span of the beams near the main entrance is shorter than the span of all other beams. Consequently, the beams near the main entrance are less flexible than other beams and are more prone to shear cracking. The combination of all of these factors seriously compromise the durability of these beams.
Figure 3.6 - Reinforcement detailing and spans for two second-floor beams.

All dimensions in mm
3.4 TASK 3 - THE DETAILED SURVEY

3.4.1 PART A - NONDESTRUCTIVE TESTING (NDT) AND CONCRETE CORE SAMPLING

Visual inspection is one of the most important NDT methods. The visual condition of the parking levels and second-floor beams was observed. Cracking was easily noticeable in all areas outlined in previous reports. Patterns were noticeable in vertical wall cracks and East-West slab cracks. No spalled concrete or exposed reinforcement was detected. The leakage of water was seen through certain wall and slab cracks. Serious heaving was also noticed in the slab-on-grade in the North-West corner of P4. It should be noted that no control or expansion joints were visible in the walls or suspended slabs of the entire parking garage. In addition, it was evident that a small number of cracks had been epoxy injected. This was, more than likely, an attempt to test the effectiveness of this procedure as a repair alternative.

Limited measurements of crack width and length were carried out and were, for the most part, consistent with previous reports. The location and width of cracks had previously been mapped out\textsuperscript{3,17}. Since only visible surfaces could be inspected, it was essential to perform other nondestructive tests and to extract concrete cores. This would provide valuable insight into the as-built conditions of the structure.

Consulting Engineers\textsuperscript{15} were requested to carry out a covermeter and coring survey of the parking garage. The objectives for this survey were:
1. To determine the location and concrete cover of the bottom reinforcing steel in the suspended slabs in two bays.

2. To determine the location and concrete cover of the inner layer of reinforcing steel in the perimeter walls at two bays.

3. To remove concrete core samples from the suspended slabs and perimeter walls for compressive strength testing. To document the condition of the waterproofing system on the interior suspended level.

4. To excavate several areas (Test Pits) in the landscaped region over the garage and examine the construction details and condition of the waterproofing system.

5. To measure the concrete slab and wall thickness at several random locations.

The following locations were chosen to be representative of typically cracked bays:

- Slab Test Area 1 - Between Grid Lines 14-15X and T-U on level P2 (*Ceiling of P4)
- Slab Test Area 2 - Between Grid Lines 10-11 and X-Y on level P1 (*Ceiling of P3)
- Wall Test Area 3 - Between Grid Lines 14-15X on Line T (*Level P4 to P2)
- Wall Test Area 4 - Between Grid Lines 11-12 on Line Z (*Level P3 to P1)

Geotechnical Engineers were also contracted to carry out a geotechnical investigation of the slab-on-grade to reveal the subsurface conditions and to determine the probable cause of the movement of the slab.

3.4.1.1 Reinforcing Bar Spacing and Concrete Cover

The concrete cover and spacing of the reinforcing bars in the four test areas are indicated in the following tables. For the wall regions, it should be noted that the concrete cover at the bottom of the wall is generally greater than at the top of the wall.
Table 3.1 - Slab Reinforcing Bars\textsuperscript{15}.

<table>
<thead>
<tr>
<th>Reinforcing Bar Direction</th>
<th>Area 1</th>
<th>Area 1</th>
<th>Area 2</th>
<th>Area 2</th>
<th>Required</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Cover Range</td>
<td>N-S</td>
<td>E-W</td>
<td>N-S</td>
<td>E-W</td>
<td></td>
</tr>
<tr>
<td>Concrete Cover Range</td>
<td>25-45</td>
<td>23-55</td>
<td>18-45</td>
<td>26-53</td>
<td>25mm(N-S)</td>
</tr>
<tr>
<td>Avg. Concrete Cover</td>
<td>32</td>
<td>36</td>
<td>28</td>
<td>42</td>
<td>35mm(E-W)</td>
</tr>
<tr>
<td>Cover Standard Dev.</td>
<td>5</td>
<td>10</td>
<td>8</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>Design Bar Spacings</td>
<td>200,</td>
<td>150,</td>
<td>230,</td>
<td>250,</td>
<td></td>
</tr>
<tr>
<td></td>
<td>215,250</td>
<td>200,250</td>
<td>275,</td>
<td>375,</td>
<td></td>
</tr>
<tr>
<td>Average Bar Spacing from</td>
<td>263</td>
<td>254</td>
<td>340</td>
<td>312</td>
<td></td>
</tr>
<tr>
<td>investigation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

All dimensions are in mm.

Table 3.2 - Wall Interior Reinforcing Bars\textsuperscript{15}.

<table>
<thead>
<tr>
<th>Reinforcing Bar Direction</th>
<th>Area 3</th>
<th>Area 3</th>
<th>Area 4</th>
<th>Area 4</th>
<th>Required</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Cover Range</td>
<td>35-78</td>
<td>23-60</td>
<td>45-60</td>
<td>28-60</td>
<td>25mm(Vert.)</td>
</tr>
<tr>
<td>Avg. Concrete Cover</td>
<td>49</td>
<td>36</td>
<td>54</td>
<td>44</td>
<td>35mm(Hor.)</td>
</tr>
<tr>
<td>Cover Standard Dev.</td>
<td>11</td>
<td>10</td>
<td>6</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>Bar Spacing Range</td>
<td>390-480</td>
<td>310-390</td>
<td>290-412</td>
<td>280-364</td>
<td>325mm(Vert.)</td>
</tr>
<tr>
<td>Average Bar Spacing</td>
<td>403</td>
<td>350</td>
<td>374</td>
<td>334</td>
<td>400mm(Hor.)</td>
</tr>
<tr>
<td>Bar Spacing Stand. Dev.</td>
<td>48</td>
<td>22</td>
<td>44</td>
<td>23</td>
<td></td>
</tr>
</tbody>
</table>

All dimensions are in mm.

3.4.1.2 Coring Results

Nine cores were taken from the slab regions. The waterproofing membrane on top of the suspended levels measured 2 mm in thickness while the mastic asphalt measured 15 mm in thickness. These measurements were consistent at all locations. The bond of the waterproofing membrane to the concrete and asphalt was found to be good at all locations. A reinforcing fabric was found between the mastic asphalt and waterproofing membrane. None of the reinforcement encountered during drilling was corroded.
Seven cores were taken from the wall regions. Certain cores were taken over cracks in order to investigate their depth of penetration. Cracks penetrated to 125 mm at one location (diagonal crack drilled to 220 mm), to 220 mm at another location (vertical crack drilled to 220 mm), and to 160 mm at a third location (horizontal crack drilled to 160 mm).

The results from the cores, tested for compressive strength, are given in Table 3.3. The required concrete compressive strength for both slabs and walls was 35 MPa.

Table 3.3 - Compressive Strength

<table>
<thead>
<tr>
<th>Core Number</th>
<th>Compressive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 (P2 Slab)</td>
<td>40.7</td>
</tr>
<tr>
<td>3 (P2 Slab)</td>
<td>28.8</td>
</tr>
<tr>
<td>4 (P2 Slab)</td>
<td>36.7</td>
</tr>
<tr>
<td>5 (P1 Slab)</td>
<td>50.6</td>
</tr>
<tr>
<td>6 (P1 Slab)</td>
<td>47.1</td>
</tr>
<tr>
<td>7 (P1 Slab)</td>
<td>42.3</td>
</tr>
<tr>
<td>Average:</td>
<td>41.0</td>
</tr>
<tr>
<td>10 (P4 Wall)</td>
<td>66.2</td>
</tr>
<tr>
<td>13 (P4 Wall)</td>
<td>44.9</td>
</tr>
<tr>
<td>15 (P4 Wall)</td>
<td>48.3</td>
</tr>
<tr>
<td>Average:</td>
<td>53.1</td>
</tr>
</tbody>
</table>

3.4.1.3 Test Pit Results

Four test pits were dug in the landscaped portion of the roof slab at the East and West sides of the building. A rubberized asphalt membrane ranging from 5 to 7 mm was found on the slab surface and upturn. This membrane was situated below layers of soil,
limestone screenings, a filter cloth, rounded stone, and a plastic protection board. The bond of the membrane to the concrete was good at one location and fair to poor at two other locations. The bond at the upturn was generally poor. It should also be noted that gravel was found under the protection board embedded in the membrane.

### 3.4.1.4 Concrete Slab and Wall Thickness

Impact-echo equipment was used to measure the thickness of the concrete slabs and walls. The design slab thickness was 200 mm but the measured thickness ranged from 181 mm to 209 mm and averaged approximately 202 mm (Stand.dev.= 6.5 mm). Slab thickness less than 195 mm was located in the North-West portion of level P2 (Ceiling of P4). The design wall thickness was 250 mm but the measured thickness ranged from 223 mm to 272 mm and averaged approximately 244 mm (Stand.dev.=17.8 mm). However, the thickness of the wall at Test area 3 ranged from 223 mm to 236 mm and averaged 228 mm (Stand.dev.= 6 mm). Once again, this is the North-West corner of the parking level.

### 3.4.1.5 Slab-On-Grade

A series of boreholes were dug into various locations of the concrete slab to reveal subsurface conditions. In addition, standard penetration tests were carried out at regular intervals of depth. Results from these tests are available in Reference 9. Some of their more important findings include the presence of frozen subgrade in two boreholes. In addition, frost-susceptible silty-soil was also detected in certain portions of the boreholes. As a result, it appears that the eastern part of the slab-on-grade in P4 rose due to frost.
heave while the western portion settled due to rotation or consolidation of the wall foundation backfill.

3.4.1.6 Summary

A comparison of some of the survey results was made with the "issued for construction" drawings and the details are listed in the following table.

Table 3.4 - Comparison of survey results with design requirements

<table>
<thead>
<tr>
<th>Description</th>
<th>From Drawings</th>
<th>From Survey</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab Thickness</td>
<td>200 mm</td>
<td>181-209 mm, Avg-202 mm</td>
</tr>
<tr>
<td>Wall Thickness</td>
<td>250 mm</td>
<td>223-272 mm, Avg-244 mm</td>
</tr>
<tr>
<td>Slab Compress. Strength</td>
<td>35 MPa</td>
<td>28.8-50.6 MPa, Avg-41 MPa</td>
</tr>
<tr>
<td>Wall Compress. Strength</td>
<td>35 MPa</td>
<td>44.9-66.2 MPa, Avg-53.1 MPa</td>
</tr>
<tr>
<td>Wall Bar Concrete Cover</td>
<td>25 mm(Vert.Bars)</td>
<td>23-60 mm, Avg-40 mm</td>
</tr>
<tr>
<td>Wall Bar Spacing</td>
<td>400 mm Horiz.</td>
<td>290-480 mm, Avg-388 mm</td>
</tr>
<tr>
<td></td>
<td>325 mm Vert.</td>
<td>280-390 mm, Avg-342 mm</td>
</tr>
<tr>
<td>Slab Bar Concrete Cover</td>
<td>25 mm(N-S Bars)</td>
<td>18-45 mm, Avg-30 mm</td>
</tr>
</tbody>
</table>

3.4.2 PART B - MONITORING CRACK MOVEMENT

The measured response of a structure to changes in temperature, loads, and internal conditions may provide clues to finding the causes of deterioration. The data obtained can also verify whether certain cracks are active, dormant, or growing. This information is also helpful in the selection of a repair procedure.

Although crack movement can be monitored in several ways, the mechanical arrangement shown in Figure 3.7 was selected for its simplicity and accuracy. The figure also shows Zurich targets mounted on both sides of a crack. An initial reading of the
distance between the targets was taken at the time of installation. This was accomplished with the specialized device (Pfender Gauge) also shown in the figure. Subsequent readings were then compared to the initial reading to estimate crack movement. All distances were rounded to the closest one-thousandth of a millimeter.

Figure 3.7 - Zurich targets and Pfender Gauge used to monitor cracks.

Target locations were selected to be representative of the various types of cracks encountered in the structure. All targets were numbered and their locations were recorded according to the Grid Lines found in Appendix A. A complete list of target locations has been included in Appendix B. Only cracks in parking levels P4 and P3 were monitored since these were more severe than the ones found in levels P2 and P1.
Locations 1 to 12 correspond directly to the zones that were surveyed by the Consulting Engineers. Appendix B also lists approximate crack widths at the time the targets were installed.

Targets were also installed to monitor specific cracks on the second-floor beams and columns. These targets were installed around the main entrance as shown in Figure 3.8. However, column targets were installed several months after the initial installation on the beams or in the parking levels. As a result, the column data is not as extensive as the other locations.

Figure 3.8 - Target locations at main entrance.

Readings were taken at random intervals over a period of approximately twenty months. Collected data was plotted on graphs of “Crack Width Versus Time”. Graphs were categorized in terms of the type of crack and the parking level on which the cracks
were found. For instance, all horizontal wall cracks in parking level P4 were grouped together and so forth. All of the graphs have been included in Appendix C.
CHAPTER 4 - EVALUATION OF CRACKS

4.1 GENERAL

The objective of this chapter is to identify the cause and extent of damage throughout the lower parking levels and on the exterior beams. For simplicity, the damaged areas will be classified into categories according to crack pattern and location.

4.2 SLAB CRACKS

4.2.1 EAST-WEST SLAB CRACKS (Slab Soffits of P4 and P3)

There appears to be a number of factors that have contributed to this damage. Since the majority of these cracks ran in parallel at regular intervals, they seemed to be a result of the stresses that were induced into the slabs due to the restraint to volume changes in the reinforced concrete. These volume changes may have been caused by drying shrinkage, thermal effects, or the combination of both. This claim may be verified by looking at specific data collected during the condition survey.
A statistical analysis was performed on compressive strength test data that was available from the time of construction. The analysis was performed by a Concrete Consultant in accordance with ACI 214 "Recommended Practice For Evaluation of Strength Test Results of Concrete". It must be noted that only data pertaining to suspended parking slab cores was readily available. The results of the analysis are as follows:

No. of results: 30  
Specified Strength: 35 MPa  
Mean Strength: 34.35 MPa  
Standard Deviation: 3.78 MPa  
No. of results below 35 MPa: 11  
Required Mean Strength to meet CSA requirements: 40.6 MPa

This analysis showed that the concrete supplied to the site did not meet contract requirements at 28 days. It was suspected that water was added to the mix on site in order to improve workability. The addition of water to a mix can lower concrete strength, increase the amount of shrinkage and, in turn, increase the likelihood of cracking. In one instance, the slump was found to be as much as 60 mm greater than the specified 80 mm while compressive strengths were found to be as low as 72% of the 28 day strength. As a result, it is likely that these factors contributed to the East-West Cracks in the suspended parking slabs.

Crack movements obtained during the monitoring period also provided valuable insight. Data pertaining to the suspended slabs are included in Appendix C as Graphs C4, C5, and C10. It was readily appreciated that the plotted data was cyclical in nature and indicative of seasonal variations. As a result, an effort was made to predict the movement
of cracks in response to changes in ambient temperature. Graphs of "Change in Crack Width Vs. Time Due To Temperature" were plotted and included in Appendix D.

Predicted values were calculated using the following relation:

$$\delta W = \alpha_c \cdot \delta T \cdot TL,$$

where $\delta W$ = the change in crack width in mm, $\alpha_c$ = the coefficient of thermal expansion of concrete which was taken as $10 \times 10^{-6}$/°C, $\delta T$ = ambient temperature at the time of the reading minus the initial ambient temperature when the targets were installed, and $TL$ = the length of concrete that would contribute to the movement in a crack or the "Tributary Length".

![Figure 4.1 - Expression to calculate the Tributary Length.](image)

Positive values for changes in crack width represented opening of the cracks. In addition, if the distance between cracks was larger, then the tributary distance would be larger, and as a result, more movement would be displayed at a crack upon concrete expansion and contraction. Tributary lengths were calculated according to the expression in Figure 4.1.
It was shown by comparing Graphs C4, C5, and C10 to Graphs D4, D5, and D10 that the movements across the cracks were mostly temperature related. This data suggested that thermal effects may have contributed to the formation of these cracks and continue to contribute to their movement. Although these cracks were considered active, there was no current evidence to suggest that they were growing. In fact, crack widths returned to their initial values when ambient conditions reverted back to those found when the targets were first installed. This information should be valuable for the selection of a repair alternative. In this case, a repair technique that could satisfy these cyclical movements would have to selected.

Although trends were found to be comparable, it was found that crack movements were actually less than those predicted. This may be explained by the presence of restraints in the field. For instance, the suspended slabs are monolithically connected to reinforced concrete walls and columns that limit the amount of expansion or contraction to values less than those predicted.

4.2.2 NORTH-SOUTH SLAB CRACKS (Slab Soffits of P4 and P3)

The North-South cracks on the underside of the suspended slabs cover almost the entire length of the structure. It was speculated in a previous report\(^3\) that these cracks were structural cracks since they coincided with regions of maximum moment. As a result, data from the condition survey was analyzed in order to confirm this opinion.

Data was obtained by Consulting Engineers in two slab regions of P4 and P3. This data revealed that the amount and spacing of reinforcement generally corresponded to the
construction drawings\textsuperscript{15}. The slab thicknesses in the vicinity of the North-South cracks were also within a few millimeters of the specified thickness. However, the concrete cover to the bottom reinforcement in these slabs ranged from 18 to 45 mm with a mean of 30 mm and a standard deviation of approximately 8 mm. This varied widely from the specified concrete cover of 25 mm. Since it was already shown that concrete strength did not meet contract requirements, the cumulative effect was a decrease in the structural capacity of the suspended slabs.

Table 4.1 - Concrete Strength\textsuperscript{16}.

<table>
<thead>
<tr>
<th>Date Cast</th>
<th>Age on Test Day (days)</th>
<th>Avg. Compressive Strength (MPa)</th>
<th>Avg. 7-Day Compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>April 8</td>
<td>2</td>
<td>21.2*</td>
<td>26.8</td>
</tr>
<tr>
<td>10</td>
<td>3</td>
<td>23.7*</td>
<td>29.0</td>
</tr>
<tr>
<td>14</td>
<td>2</td>
<td>26.7</td>
<td>32.6</td>
</tr>
<tr>
<td>20</td>
<td>2</td>
<td>29.7</td>
<td>34</td>
</tr>
<tr>
<td>21</td>
<td>2</td>
<td>19.0*</td>
<td>27.2</td>
</tr>
<tr>
<td>23</td>
<td>4</td>
<td>17.2*</td>
<td>29.3</td>
</tr>
<tr>
<td>27</td>
<td>2</td>
<td>21.5*</td>
<td>26.8</td>
</tr>
<tr>
<td>28</td>
<td>2</td>
<td>29.1</td>
<td>28.9</td>
</tr>
<tr>
<td>30</td>
<td>4</td>
<td>22.1*</td>
<td>23.3*</td>
</tr>
<tr>
<td>May 21</td>
<td>4</td>
<td>25.4*</td>
<td>26.0*</td>
</tr>
<tr>
<td>29</td>
<td>4</td>
<td>23.8*</td>
<td>26.7</td>
</tr>
</tbody>
</table>

\textsuperscript{*fails to meet form removal strength}

Furthermore, the Concrete Consultant noted that some of the compression test reports showed test results at two, three, and four days after casting. This data is included in Table 4.1. These tests were used to determine the earliest age at which the concrete was strong enough to allow form removal. The contract specified that this strength was to
be 75% of the specified strength or 26.2 MPa. Of the eleven early tests, eight fell below 26.2 MPa. There is no record that tests were repeated at a later date to allow safe form removal. In fact, even some of the seven-day test results fell below 26.2 MPa and it was very unlikely that forms were still in place seven days after casting. Studies have shown that shoring loads and construction live loads often exceed service loads and they occur at a time when the concrete has not attained its full design strength. Consequently, early form removal may have also contributed to the North-South cracks on the underside of the slabs.

A review was also made of the “Change in Crack Width” data that was collected during the monitoring period. Reference should be made to Graphs C6 and D6 in the appendices. A comparison of the two graphs showed that temperature fluctuations had very little influence on the movement of the long North-South crack in P4. As a matter of fact, crack widths were generally less than 0.05 mm smaller than their initial crack widths. Only location #17 showed movement slightly inconsistent with the other locations. However, this target was located below the entrance to P2 and experienced loads from the greatest number of moving vehicles. Hence, this long crack was considered slightly active but did not grow during the monitoring period.

The evidence presented supports the opinion that the North-South cracks are flexural or structural cracks. Large moments may have been introduced during the construction stage or from moving vehicles. Furthermore, inadequate concrete strength and the increase in concrete cover contributed to a decrease in structural capacity.
Despite the cause, it was reassuring to learn of the limited crack movement in this area during the monitoring period.

4.2.3 **DIAGONAL SLAB CRACKS (Slab Soffits of P4 and P3)**

In addition to East-West and North-South slab cracks, there are also a number of cracks that run in diagonal directions. Diagonal cracks are widespread and are also evident in the two slab areas where the Consulting Engineers performed their covermeter and coring studies\(^{15}\) (see Chapter 3). Hence, diagonal cracks in these areas were monitored to provide insight on their movement.

The first slab area that the Consulting Engineers investigated coincided with the location that was designated as a problem area. The North-West corner of P4 has a large opening that exposes the area to large temperature fluctuations. The slab-on-grade is also heavily damaged due to uplift from frost action. It was felt that the frost heave in this area was also responsible for pushing parts of the wall upward. Since the structure is reasonably monolithic, this movement of the wall has also caused distress and cracking in the suspended slab\(^{3}\). Consulting Engineer data revealed that slab thickness in this area ranged from 181 mm to 192 mm rather than the specified 200 mm. In addition, concrete cover to the bottom reinforcement ranged from 25 mm to 45 mm and averaged 32 mm rather than the specified 25 mm. Accordingly, these factors have resulted in a reduction in slab structural capacity and combined with frost action and possible early form removal, are responsible for this damage.
A comparison between Graphs C7 and D7 showed that there is some trend in the crack width movement related to ambient conditions. However, it seems that crack movement was inversely related to temperature fluctuations. Crack widths decreased when they should have increased and vice versa. Crack width changes deviated less than 0.05 mm from their initial values. Once again, these cracks were slightly active but did not appear to be growing. However, it should be noted that any growing movement in this area would be distributed throughout the high incidence of cracking and would be difficult to trace.

The second area investigated by the Consulting Engineers was in P3 (see Chapter 3). In this location, slab thickness was satisfactory. Yet, structural capacity may have been compromised since the concrete cover to the bottom reinforcement ranged from 18 mm to 45 mm rather than the specified 25 mm. The high incidence of diagonal cracking in the center of this slab area also suggested possible construction overloading or early formwork removal.

A comparison between Graphs C11 and D11 showed that most of the movement across these diagonal cracks could be explained by ambient temperature fluctuations. Once again, these cracks were active but could not be considered growing at this time.

It follows once more that any growing movement in this area would be distributed throughout the high incidence of cracking and would be difficult to trace.
4.3 WALL CRACKS

4.3.1 HORIZONTAL WALL CRACKS

These cracks appeared to be structural flexural cracks. It was speculated that the bending moments responsible for these cracks could be attributed to either excessive soil pressure against the foundation walls or the inappropriate timing of backfill during the construction stage. The basis of this speculation was that the cracks were located in the vicinity of the maximum moment generated by the soil pressure. In addition, horizontal cracks in P4 ended where exterior grade elevations were considerably lower.

The data collected by the Consulting Engineers was very valuable when comparing design details in the walls of the parking structure with the actual as-built condition. Significant differences were detected in the North-West portion of P4 or Test Area 3 in the field investigation. Figure 4.2 shows the differences in wall thickness, concrete cover, and vertical reinforcement spacing.

![Diagram showing wall thickness, concrete cover, and vertical reinforcement spacing](image)

*Figure 4.2 - Comparison of design and actual wall detailing (all dimensions in mm).*
Rough calculations of moment capacity per meter of wall length revealed that the design was to provide an ultimate moment resistance of approximately 40.8 kN-m at 28 days. However, according to the actual details, a moment resistance of only about 34.3 kN-m could have been developed. Therefore, the required capacity is about 19% higher than the available capacity. Since 28-day concrete strength data does not exist for the walls, it is difficult to establish whether early concrete strength was satisfactory or whether early form removal occurred. One can only speculate that the poor construction practice found in building the slabs was consistent throughout the project.

In comparing Graphs C2 and C3 with Graphs D2 and D3, it was difficult to attribute horizontal crack movement in P4 to temperature fluctuations. However, there were two sharp spikes in the data (Target #3) that were also evident in the predicted data. This target was located in the problematic North-West corner of P4. Accordingly, the high drops in ambient temperature may have led to frost heaving. In turn, this may have forced the upper and lower horizontal cracks to open more than they have in other instances.

Moving to P3, comparisons were made between Graphs C8 and C9 with Graphs D8 and D9. Movement of these upper and lower horizontal cracks seemed inversely related to ambient temperature conditions. Cracks were actually closing when they should have been opening and vice versa. The movement in Target #35 was inconsistent with the movement displayed at the other locations because this crack was a single diagonal crack that branched off into the upper and lower horizontal cracks. Hence, more movement seemed to be focused here.
Overall, upper and lower horizontal cracks in P4 and P3 were considered active as crack widths fluctuated within about 0.05 mm of their initial values. However, movement was not substantial. It should be realized that there will be a gradual increase in strain with time under the sustained load of the soil pressure. This effect is known as creep and may result in future horizontal crack width growth.

4.3.2 VERTICAL AND DIAGONAL WALL CRACKS

Most of the vertical wall cracks were joined with the East-West slab soffit cracks. Hence, these cracks were believed to be caused by the same factors. They were a result of the stresses that were induced into the walls due to the restraint to volume changes in the reinforced concrete. These volume changes were the result of a combination of drying shrinkage and thermal effects.

In the slabs, it was shown that water may have been added to the concrete mixes at the site since concrete strengths were low and slumps were high. If this practice occurred in the construction of the walls, an increase in shrinkage would result and would, ultimately, increase the likelihood of cracking.

Since the vertical wall cracks were similar to the East-West slab cracks, it was decided not to monitor them. However, some vertical cracks contained diagonal components. It had been postulated that these diagonal cracks were the result of shear stresses. These shear stresses may have been induced into the wall as frost heaving may have pushed parts of the walls upward. These cracks were considered structurally significant and, hence, it was decided that they would be monitored. It can be seen from
comparing Graph C1 with Graph D1 that the movements across these cracks were generally temperature related. Although these cracks were considered quite active, there was no evidence that suggested that they were growing. Most crack widths returned to their initial values when ambient conditions reverted back to those found when the targets were first installed. The only exception occurred at Target #32. Once again, this crack was in the problematic North-West corner of P4. This crack was close to 2 mm in width and was located at the foot of the stairs. Here, crack movement was unpredictable but it reverted to its original width a few times during the monitoring period.

4.4 SLAB-ON-GRADE

It seems that frost heaving was the main cause of deterioration in this slab. A poorly prepared subgrade and weeping tile system seemed to be the source of the problem. Water accumulated between the raft foundation and the foundation wall in the North-West corner of P4. In addition, there is a large opening in the North foundation wall that created very cold conditions in this region. Reference to the report made by the Geotechnical Engineers\(^9\) should be made for further details.

4.5 SECOND-FLOOR BEAMS

It was previously stated that designs were inconsistent for the second-floor beams. The two beams symmetrically situated around the main entrance had totally different reinforcement detailing as was shown in Figure 3.6. The problem with this situation is that the simpler design may have been adopted for both beams in the field. It was also
discovered that the simpler design had an inadequate amount of transverse or shear reinforcement.

Certain portions of these beams also supported rigidly connected concrete load-bearing walls. As a result, the span of the beams near the main entrance was shorter than the span of all other beams. Consequently, the beams near the main entrance were considered less flexible than the others and were more prone to shear cracking. Moreover, it was speculated that the center of the raft foundation was settling more than the external portions of the foundation. This would cause the interior portions of the beams to displace downward more than the exterior portions of the beams that rested on columns. As a result, it was likely that shear stresses were induced into these deep beams causing the diagonal cracking. Beam crack widths approaching 0.8 mm were attributed to the inadequate amount of shear reinforcement. As well, the differential movement may have also induced flexural stresses in the columns resulting in horizontal cracks on the outer face of the columns (refer to Figure 3.8).

Shear cracks on the beams and horizontal cracks on the columns were monitored to establish whether or not there was any movement at these locations. Graphs of “Change in Crack Width vs. Time” are included in Appendix E. The beam data in Graph E1 suggested that the beam just South of the main entrance showed substantially more movement than the beam on the North side. However, upon further examination it was revealed that a valid comparison could not be made. This is because the crack monitored on the South beam was one of the main shear cracks while the crack monitored on the North beam was a minor shear crack.
The individual sides of the South beam also showed different amounts of movement. This was attributed to the sun which tended to shine predominantly on the South-facing surface of the South beam. It was felt that the sun heated up this surface causing the concrete to expand and, in turn, caused the cracks to close. Movement on the North side of the South beam was considered more substantial as crack widths did not revert to their original value. As a result, there was evidence that this crack was growing slightly.

Since initially only minor cracks were monitored on the North beam, there was little surprise that movement was trivial. However, horizontal cracks on the column below the North beam have shown signs of growth (see Graph E2). In addition, new vertical cracks have been recently observed on the exterior face of this column. This implied that movement may have also occurred on other shear cracks located on the North beam.

As a result, additional targets were installed on both beams as it was felt that other cracks may have been more critical and would display more movement. The locations and data of the new and existing targets have all been provided in Appendix E. It is worthy to note that the limited data (Graphs E3 to E6) from the newer targets has confirmed that other cracks on the North beam were more critical. This was determined as the newer targets demonstrated more movement than the original targets. This new data suggested that crack growth had also occurred on shear cracks found on the North beam. This is consistent with the data that showed horizontal-crack growth on the column below the North beam.
CHAPTER 5 - REPAIR CONSIDERATIONS AND FIBRE REINFORCED PLASTICS

5.1 GENERAL

Most of the cracks in the parking structure were considered active but not growing. Most of the patterns of crack movement followed the variation in temperature. To the contrary, movement on the second-floor beams and underlying columns was difficult to explain. Crack growth has occurred and seems to be confirmed with the subsequent data from the new targets. Moreover, it was warned that horizontal wall cracks in the parking levels may show signs of future growth as they are exposed to sustained soil loads. Finally, some of the cracks in the slabs have also been found to move independent of temperature variations.

Although structural safety is not an immediate concern, crack widths far exceed serviceability requirements. A previous report\(^3\) on this structure pointed out that cracking should not exceed a point such that the long-term safety, durability, and serviceability of the structure are jeopardized. The European CEB-FIP Code recommends that for
moderate exterior exposure, the average crack thickness should not exceed 0.15 mm and no more than 5% of the cracks should exceed a width of about 0.25 mm. Reference to crack width descriptions in Chapter 3 will reveal that many crack widths exceed these limits.

It was shown through the extracted concrete cores that certain cracks in the parking levels penetrated the entire foundation wall. This is confirmed by the seepage of water through specific locations in the walls and slabs. Since oxygen, moisture, and chlorides are allowed to reach the reinforcing steel in abundance, accelerated steel corrosion is conceivable and the durability of this reinforced concrete is substantially threatened. In turn, this may compromise structural safety in the near future.

As a result, water proofing and membrane problems should be examined and immediately repaired where signs of seepage exist. As well, crack repair alternatives should be investigated that can satisfy the cyclical movements detected in the monitoring period. Previous attempts have utilized the epoxy injection method. However, it seems that cracks have reappeared since this method does not allow relief from active crack movement.

Parts of the beams also extend beyond the building envelope into an unprotected environment. As a result, part of the span is exposed to moisture, wind, and temperature fluctuations while the remaining portion is sheltered. The combinations of all of these factors seriously compromise the durability of these beams.

Eventually, the selection of an appropriate repair procedure for the parking levels and the exterior beams will be required. The selection of a repair material and technique
may be based on factors such as: durability, cost, appearance, time required for application, and the ease of application. Any repair procedure selected must address the cause of cracking, any future movement, strengthening requirements, and the moisture environment of the crack. Quality should be assured in a repair technique by conducting appropriate tests.

5.2 FIBRE REINFORCED PLASTICS

Various reinforced concrete rehabilitation techniques employ traditional materials such as steel, concrete, or other cementitious materials. At times, these repairs may lack durability and are often repeated. As well, these techniques are often very labour intensive. Furthermore, most traditional techniques are cumbersome requiring closing of the facility for a long duration of time. In turn, this is often very costly. As our nation's infrastructure continues to deteriorate, the need for more economical and more durable rehabilitation techniques has emerged. An alternative that has been growing in popularity is the use of Advanced Composite Materials (ACM). These materials are also commonly referred to as Fibre Reinforced Plastics (FRP).

The use of these fibre composites can provide a number of advantages over traditional rehabilitation techniques. For instance, these light-weight materials are easy to install and, in turn, are less labour- and equipment-intensive. In addition, these materials enhance durability as they prevent the passage of chlorides and other harmful chemicals. Finally, glass, carbon, and aramid composites possess high strength-to-weight ratios and can significantly enhance the strength of reinforced-concrete sections.
Today, FRP products include bars, cables, grids, plates, laminates, and impregnated fabrics. These products may provide internal and external reinforcement for new construction as well as rehabilitation. FRP composites are defined as a polymer matrix reinforced by fibres. The polymer matrix may be thermosetting or thermoplastic. Thermosetting polymer matrices include polyester, vinyl ester, epoxy, and phenolic while thermoplastic matrices include nylon. The reinforcing fibres provide the composite with its structural properties. The role of the matrix is to protect the fibres from environmental and mechanical damage and to distribute the load among them.

5.2.1 PHYSICAL AND MECHANICAL PROPERTIES

The main fibres used in civil engineering applications are glass, carbon, and aramid. Typical properties of commercially available fibres are shown in Table 5.1. The most common form of FRP used in structural applications is called a laminate. Laminates are created by stacking a number of thin layers of fibres and matrix and unifying them into a desired thickness. The orientation of the fibres can be controlled in each layer to yield a range of physical and mechanical properties. A unidirectional arrangement in fibres is anisotropic with the maximum strength and modulus in the direction of the fibre axis. Two-dimensional and three-dimensional arrangements also exist and the mechanical properties in any one direction are proportional to the amount of fibre by volume oriented in that direction.

Another popular FRP product used in rehabilitation is a woven roving or a fabric. This product is similar to a laminate in that fibres may be woven together at various
angles. However, the reason for its popularity is that the epoxy or other type of polymer matrix is introduced in the field. This allows the fabric to conform to the shape of the element to be repaired. Since laminates are usually produced in plants under higher quality control, a significant difference exists between laminates and fabrics. The production process of laminates allows the fibre content to be close to 60% by volume. In the hand lay-up system used in some projects, fabrics are often saturated with plenty of resin, resulting in a composite that may contain only 30% fibre by volume. As a result, tensile strength and stiffness of FRP products may vary.

Table 5.1 - Typical properties of commercial composite reinforcing fibres.

<table>
<thead>
<tr>
<th>Fibre Type</th>
<th>Tensile Modulus (MPa)</th>
<th>Tensile Strength (MPa)</th>
<th>Failure Strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Glass</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>E-Glass</td>
<td>72,400</td>
<td>3,450</td>
<td>4.8</td>
</tr>
<tr>
<td>S-Glass</td>
<td>86,900</td>
<td>4,300</td>
<td>5.0</td>
</tr>
<tr>
<td>Carbon</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T-300</td>
<td>231,000</td>
<td>3,650</td>
<td>1.4</td>
</tr>
<tr>
<td>HSB</td>
<td>344,500</td>
<td>2,340</td>
<td>0.58</td>
</tr>
<tr>
<td>Aramid</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kevlar 49</td>
<td>131,000</td>
<td>3,620</td>
<td>2.8</td>
</tr>
<tr>
<td>Twaron 1055</td>
<td>127,000</td>
<td>3,600</td>
<td>2.5</td>
</tr>
</tbody>
</table>

Figure 5.1 shows typical stress versus strain diagrams for GFRP and CFRP fabrics impregnated with epoxy resin. These plots are compared with steel. It should be noted that the behaviour of the composites is linear elastic to failure. However, failure occurs at fairly high strains. Hence, properly designed elements reinforced with these composites will still exhibit large deflections prior to failure.
Figure 5.1 - Typical stress versus strain behaviour of steel and composites.

The fatigue behaviour of composites is, generally, quite good. In tests where the loading was repeated for ten million cycles, it was concluded that carbon-epoxy composites have better fatigue strength than steel\textsuperscript{21}. Other research has shown that unidirectional glass reinforced composites do not fatigue when stressed below 50\% of their tensile strength\textsuperscript{22}. Fatigue tests performed on aramid-based composites is not yet readily available.

Furthermore, carbon and glass fibres have excellent resistance to creep. However, the same is not true for the polymer matrix. Hence, the orientation and volume of fibres influence the creep performance of FRP's. Creep tests were conducted in Germany on GFRP composites with various cross sections. These studies indicated that creep rupture does not occur if sustained stress is limited to 60\% of the short-term strength\textsuperscript{21}. Other research has included the application of long-term (1 year) sustained loads corresponding to 50\% of the ultimate strength to GFRP and CFRP tendons at room temperature.
Results from these tests showed very little creep\textsuperscript{21}. On the other hand, the phenomenon of creep-rupture failure has been observed in aramid-based fibre composites\textsuperscript{23}. However, new PPD-T aramid fibres (Kevlar and Twaron) have shown to be resistant to fatigue and creep rupture\textsuperscript{21}. More research is required to show the fatigue and creep response of these aramid fibres when embedded in a polymer matrix.

If these fibre-composites are to be used with concrete, it is important that the behaviour under thermal stresses for the two materials be similar so that differential deformations are minimized. An average coefficient for thermal expansion of concrete was reported earlier in this thesis as being $10 \times 10^{-6}$ per degree Celsius\textsuperscript{2}. The coefficient of thermal expansion for some GFRP products is approximately $9.9 \times 10^{-6}$ per degree Celsius. For CFRP, the coefficient of thermal expansion is close to zero\textsuperscript{21}. Hence, differential deformations should not be a problem for glass composites bonded to concrete and exposed to temperature fluctuations. However, problems may arise with carbon composites. Kaiser\textsuperscript{24} studied the temperature effects over 100 freeze-thaw cycles from +20$^\circ$C to -25$^\circ$C on concrete beams strengthened with CFRP plates and found no negative influence on the flexural capacity. Yet, a manufacturer of carbon fabric recommends the use of Tyfo\textsuperscript{TM} Fibre anchors oriented in a radial pattern around an epoxy grouted hole\textsuperscript{25}. This will provide the required anchorage to account for differential deformations between the impregnated fabric and the concrete when temperature fluctuations arise.
5.2.2 FACTORS AFFECTING MECHANICAL PROPERTIES

The absorption of water in composites could result in significant loss of strength and stiffness. However, there are special resins that are moisture resistant and may be used when a structure is expected to be exposed to moisture\(^25\).

Most fibres do not burn easily. However, resins contain large amounts of carbon and hydrogen that are flammable. Strength loss for FRP bars increases at high temperatures and approaches that of steel\(^21\). For FRP's used inside concrete, the concrete serves as a barrier from contact with flames. However, external composites used in rehabilitation may require special coatings to meet fire standards. For instance, a treatment is commercially available that can be used to achieve a UL Class 1 rating in a 3 hour flame and smoke spread test (ASTM E-84)\(^25\).

Composites exposed to direct sunlight can be damaged by ultraviolet rays. These rays cause chemical reactions in the polymer matrix, which can lead to a degradation of properties. There are commercially available paints that can provide UV protection as well as an aesthetically pleasing finish\(^25\).

While composites do not corrode, silica-based glass composites may deteriorate in an alkaline environment. During field application of fabrics, a layer of epoxy is initially placed on the concrete surface. This layer of epoxy serves as a barrier between the glass fibres and the concrete. However, the epoxy must be chosen so that micro cracks do not form as a result of stresses induced by temperature fluctuations. These micro cracks could allow seepage of alkaline materials to the glass fibres. Specialty polyester resins are currently being developed specifically for this purpose. It should be noted that products
such as Tyfo™ epoxy membrane also exist that can be installed to protect glass composites from potentially harmful alkalis\textsuperscript{25}.

### 5.3 Economic Considerations

Many advantages have been presented for the use of Fibre Reinforced Plastics as a rehabilitation technique. At first glance, one of the few drawbacks seems to be the higher material cost of Fibre Reinforced Plastics over traditional repair materials. However, this cost alone should not influence the decision to use these materials. A group of researchers\textsuperscript{26} has developed a methodology for evaluating the use of fibre composites in civil engineering applications. The model includes not only primary costs but also secondary or social costs.

Primary costs include material costs, installation costs and maintenance requirements. Although composites have a higher material cost, they also have a lower installation cost. Easier installation reduces requirements for labour and heavy equipment. The durability of composites also increases their economic life and reduces maintenance costs.

Secondary costs are often overlooked leading to improper decisions. For instance, the availability of the structure for its intended use should be considered. If industrial processes must be halted, loss of revenue may result. In the case of transportation projects, traffic congestion or detours may lead to social costs such as lost time, increased fuel consumption and an increase in fuel emissions. Many of these costs can be eliminated with the use of fibre-composites.
The model is capable not only of comparing economic factors but also a variety of criteria including material performance, construction duration, architectural aspects, codes and regulations, and material availability. As a result, the developed model provides decision makers with a tool to select an optimal structural material for infrastructure repair and construction. Many experts in the field of rehabilitation have tested the model with data from sample case studies and have demonstrated the model's applicability in selecting the repair material\textsuperscript{26}.

5.4 ACCEPTANCE

Examples of external strengthening of concrete structures with fibre composites are numerous. Applications include columns, walls, tanks, pier caps, light poles, beam-column joints, beams, elevator shafts, towers, slabs, prestressed I-beams, transmission poles, etc\textsuperscript{25}.

Despite all the available information surrounding the use of fibre composites, many designers are still reluctant to recommend their use. New technologies raise liability concerns deterring practicing engineers from using them. As a result, the development of standards and design specifications are vital for the expansion of composites in civil engineering applications, which in turn require extensive research results from analytical as well as realistic experimental work.

In order to investigate the use of fibre composites to rehabilitate the structure in this study, it was decided to conduct appropriate tests in the structural laboratories at the
University of Toronto. The feasibility of applying fibre composites on the walls of the lower parking levels and the second-floor beams was investigated in detail.
CHAPTER 6 - REPAIR OF WALLS: EXPERIMENTAL STUDY

6.1 GENERAL

The objective of this experimental study was to evaluate the use of fibre composite materials for the repair and strengthening of walls. In order to achieve this objective, the structural distress encountered in the field was simulated in two full-scale specimens in the laboratory. Once this was accomplished, specimens were rehabilitated with carbon and glass composites to test their effectiveness. A companion control specimen was tested to failure without rehabilitation to provide a basis for comparison.

6.2 BACKGROUND

The bonding of plates to the tension zone of concrete elements to improve flexural capacity is nothing new. The bonding of steel plates with epoxy resin has been used extensively in the rehabilitation of bridges and buildings. Since these steel plates are prone
to corrosion, attention has turned to fibre composites as a replacement. Initial developments in this area took place in Switzerland\textsuperscript{24,27}.

In 1987, Meier\textsuperscript{27} reported that thin CFRP sheets could be used as flexural strengthening reinforcement for concrete beams. He also showed that CFRP could replace steel with overall cost savings of 25\%. In 1989, Kaiser\textsuperscript{24} tested the use of CFRP composites on full scale reinforced concrete beams. He warned that inclined cracking could lead to premature failure as the composite sheet could peel-off. Kaiser also showed the validity of using the principle of strain compatibility in the analysis of cross-sections.

In 1991, Saadatmanesh and Ehsani successfully used GFRP plates to strengthen reinforced concrete beams\textsuperscript{28}. They concluded that the preparation of the concrete surface and the selection of adhesive was of primary importance. In addition, the strengthening technique was particularly effective for beams with relatively low steel reinforcement ratios. Furthermore, they also managed to reasonably approximate the behaviour of the strengthened beams using the strain compatibility approach.

\section*{6.3 DESIGN CONSIDERATIONS}

Most tests in the past have focused on the flexural strengthening of beams. As a result, it was decided to extend these findings to larger elements such as walls. In this study, it was decided to simulate the distressed foundation walls encountered in the parking levels of the structure reviewed in the condition survey. Reinforced concrete wall panels were built according to the structural design plans for the foundation walls. The reinforcement ratio for the flexural steel was approximately 0.15\%. Reinforcement
consisted of 4-10 M bottom bars and 3-10 M top bars in the direction of the span and 5-10 M top and bottom bars in the transverse direction. Small plates (40 mm x 25 mm) were welded to the ends of the bars to ensure anchorage. All dimensions and reinforcement detailing are provided in Figure 6.1.

*Figure 6.1 - Typical specimen details.*
6.4 SPECIMEN FABRICATION AND PREPARATION

6.4.1 FABRICATION

A total of three specimens were built for this study. One specimen was to act as a control specimen while the other two specimens were to be repaired with carbon and glass composites. Steel reinforcement for each specimen was cut to size and strain gauged. Each specimen contained fifteen electric resistance strain gauges to provide us with strain information at various locations on the steel. These locations have been mapped out in Figure 6.2. Once strain gauging was completed, reinforcement grids were assembled.

Figure 6.2 - Strain gauge locations.
Wooden formwork was prepared in order to cast all three specimens at the same time. Formwork was coated with shellac and maxithane to prevent the wood from absorbing water from the concrete mix and to allow easy removal of the specimens. Reinforcement grids were then placed in the wooden forms as shown in Figure 6.3. This figure also shows how the strain gauge wires were placed prior to concrete casting.

![Figure 6.3 - Wall reinforcement in wooden formwork prior to casting.](image)

The following concrete mix was ordered: $f'c = 35$ MPa at 28 days, Type 10 Portland cement, 20 mm maximum size coarse aggregate, 80 mm slump, and 5% air entrainment. After casting, the specimens were covered with moist burlap and polyethylene sheeting for 7 days. Specimens were allowed to age for at least 28 days prior to testing. Fifteen cylinders were also cast to provide us with compressive strength data.
6.4.2 PREPARATION

Due to rough timber formwork, the bottom surfaces of the wall specimens were uneven. Since two of the three specimens were to be repaired with FRP, it was decided to grind the bottom surfaces to yield a smooth, clean surface to ensure good bond between the concrete and the FRP.

All specimens were tested in a universal MTS testing machine under deformation control. It was decided to apply two line loads to the specimens in order to yield cracking similar to that encountered in the field. Figure 6.4 shows the loading and support conditions under the test frame. This figure also shows the locations of external instrumentation. Two curvature meters and three Linear Variable Differential Transducers (LVDT's) were mounted to obtain additional information during loading.

*Figure 6.4 - Loading and support conditions.*
Figure 6.5 - Plan view of external instrumentation on wall section under test frame.

Figure 6.5 shows a plan view of the external instrumentation affixed to the wall specimens. LVDT’s provided deflection data. Curvature meters employed LVDT’s to provide us with curvature data. An algebraic derivation has been included in Appendix G to transform vertical deflections on the curvature meters to curvatures. All strain gauges, LVDT’s, and curvature meters were connected to and monitored by a data acquisition system. A typical photograph of a specimen prior to testing has been included as Figure 6.6.
Figure 6.6 - Typical test setup.
6.5 MATERIAL PROPERTIES

Standard concrete cylinders and steel coupons were tested to determine material properties. Furthermore, tensile coupons were constructed from the composite fabrics impregnated with epoxy adhesive (TYFO™ S)\textsuperscript{25}. The details of these coupons have been included in Appendix H while a photograph is shown in Figure 6.7. Typical plots of Force versus Strain for the concrete, steel, and the fibre composites have been included in Appendix I. A summary of this data has been included as Table 6.1. Typical plots of Tensile Force (per unit width of fabric) versus Strain for the CFRP and GFRP used are provided in Figure 6.8. The thickness of FRP composite depends on the amount of epoxy used. Since the structural properties of the composite do not change appreciably by the amount of epoxy used, it was decided to represent its strength in force per unit width rather than stress.

<table>
<thead>
<tr>
<th>Concrete</th>
<th>( f'c ) (MPa)</th>
<th>( e'c )</th>
<th>Mod. of Elasticity E (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall 1</td>
<td>48.4</td>
<td>1.86E-3</td>
<td>31 300</td>
</tr>
<tr>
<td>Walls 2 and 3</td>
<td>53.9</td>
<td>1.96E-3</td>
<td>33 000</td>
</tr>
<tr>
<td>Steel (E=200 000 MPa)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Yield Stress (MPa)</td>
<td></td>
<td>Ultimate Stress (MPa)</td>
<td>Rupture Strain</td>
</tr>
<tr>
<td>458</td>
<td></td>
<td>692</td>
<td>0.168</td>
</tr>
<tr>
<td>CFRP</td>
<td>Q (Force/unit width) (N/mm/layer)</td>
<td>Rupture Strain</td>
<td></td>
</tr>
<tr>
<td></td>
<td>850 - 956</td>
<td>0.0142</td>
<td></td>
</tr>
<tr>
<td>GFRP</td>
<td>Q (Force/unit width) (N/mm/layer)</td>
<td>Rupture Strain</td>
<td></td>
</tr>
<tr>
<td></td>
<td>490 - 568</td>
<td>0.0197</td>
<td></td>
</tr>
</tbody>
</table>

The carbon fabric used (SCH41) had fibres oriented in only the longitudinal direction\textsuperscript{25}. However, the glass fabric (SEH51) had glass fibres in the longitudinal
direction and aramid fibres in the transverse direction. There were substantially more glass fibres than aramid fibres and, as a result, transverse properties were not found.

Figure 6.7 - Glass and carbon coupons constructed to yield material properties.

Figure 6.8 - Results from tests on tensile coupons.
6.6 TESTING OF SPECIMENS

The first specimen was referred to as the control specimen (Wall 1). The details of loading, shear and moment diagrams are provided in Figure 6.9. Four flexural cracks appeared during loading. The changes in crack width were observed as the total load increased in order to determine the point where the other two specimens would be repaired. Eventually, the specimen failed in flexure at a total load (2P) of approximately 193 kN as shown in Figure 6.10.

![Diagram showing loading, shear, and moment diagrams](image)

Figure 6.9 - Loading, Shear, and Moment diagrams.
Figure 6.10 - Flexural failure of Wall 1.

Figure 6.11 - Flexural cracks under Wall 2 at a total load (2P) of 135 kN.
The second specimen (Wall 2) was to be rehabilitated with the carbon fabric. This specimen was initially loaded to approximately 135 kN. At this point, two cracks averaging approximately 0.4 mm in width had formed. The average strain at the center of the bottom flexural steel was $3.3 \times 10^{-3}$ at this stage. These cracks appeared similar to the ones in the distressed structure and are shown in Figure 6.11. As a result, the load was maintained while the repair procedure was initiated. All external instrumentation was removed in order to apply the carbon fabric. R.J. Watson Inc. was called in to perform the repair.

It should be noted that two types of epoxy were used in the repair process. The first type was referred to as a Tack Coat Epoxy for the TYFO™ Composite FIBRWRAP™ System\textsuperscript{25}. It was applied directly to the concrete surface. This viscous epoxy was essential to ensure that the fabric would adhere to the horizontal surface below the specimen during the curing process. The carbon fabric was then saturated in the second type of epoxy prior to being applied beneath the specimen. The second type of epoxy was referred to as the TYFO™ S Epoxy for the TYFO™ Composite FIBRWRAP™ System\textsuperscript{25}. Each type of epoxy consisted of components A and B. The mix ratio by volume was 100 parts of A to 42 parts of B. Components were mixed for 5 minutes with a mixer running at a speed of 400-600 RPM until uniformly blended.

Three strips of fabric approximately 600 mm in width were applied to the specimen as shown in Figures 6.12 and 6.13. Epoxy thickness was not controlled but excess epoxy was squeezed out. The outer strips of fabric were folded up and bonded to the sides of the specimen. Previous research\textsuperscript{29} had shown that this provided an effective anchorage in
beams to eliminate premature plate separation and develop the flexural strength of the repaired element. Although approximately 90% of the epoxy strength is gained in the first 24 hours, the epoxy was allowed to cure for approximately three days.

Cross-Section

East-West Elevation

Figure 6.12 - Application of fabric to wall specimens.

Figure 6.13 - Various stages in the repair of Wall 2.
Figure 6.13 (con’d) - Various stages in the repair of Wall 2.
The specimen remained under load throughout this period as it was felt that soil loads could not be removed in the field prior to rehabilitation. Over the three days, the load actually fell to approximately 115 kN under displacement control of the testing machine. At this time, external instrumentation was reapplied as shown in Figure 6.14. Loading recommenced and continued until the specimen was damaged extensively. The specimen failed in shear with large inclined cracks and delamination of CFRP (see Figure 6.14). The maximum load (2P) carried by the specimen was 478 kN whereas the maximum predicted load using the General Method of Shear Design was 484 kN (see Appendix J). This prediction included the contribution from the FRP in resisting shear.

The third specimen (Wall 3) was tested the same way as Specimen 2 except that glass FRP was used for repair. The behaviour of Specimen 3 was very similar to that of Specimen 2 but the failure load was smaller. Shear failure occurred at a total load (2P) of 422 kN as shown in Figure 6.15. Strain in the longitudinal flexural steel at the bottom at the time of repair was approximately $3.3 \times 10^3$. In addition, the predicted shear capacity of the wall specimen, including the contribution from the FRP, was 430 kN (see Appendix J).
Figure 6.14 - Reloading and failure of Wall 2.
Figure 6.15 - Reloading and failure of Wall 3.
6.7 TEST RESULTS

The load-deflection curves for all three specimens have been included in Figure 6.16. The use of FRP resulted in a substantial increase in the ultimate capacity of the wall specimens. The ultimate capacity of Wall 2 was increased by 148% while the ultimate capacity of Wall 3 was increased by 119%. However, the full potential of FRP was not realized. The external FRP reinforcement was specifically aimed at enhancing the flexural capacity of the slabs. The load corresponding to the shear capacity of the wall section was much lower than that for the enhanced flexural capacity. The failure in both repaired slabs was, therefore, caused by shear. There was no indication of premature bond failure of FRP or its peeling off the concrete surface.

![Control Specimen vs. Repaired Specimens](image)

Figure 6.16 - Load-deflection behaviour of wall specimens.
A comparison of curves in Figure 6.16 indicates that the repaired specimens were restored to stiffness values nearly equal to those of the original undamaged specimens. In addition, the responses of the repaired specimens were reasonably ductile and resulted in large energy dissipation although not as ductile as the control specimen. The GFRP repaired specimen showed higher ductility than the CFRP repaired specimen.

6.8 ANALYTICAL RESULTS

In order to specify these materials in the field, it is important to develop analytical tools to aid in the design process. Analytical results are compared with the experimental results in Figures 6.17, 6.18, and 6.19 in the form of moment-curvature behaviour. The strain-compatibility method as outlined in a report by Sheikh and Bayrak\textsuperscript{30} was used for section analysis. The Hognestad parabola was used for the concrete stress-strain curve. Elastic-plastic and elastic stress-strain relationships for the steel and fibre composites were used respectively. Calculations were performed by hand and verified using program RESPONSE\textsuperscript{7,31}.

Hand calculations were performed with and without tension stiffening effects. The hand calculations neglecting tension stiffening effects matched up better with the experimental responses. All hand calculations have been included in Appendix K. It was observed that a tension stiffening factor of 0.5 in program RESPONSE provided satisfactory predictions of the experimental responses. This factor was used since it was estimated that only about half of the area below the neutral axis was effective in resisting
tensile stresses after cracking. Alternatively, one can specify zones of concrete around reinforcement bars that are fully effective in resisting tensile stresses after cracking.

Figure 6.17 - Experimental and predicted behaviour of Wall 1.

In the repaired specimens (Figures 6.18 and 6.19), the initial curvature at the time of repair must be considered in the analysis for accurate prediction of behaviour. This was achieved by allowing for initial strains at the top and bottom fibres of the wall specimens. The bottom fibre concrete strain at repair ($\varepsilon_o$) becomes the locked-in strain difference between the concrete and FRP$^{32}$. Typical strain profiles encountered in the analysis of Wall 2 have been included in Figure 6.20.

Program RESPONSE allows the user to enter initial strains. As a result, two analyses were performed for each repaired specimen. The first analysis was performed up
until the repair point. The second analysis was performed after the repair point considering the material properties of the fibre composites and the initial strains in the concrete. The two analyses were then superimposed to generate the moment-curvature responses in Figures 6.18 and 6.19.

![Graph showing experimental and predicted responses of CFRP-repaired wall.](Image)

**Figure 6.18** - Experimental and predicted behaviour of CFRP-repaired wall.

In general, the predicted and experimental responses of the repaired specimens matched up quite well up until the shear failures were realized. It can be concluded that available analytical models with appropriate material properties can be used to evaluate the performance of a repaired structural component reasonably accurately.
Figure 6.19 - Experimental and predicted behaviour of GFRP-repaired wall.

Figure 6.20 - Typical strain profiles in the analysis of Wall 2.
6.9 DISCUSSION

The laboratory tests have shown that the use of FRP provides a feasible rehabilitation technique for distressed walls. Furthermore, analytical models can be used to evaluate the performance of the repaired elements. Both carbon and glass composites provide a significant enhancement in flexural strength that shifted the failure to shear mode. The undesirable shear failure can be avoided by reducing the amount of fibre composite reinforcement. This can be achieved by using strips of FRP rather than covering the entire tension face with FRP. This will allow more efficient use of FRP and result in a more ductile behaviour. This idea has been tested in Japan33. CFRP laminates measuring 200 mm in width were bonded to the tension face of reinforced concrete slabs. One specimen had gaps of 100 mm between strips of FRP while another specimen had gaps of 200 mm. It was found that the bending resistance was proportional to the amount of CFRP and that the CFRP intervals were of little consequence.

It has already been stated that the ease of application and the durability of fibre composites make them very attractive for the repair of reinforced concrete elements. In addition to these advantages, the use of fibre composites to rehabilitate walls has several advantages over the use of conventional approaches. For instance, the stiffness of the wall will not increase as much as it would if shotcreting were used. This is important because it would not attract more load to the repaired element. In addition, if a fabric with vertical and horizontal fibres is chosen, then the in-plane shear capacity of the wall is also improved. This may be important in seismic regions that require seismic upgrading to meet current standards. Furthermore, the fibre composite fabric will add very little weight
to the structure. In turn, this will not require any strengthening of the foundation. As well, only a small increase in wall thickness will result in the loss of very little floor space. Moreover, composite fabrics can be easily cut in the field to allow for openings in the wall. As a result, all of these factors demonstrate that fibre composites can be very effective in rehabilitating distressed walls.
7.1 GENERAL

The objective of this study was to evaluate the use of fibre composite materials for the repair and shear strengthening of beams. In order to achieve this objective, the structural distress encountered in the field was simulated in an almost full-scale beam specimen in the laboratory. Once this was accomplished, the beam specimen was rehabilitated with carbon fabric to test the effectiveness of this material. A companion control beam specimen was also tested to provide data for direct comparison.

7.2 BACKGROUND

While there has been a considerable amount of research conducted on flexural strengthening of concrete beams with fibre composites, only a limited amount of work has been done involving shear strengthening, particularly in large beams. It is generally difficult to add external steel shear reinforcement to concrete beams, especially when they
are part of a floor-beam system. Since it is desirable to encase the beam's web with external reinforcement, this can be easily accomplished using a flexible material, such as a composite fabric.

A study conducted at the University of Delaware\textsuperscript{34} demonstrated that externally bonded composite fabric can be used successfully to provide a significant amount of shear reinforcement. Small scale T-beams were tested for this study. Externally applied fabrics made of aramid, E-glass, and graphite fibres were bonded to the webs of these beams to enhance their shear capacity. Increases in ultimate strength of 60 to 150\% were realized. In addition, it was shown that the orientation of the fabrics' fibres influenced the shear strength contribution.

A recent study was also completed for the Federal Highway Administration in the United States. Although details are not readily available, conclusions from the study have been published\textsuperscript{22}. Several beams were strengthened for shear by epoxy-bonding carbon fabrics to their sides. The unretrofitted beams experienced a brittle, shear failure. However, with the presence of fabric, premature failure was eliminated and the beams failed in a ductile flexural mode.

\subsection*{7.3 DESIGN CONSIDERATIONS}

Before this type of rehabilitation procedure can be utilized, results from large-scale tests should be available to evaluate the effectiveness of FRP. In this study, it was decided to simulate the distressed second-floor beams encountered in the structure that was reviewed in the condition survey. Almost full-scale reinforced concrete beams were built
according to the structural design plans for the beams. All dimensions and reinforcement detailing are provided in Figure 7.1.

The specimens were scaled down slightly to accommodate construction in the laboratory. As a result, the specimens built were 5/6 of the size of the beams in the field. In addition, the beams found in the field framed into walls. This was simulated by building a haunched region and increasing the amount of transverse reinforcement for half of the beam. As a result, shear cracks were expected to occur through Section B-B just as they have appeared in the field.

The amount of shear reinforcement in Section B-B was kept less than the minimum required by A23.3-94 19. The area of shear reinforcement was appropriately scaled down from the beams in the field. All transverse reinforcement consisted of deformed American Number 3 bars. As for flexural reinforcement, 5-25 M bars were used at the top while 6-30 M bars were used at the bottom. The amount of flexural reinforcement at the bottom was actually increased from 5-25M bars in the field to 6-30M bars in the lab to ensure a shear failure. Finally, the beams were constructed longer than necessary for the test span in order to increase the amount of anchorage for the flexural reinforcement.
Figure 7.1 - Typical specimen details.

All dimensions in mm
7.4 SPECIMEN FABRICATION AND PREPARATION

7.4.1 FABRICATION

Two specimens were built for this study. One specimen acted as a control specimen and was loaded all the way to failure. The other specimen was loaded until the distress resembled that in the field and then repaired with carbon fabric. This was done in order to directly evaluate the effects of strengthening with carbon fabric.

Steel reinforcement for each specimen was cut to size and bent if required. Each specimen contained twenty-one electric resistance strain gauges to provide strain information at various locations on the flexural and transverse steel. These locations have been mapped out in Figure 7.2. Once strain gauging was completed, reinforcement cages were assembled as shown in Figure 7.3.

Wooden formwork was prepared in order to cast one specimen at a time. Formwork was coated with shellac and form-oil to prevent the wood from absorbing water from the concrete mix and to allow easy removal of the specimens. Reinforcement cages were then placed in the wooden forms as shown in Figure 7.3.

The following concrete mix was ordered: $f'c = 30$ MPa at 28 days, Type 10 Portland cement, 20 mm maximum size coarse aggregate, 100 mm slump, and 5% air entrainment. Soon after casting, the specimens were covered with moist burlap and polyethylene sheeting for 3 days. At this point, the additional formwork shown in Figure 7.3 was built on top of the beam in order to cast the haunched portion. Once again, similar concrete was used and soon covered with moist burlap and polyethylene sheeting.
for 3 days. Specimens were allowed to age for at least 28 days prior to testing. Eight cylinders were also cast for each specimen to provide compressive strength data.

$T =$ Top Longitudinal Steel,  $S =$ Stirrups,  $B =$ Bottom Longitudinal Steel

**Section A–A**

**Figure 7.2 - Strain gauge locations.**
Figure 7.3 - Various stages in the fabrication of beam specimens.
7.4.2 PREPARATION

Smooth formwork was used to yield a smooth, clean surface to ensure good bond between the concrete and the FRP. Both specimens were tested using a hydraulic Jack connected to a rigid Baldwin testing machine frame. A load-cell connected to the data acquisition system was used to measure the applied load. It was decided to apply a single point load to the specimen in order to produce cracking similar to that encountered in the field. Figure 7.4 shows the locations of external instrumentation. Eight diagonal and four vertical LVDT's were mounted to obtain additional information during loading.

Vertical LVDT's provided deflection data. Diagonal LVDT's provided the data required to calculate shear strains. All strain gauges and LVDT's were connected to and monitored by a data acquisition system.

![Diagram of experimental setup](image)

Figure 7.4 - Diagonal and vertical LVDT's as external instrumentation on beams.
7.5 MATERIAL PROPERTIES

Standard concrete cylinders and steel coupons were tested to determine material properties. Carbon properties were taken from the tensile coupons tested in the wall study. Typical plots of Force versus Strain for the concrete and steel have been included in Appendix L. A summary of this data has been included as Table 7.1.

The carbon fabric used (SCH41) was the same as that used in the wall study. However, there was a small difference. The fibres used in the transverse direction to hold the longitudinal fibres in place were yellow in colour. As a result, the fabric merely appeared different.

Table 7.1 - Material Properties.

<table>
<thead>
<tr>
<th>Concrete</th>
<th>$f'c$ (MPa)</th>
<th>$e'c$</th>
<th>$E$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam 1</td>
<td>44.7</td>
<td>1.96E-3</td>
<td>30 000</td>
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<tr>
<td>Beam 2</td>
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<td>No.25</td>
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<tr>
<td>Amer.No.3</td>
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<td><strong>CFRP</strong></td>
<td><strong>Q</strong> (Force/unit width) (N/mm/layer)</td>
<td><strong>Rupture Strain</strong></td>
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<tr>
<td></td>
<td>850 - 956</td>
<td>0.0142</td>
<td></td>
</tr>
</tbody>
</table>
7.6 TESTING OF SPECIMENS

The first specimen was referred to as the control specimen (Beam 1). The loading, shear, and moment diagrams are provided in Figure 7.5. Figure 7.6 includes a photograph of the first specimen as it was ready to be loaded. Upon loading, several diagonal shear cracks appeared. The changes in crack width were recorded as the load increased (See Appendix M). Figure 7.6 also includes a photograph of the crack pattern at a load of 1000 kN. Loading continued until a sudden shear failure occurred at a load (P) of approximately 1700 kN (see Figure 7.7).

![Figure 7.5 - Loading, Shear, and Moment diagrams.](image-url)
Figure 7.6 - Photographs of Beam 1 prior to and during loading.
Figure 7.7 - Photographs of Beam 1 prior to and after failure.
Figure 7.8 - Photographs of Beam 2 during loading and at repair point (1180 kN).
The second specimen (Beam 2) was to be rehabilitated with the carbon fabric. Before loading commenced, the haunched portion of this beam was clamped together with the beam and bar assembly as shown in Figure 7.8. This was done to ensure that this portion of the beam would not fail in shear after the repair had taken place.

This specimen was initially loaded to approximately 1180 kN. At this point, approximately 5 diagonal cracks ranging from a minimum of 0.2 mm to a maximum of 0.8 mm had formed (See Appendix M). The average strain at the center of the bottom flexural steel was approximately 1.5 x 10^{-3} at this stage. These cracks appeared similar to the ones in the distressed structure and are shown in Figure 7.8. As a result, the load was maintained while the repair procedure was initiated. All external instrumentation was removed in order to apply the carbon fabric. The repair was performed by the author.

It should be noted that unlike for the slab repair work where the fabric was installed on the underside, only one type of epoxy was necessary in this repair process. Since the carbon fabric was to be wrapped around the beam, only the TYFO™ S Epoxy consisting of components A and B was required. The mix ratio by volume was 100 parts of component A to 42 parts of component B. Components were mixed for 5 minutes by a mixer operating at a speed of 400-600 RPM until uniformly blended. The epoxy was initially applied to the concrete surface with a roller. The carbon fabric was then saturated in the epoxy prior to being wrapped around the specimen.

Three strips of fabric approximately 610 mm in width were applied to the specimen as shown in Figures 7.9 and 7.10. A fabric overlap of 200 mm was used at the top of the
specimen upon wrapping. Epoxy thickness was not controlled but excess epoxy was squeezed out. The epoxy was then allowed to cure for approximately three days.

![Diagram of beam specimen with fabric application](image)

**Figure 7.9 - Application of fabric to beam specimens.**

The specimen remained under load throughout this period as it was felt that loads could not be removed in the field prior to rehabilitation. Over the three days, the load actually fell to approximately 1000 kN. At this time, external instrumentation was reapplied as shown in Figure 7.10.

Loading recommenced and continued to approximately 1911 kN. At this point, a compression failure was realized in the haunched portion of the beam directly below the load as shown in Figure 7.11. As a result, the specimen was unloaded. The damaged
concrete was removed. A steel enclosure was then constructed and attached to the specimen using threaded rods as shown in Figure 7.11. Finally, the steel enclosure was filled with a high-strength mortar that was allowed to cure for four days.

At this time, loading recommenced and was allowed to reach 2473 kN. The specimen was slightly unloaded a few times during this loading sequence as external instrumentation had to be reset to capture the substantial deflections that were demonstrated. The carbon fabric had succeeded in changing the response of the specimen to a very ductile one. It should also be noted that there were some vertical tears in the fabric as a small amount of transverse fibres were ruptured at high loads.

Loading was halted at 2473 kN as one of the support plates below the specimen appeared dangerously close to falling off its roller. The specimen was unloaded as this problem was to be eliminated. Loading soon recommenced and reached 2528 kN. There was enough evidence that suggested that the flexural steel was past the strain-hardening stage. As a result, either rupture of the bottom flexural steel or a compressive concrete failure at the top of the specimen was expected. At this load, the carbon fabric failed at a top corner close to the applied load as shown in Figure 7.12. Concrete below the fabric in this area appeared to be severely damaged. Since concrete in this area was no longer confined, a substantial amount of concrete suddenly spalled off and the beam could not take any more load. It should be noted that the rupture of the fabric occurred at the edge of the beam. Before the repair was carried out, the sharpness of the edges was somewhat reduced by slight grinding. A well-rounded edge perhaps would have eliminated this type of fabric failure.
Figure 7.10 - Photographs of Beam 2 during and after the repair.
Figure 7.11 - Failure below load point that was repaired before loading continued.
Figure 7.12 - Beam 2 failure at a load of 2528 kN.
7.7 TEST RESULTS

The load-deflection curves for both specimens have been included in Figure 7.13. The use of CFRP resulted in a substantial increase in the ultimate capacity of the beam specimen. The ultimate capacity of Beam 2 was increased by 49%. More importantly, premature shear failure was eliminated as the beam failed in a ductile flexural mode. In fact, the central deflection at failure was increased from 14 mm in the control specimen to 143 mm in the repaired specimen. In addition, the response of the repaired specimen resulted in much larger energy dissipation as can be seen from comparing the area below the curves in Figure 7.13.

![Figure 7.13 - Load-deflection behaviour of beam specimens.](image)
7.8 ANALYTICAL RESULTS

An attempt was made to predict the capacities of the beam specimens prior to testing. These calculations were performed by hand and are included in Appendix N. The Simplified and General Methods of Shear Design\textsuperscript{19} were used to predict the capacity of the control specimen. The Simplified Method predicted a shear failure through section B-B at a load $P$ of 1167 kN. The General Method predicted a shear failure through section B-B at a load $P$ of 1095 kN. Both methods were conservative as shear failure occurred at a load $P$ of 1700 kN. The shear span-depth ratio was approximately 2.0 for this test configuration. As a result, the development of compressive struts may have resulted after significant cracking had occurred\textsuperscript{7}. In turn, this may have contributed to the larger shear capacity displayed in the test.

The Simplified and General Methods of Shear Design\textsuperscript{19} were also used to predict the capacity of the repaired specimen. For these analyses, the carbon fabric was turned into a series of equivalent stirrups (see Appendix N). According to the Simplified Method, shear failure of the repaired beam was predicted to occur at a load $P$ of 4985 kN. According to the General Method, shear failure of the repaired beam was predicted at a load $P$ of 5245 kN.

However, the flexural capacity of the beam specimen also needed to be checked. Program RESPONSE 2000\textsuperscript{35} was used to develop the moment-curvature response shown in Figure 7.14. This was purely a flexural analysis of a section taken at the midspan of the specimen. The maximum moment predicted was 1988 kN-m. Looking back at the
moment diagram in Figure 7.5, the moment at the center of the beam can be found to be 0.91P. The load P for flexural failure can then be determined as follows:

\[
0.91P = 1988 \text{ kN}\cdot\text{m}
\]

\[
P = 2185 \text{ kN}
\]

As a result, a flexural failure was predicted to govern and occur in the repaired specimen at a load P of 2185 kN. A flexural response was achieved with failure occurring at a load P of 2528 kN.

After the test, a moment-curvature response was also generated from the strain gauge data. This response was plotted in Figure 7.14 and compared to the predicted response. The two curves correlated quite well.

Figure 7.14 - Comparison of predicted and measured responses of repaired beam.
After testing was completed, program RESPONSE 2000 was also used to model beam specimen behaviour. Shear strains were calculated from the diagonal LVDT data. The intersections of the diagonal LVDT's corresponded to sections 475 mm and 1675 mm from the point of load application. As a result, these were the sections chosen for analysis. Figure 7.15 shows the moment and shears at these sections as a function of the load. These ratios were entered into program RESPONSE 2000 along with all material properties. Plots of Shear Force versus Shear Strain were generated for both beam specimens.

![Diagram](image)

**Figure 7.15 - Moment and shear as a function of load at various sections.**
* Length of LVDT's were 1344 mm
* A typical formulation to find the shear strains was as follows:
  \[ \frac{(SD2/1344 - SD1/1344) + (N2/1344 - N1/1344)}{2} \]

Figure 7.16 - Comparison of predicted and measured responses for Beam 1.
Measured and predicted plots for the control specimen are included in Figure 7.16. The first graph shows responses for a section 475 mm from the load. The initial portions of the measured and predicted curves matched up quite well. However, the predicted shear force began to level off at less than 600 kN. The measured shear force actually increased to over 800 kN after it had appeared as though it would level off. Once again, this is evidence that a compressive strut may have formed that influenced behaviour after substantial cracking had occurred. Predicted and measured capacities were similar at a section 1675 mm away from the load. However, predicted strains were much less than those measured.

Measured and predicted plots for the repaired specimen are included in Figure 7.17. Once again, the first graph shows responses for a section 475 mm from the load. The entire portions of the measured and predicted curves compared quite well. At a section 1675 mm away from the load, the initial portions of the measured and predicted curves were reasonably similar. However, the measured portion ended when the flexural failure was realized at a more critical section. Each sectional analysis is completely independent. As a result, the user must determine the most critical section and the mode of failure.

In general, the predicted and experimental responses of Shear Force versus Shear Strain for the control and repaired specimens compared quite well. In addition, the Simplified and General Methods were shown to be helpful in the design process. As a result, analytical models with appropriate material properties can be used to evaluate the performance of a repaired structural component reasonably accurately.
Figure 7.17 - Comparison of predicted and measured responses for Beam 2.
7.9 DISCUSSION

The laboratory tests have shown that the use of FRP provides a feasible rehabilitation technique for beams that require shear strengthening. The carbon fabric prevented the widening of shear cracks. In turn, the concrete contribution to shear resistance was maintained. As a result, the carbon composites provided a significant enhancement in ultimate strength. In addition, premature shear failure was eliminated as the repaired beam failed in a ductile flexural mode. Finally, analytical models were used to effectively model the performance of the repaired elements.

At first glance, it may have seemed that the repaired beam was significantly over-reinforced for shear. This is because a flexural failure was to occur at a load P of 2185 kN while a shear failure may have occurred at a load P ranging from 4985 kN to 5245 kN. However, experimental results on internal shear reinforcement made of fibre composites indicated that stirrups failed in the corners due to premature failure at the bend. In addition, it was desired to cover the entire shear span with at least one layer of carbon fabric. This would eliminate the possibility of shear failure anywhere along the span. As a result, it was felt that an over-reinforced shear repair would be successful in eliminating any premature failures. Although the fabric did manage to fail at a corner, it may be argued that this coincided with a flexural failure and in fact, may have been triggered by large flexural deformations. A significant enhancement in the ultimate capacity and displacement had already occurred. In conclusion, this repair can be recommended for the second-floor beams of the apartment complex in the condition survey. However, more research should be conducted and unified to yield appropriate reinforcement ratios for
beams with other configurations. This data could also be used to refine present analytical techniques.

The ease of application and the durability of fibre composites makes them very attractive as a rehabilitation material. In addition to these advantages, the use of fibre composites to strengthen beams in shear has several advantages over the use of conventional approaches. For instance, the stiffness of the beam will not increase as much as it would if external steel stirrups were encased in a concrete overlay. This is critical to ensure that the repaired element would not attract more load. Furthermore, the fibre composite fabric will add very little weight to the beam which means that the enhanced strength will be available to resist the already existing loads rather than the additional weight of the repair materials. Moreover, composite fabrics can be easily cut in the field to allow for any openings and changes in the structural configuration. As a result, all of these factors demonstrate that fibre composites can be very effective in rehabilitating distressed beams.
CHAPTER 8 - CONCLUSIONS AND RECOMMENDATIONS

8.1 GENERAL

Cracks in concrete structures may become significant if they begin to compromise the strength, durability, or aesthetics of a structure. As a result, repair may be necessary. Prior to the undertaking of any repair procedure, the cause of cracking must be established. This is because the reason for cracking must often be eliminated to prevent cracking from re-occurring. As a result, a well-planned field and laboratory test program was carried out to obtain information on the extent of damage and the cause of deterioration in a prototype structure built only five years ago.

Since quality should be assured in any repair technique, a number of tests on wall and beam specimens were performed. The structural distress encountered in the field was simulated in full-scale specimens in the laboratory. Specimens were rehabilitated with glass and carbon composites to test their effectiveness as a repair technique. Companion control specimens were also tested to failure without rehabilitation to provide a basis for comparison.
CONCLUSIONS AND RECOMMENDATIONS

8.2 CONCLUSIONS

There were a number of factors that contributed to the high incidence of cracking throughout the structure. Cracking in the parking garage slabs and walls was believed to have been caused by inadequate detailing and poor construction practices. There were a number of slab and wall locations with inadequate thickness, cover, and reinforcement spacing. In addition, water may have been added to concrete mixes as strengths were low and slumps were high. As a result, a number of cracks may have been related to shrinkage. Thermal effects may have also contributed to cracking as there was a lack of contraction joints or reinforcement detailing to alleviate thermal movements. Furthermore, there was evidence of early form removal in slab regions. Hence, concrete may have been loaded prior to attaining appropriate strength. In external beam regions, the large inclined cracks were attributed to differential foundation settlement and inadequate shear reinforcement.

Most crack movement was attributed to ambient temperature variations. However, crack growth occurred on the second-floor beams and underlying columns. Future crack growth is also likely on horizontal wall cracks in the lower parking levels. Since most cracks exceed serviceability requirements, the selection of appropriate repair procedures for the parking levels and the exterior beams is required.

As reinforced concrete structures continue to deteriorate, the need for more economical and more durable rehabilitation techniques has emerged. The use of Fibre Reinforced Plastics (FRP) provides a number of advantages over traditional rehabilitation
CONCLUSIONS AND RECOMMENDATIONS

techniques. For instance, these light-weight materials are easy to install and, in turn, are less labour- and equipment-intensive. In most cases, repairs can proceed without taking the structure out of service. In addition, these materials enhance durability as they may prevent the passage of chlorides and other harmful chemicals. Finally, fibre composites possess high strength-to-weight ratios and can significantly enhance the strength of reinforced concrete sections.

The laboratory tests on three large wall specimens and two large beams demonstrated that the use of Fibre Reinforced Plastics provided a feasible rehabilitation technique for distressed walls and beams. However, care should be exercised as failure modes may be altered with the use of these materials. In the beam study, the repair by CFRP changed the failure mode from shear to flexure which was desirable. However, strengthening of the wall panels in flexure with CFRP and GFRP changed the mode of failure from flexure to shear which, in some cases, may not be acceptable. Analytical models were also shown to be quite effective in evaluating the performance of the repaired elements. The results from the laboratory tests conducted in this study can also be extended to other structures showing similar signs of distress.
8.3 RECOMMENDATIONS

There were many factors that contributed to the distress found in the structure outlined in this study. Many of these factors could have been eliminated with prudent planning and supervision. For instance, appropriate reinforcement detailing and contraction joints could have controlled cracking to more tolerable levels. In addition, strict site supervision could have eliminated the problems associated with the addition of water to the concrete, early form removal, and inappropriate placement of reinforcement.

For an engineer to confidently recommend the use of FRP in the field, a substantial database consisting of well-instrumented large-scale tests is required along with sound analytical techniques. In order to facilitate the expansion of Fibre Reinforced Plastics in civil engineering applications, other obstacles also need to be overcome. Since new fibre composite materials are frequently entering this field, standardization of test methods to achieve basic properties is imperative. This will help in streamlining Fibre Reinforced Plastic reinforcements for inclusion in design specifications and standards. In addition, studies are required to establish the most economical type and amount of fibre composites that will allow sufficient strength, stiffness, and elongation at failure. Additional long-term and accelerated aging studies under severe environmental conditions and extreme temperatures are also required. Finally, the reduction and synthesis of all available data should be used to refine analysis techniques and develop design guidelines for rehabilitation and new construction.
REFERENCES


APPENDIX A

LAYOUT OF LOWER PARKING LEVELS
Parking Layout (P4, P3)
## APPENDIX B - TARGET INFORMATION

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APPENDIX C

GRAPHS OF OBSERVED

CHANGE IN CRACK WIDTH VERSUS TIME
GRAPH C1

CHANGE IN CRACK WIDTH VS. TIME
(DIAGONAL WALL CRACKS - P4)

GRAPH C2

CHANGE IN CRACK WIDTH VS. TIME
(UPPER HORIZONTAL WALL CRACKS - P4)

GRAPH C3

CHANGE IN CRACK WIDTH VS. TIME
(LOWER HORIZONTAL WALL CRACKS - P4)

Oct 28, 1996
GRAPH C7

CHANGE IN CRACK WIDTH VS. TIME
(SECOND SPAN DIAGONAL SLAB CRACKS - P4)

Oct. 28, 1995

GRAPH C8

CHANGE IN CRACK WIDTH VS. TIME
(UPPER HORIZONTAL WALL CRACKS - P3)

Oct. 28, 1995

GRAPH C9

CHANGE IN CRACK WIDTH VS. TIME
(LOWER HORIZONTAL WALL CRACKS - P3)

Oct. 28, 1995
CHANGE IN CRACK WIDTH VS. TIME
-FIRST SPAN SLAB CRACKS / E-W / P3-

CHANGE IN CRACK WIDTH VS. TIME
-SECOND SPAN DIAGONAL SLAB CRACKS - P3-

Oct. 28, 1995
APPENDIX D

GRAPHS OF CALCULATED
CHANGef IN CRACK WIDTH VS. TIME
DUE TO TEMPERATURE VARIATIONS
CHANGE IN CRACK WIDTH VS. TIME DUE TO TEMPERATURE

(DIAGONAL WALL CRACKS - P4)

CHANGE IN CRACK WIDTH VS. TIME DUE TO TEMPERATURE

(UPPER HORIZONTAL WALL CRACKS - P4)

CHANGE IN CRACK WIDTH VS. TIME DUE TO TEMPERATURE

(LOWER HORIZONTAL WALL CRACKS - P4)
CHANGE IN CRACK WIDTH VS. TIME DUE TO TEMPERATURE
(First Span Slab Cracks / E-W / P4)

Oct. 28, 1995

CHANGE IN CRACK WIDTH VS. TIME DUE TO TEMPERATURE
(Second Span Slab Cracks / E-W / P4)

Oct. 28, 1995

CHANGE IN CRACK WIDTH VS. TIME DUE TO TEMPERATURE
(Second Span Slab Cracks / N-S / P4)

Oct. 28, 1995
CHANGE IN CRACK WIDTH VS. TIME DUE TO TEMPERATURE

(FIRST SPAN SLAB CRACKS / E-W / P3)

CHANGE IN CRACK WIDTH (mm)

0.0
0.2
0.4
0.6
0.8
1.0

TIME (DAYS)

Oct. 28, 1995

CHANGE IN CRACK WIDTH VS. TIME DUE TO TEMPERATURE

(SECOND SPAN DIAGONAL SLAB CRACKS - P3)

CHANGE IN CRACK WIDTH (mm)

0.0
0.2
0.4
0.6
0.8
1.0

TIME (DAYS)

Oct. 28, 1995
APPENDIX E

* SECOND FLOOR BEAM TARGETS
* GRAPHS OF OBSERVED CHANGE IN CRACK WIDTH VERSUS TIME

N - New Targets
E - Existing Targets

SOUTH BEAM - SOUTH FACE
NORTH BEAM - SOUTH FACE

SOUTH BEAM - NORTH FACE
NORTH BEAM - NORTH FACE
**GRAPH E1**

CHANGE IN CRACK WIDTH VS. TIME
(SECOND FLOOR BEAMS)

- Day 0 - October 28, 1995
- See Figure 3.8

**GRAPH E2**

CHANGE IN CRACK WIDTH VS. TIME
(COLUMNS)

- Day 150 - March 26, 1996
- See Figure 3.8
Day 0 - December 11, 1996

GRAPH E3

CHANGE IN CRACK WIDTH VS. TIME
(SOUTH BEAM - SOUTH FACE)

* Day 0 - December 11, 1996

GRAPH E4

CHANGE IN CRACK WIDTH VS. TIME
(SOUTH BEAM - NORTH FACE)

* Day 0 - December 11, 1996
**GRAPH E5**

* Day 0 - December 11, 1996

**GRAPH E6**

* Day 0 - December 11, 1996
APPENDIX F

RAW DATA COLLECTED DURING MONITORING OF THE STRUCTURE
APPENDIX G - ALGEBRAIC DERIVATIONS FOR CURVATURE METER

1. $2 \cdot c \varphi = T_{xz}$
   $c \varphi = T_{xz}/2$

2. $\delta = c \varphi - Tyz$
   $\delta = (T_{xz}/2) - Tyz$

3. $T_{xz} = (M/EI) \cdot 2c \cdot c$
   $= (M/EI) \cdot 2c^2$

4. $Tyz = (M/EI) \cdot c \cdot (c/2)$
   $= (M/EI) \cdot (c^2/2)$

5. $\delta = (M/EI) \cdot c^2 - (M/EI) \cdot (c^2/2)$

6. $(M/EI) = (2 \cdot \delta) / c^2$

7. $\phi = (M/EI)$

8. $\phi = (2 \cdot \delta) / c^2$
   where $c = 200$ mm
   and $\delta =$ LVDT deflection
TENSILE COUPONS

* 5 Carbon Coupons
* 5 Glass Coupons
* 4 Steel Plates per Coupon (40 Plates)
* Each Plate: 100mm x 75 mm x 4mm

FRP and Steel Plates

All Dimensions in mm

Longitudinal Fibres must be in this direction.
APPENDIX I

MATERIAL PROPERTIES FOR WALL SECTIONS
CONCRETE

* TYPICAL COMPRESSIVE STRESS-STRAIN CURVES

![Graph of typical compressive stress-strain curves for different walls.](image)
* RAW DATA FROM CONCRETE CYLINDER COMPRESSION TESTS

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<td>53.9</td>
<td></td>
<td>33049</td>
</tr>
<tr>
<td></td>
<td>*2,3 G 18232</td>
<td>992292</td>
<td>54.4</td>
<td></td>
<td>33198</td>
</tr>
<tr>
<td></td>
<td>AVG. 18232</td>
<td>982727</td>
<td>53.9</td>
<td>1.96E-03</td>
<td>33035</td>
</tr>
</tbody>
</table>

* Tested in a different frame - no strain information
STEEL

*TYPICAL TENSILE STRESS-STRAIN CURVE FOR
No. 10 REINFORCEMENT BARS

* RAW DATA FROM STEEL COUPON TENSILE TESTS

<table>
<thead>
<tr>
<th>SAMPLE</th>
<th>X-SECT. AREA (sq.mm)</th>
<th>YIELD FORCE (kN)</th>
<th>YIELD STRESS (MPa)</th>
<th>ULTIMATE FORCE (kN)</th>
<th>ULTIMATE STRESS (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100</td>
<td>46.3</td>
<td>463</td>
<td>69.7</td>
<td>697</td>
</tr>
<tr>
<td>2</td>
<td>100</td>
<td>46.3</td>
<td>463</td>
<td>70</td>
<td>700</td>
</tr>
<tr>
<td>3</td>
<td>100</td>
<td>44.8</td>
<td>448</td>
<td>67.8</td>
<td>678</td>
</tr>
<tr>
<td>AVG.</td>
<td>100</td>
<td>45.8</td>
<td>458</td>
<td>69.2</td>
<td>692</td>
</tr>
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</table>
TENSILE FORCE VS. STRAIN CURVES FOR CFRP COUPONS
* RAW DATA FROM CFRP TENSILE COUPONS

<table>
<thead>
<tr>
<th>COUPON</th>
<th>AVG. WIDTH (mm)</th>
<th>MAX. FORCE (N)</th>
<th>FORCE PER UNIT WIDTH Qu (N/mm/LAYER)</th>
<th>RUPTURE STRAIN (mm/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>35.1</td>
<td>29821</td>
<td>850</td>
<td>1.84E-02</td>
</tr>
<tr>
<td>C2</td>
<td>35.1</td>
<td>32768</td>
<td>934</td>
<td>1.08E-02</td>
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<tr>
<td>C3</td>
<td>35.1</td>
<td>33558</td>
<td>956</td>
<td>1.26E-02</td>
</tr>
<tr>
<td>C4</td>
<td>35.1</td>
<td>33172</td>
<td>945</td>
<td>1.32E-02</td>
</tr>
<tr>
<td>C5</td>
<td>35.1</td>
<td>30731</td>
<td>876</td>
<td>1.61E-02</td>
</tr>
</tbody>
</table>
TENSILE FORCE VS. STRAIN CURVES FOR GFRP COUPONS

COUPON 1

COUPON 2

COUPON 3
\* RAW DATA FROM GFRP TENSILE COUPONS

<table>
<thead>
<tr>
<th>COUPON</th>
<th>AVG. WIDTH (mm)</th>
<th>MAX. FORCE (N)</th>
<th>FORCE PER UNIT WIDTH Qu (N/mm/LAYER)</th>
<th>RUPTURE STRAIN (mm/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G1</td>
<td>35.1</td>
<td>18459</td>
<td>526</td>
<td>2.13E-02</td>
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<tr>
<td>G2</td>
<td>35.1</td>
<td>19951</td>
<td>568</td>
<td>2.32E-02</td>
</tr>
<tr>
<td>G3</td>
<td>35.1</td>
<td>17238</td>
<td>491</td>
<td>1.59E-02</td>
</tr>
<tr>
<td>G4</td>
<td>35.1</td>
<td>17201</td>
<td>490</td>
<td>1.99E-02</td>
</tr>
<tr>
<td>G5</td>
<td>35.1</td>
<td>18130</td>
<td>517</td>
<td>1.84E-02</td>
</tr>
</tbody>
</table>
Hand Calculations for Shear Capacity of Walls
*Refer to CSA STANDARD A23.3-94
and Prestressed Concrete Structures*

**Simplified Method** - (only used for specimen without carbon or glass repair - Wall #1)

from 11.3.4, \( V_r = V_c + V_s \), where \( V_s = 0 \) since there is no transverse reinforcement
from 11.3.5.1, \( V_c = 0.2 \times \lambda \times f'_c \times \sqrt{f'_c} \times b_w \times d \)

dropping the material resistance factors, \( V_c = 0.166 \times \sqrt{53.9} \times (1200 \text{ mm}) \times (220 \text{ mm}) \) N
\( V_r = V_c = 322 \text{ kN} \)

To achieve a shear of 322 kN, the total applied load \( 2P \) must equal \( 2 \times 322 = 644 \text{ kN} \)

**General Method**
*(Wall #1 - Control Specimen)*

from 11.4.3, \( V_{rg} = V_{cg} + V_{sg} \), where \( V_{sg} = 0 \) since there is no transverse reinforcement
from 11.4.3.1, \( V_{cg} = 1.3 \times \lambda \times f'_c \times \beta \times \sqrt{f'_c} \times b_w \times d_v \)

dropping the material resistance factors, \( V_{cg} = \beta \times \sqrt{f'_c} \times b_w \times d_v \)

where \( d_v = \text{distance between tensile and compressive forces due to flexure} \)

Tensile Force in steel = \( 4 \times 100 \text{mm}^2 \times 458 \text{ MPa} = 183 \text{ kN} \)

Total Tensile Force = Compressive Force = 183 kN

\[ a = \frac{183 \times 10^3 \text{ N}}{(0.85 \times 48.4 \text{ MPa} \times 1200 \text{ mm})} = 3.7 \text{ mm} \]
\[ d_v = 250 - (a/2) - \text{(distance from bottom to tensile force)} \]
\[ d_v = 250 - (3.7/2) - 30 \]
\[ d_v = 218 \text{ mm} \]

from 11.4.7, \( s_c = d_v = 218 \text{ mm} \)

from Table 7-4, Assume \( \varepsilon_x = 2.5 \times 10^{-3}, \beta = (1.48/12) = 0.123, \theta = 46^\circ \)

\[ V_{cg} = 0.123 \times \sqrt{48.4} \times (1200 \text{ mm}) \times (218 \text{ mm}) \]
\[ V_{rg} = V_{cg} = 224 \text{ kN} \]

Calculate longitudinal strain in steel at a section \( d_v \) away from loading plate:
Modifying eq. 11.22 (A23.3-94)

\[ \varepsilon_x = \frac{0.5 \times V_f \times \cot \theta + M_f / d_v}{E_x A_x} \]
\[
\varepsilon_x = \frac{(0.5 \times 224 \times \cot 46 + (224 \times 357 / 218)) \times 10^3}{400 \times 200000} = 5.9 \times 10^{-3}
\]

\(\varepsilon_x\) need not be taken greater than yield strain of steel. Thus, \(V = 224\) kN. However, since composites do not yield, \(\varepsilon_x\) may well exceed \(2.5 \times 10^3\) for walls 2 and 3.

To achieve a shear of 224 kN, the total applied load \(2P\) must equal \(2 \times 224 = 448\) kN

(Wall #2 - Carbon Repaired)

from 11.4.3, \(V_{rg} = V_{cg} + V_{sg}\), where \(V_{sg} = 0\) since there is no transverse reinforcement

from 11.4.3.1, \(V_{cg} = 1.3 \times \lambda \times \phi_c \times \beta \times \sqrt{f_c} \times b_w \times d_v\)

dropping the material resistance factors, \(V_{cg} = \beta \times \sqrt{f_c} \times b_w \times d_v\)

where \(d_v = \) distance between the resultants of the tensile and compressive forces due to flexure

Tensile Force in steel = \(4 \times 100 \text{mm}^2 \times 458\) MPa = 183 kN

Tensile Force in CFRP = \(800\) N/mm \(\times 1200\) mm = 960 kN

Total Tensile Force = Compressive Force = 1143 kN

\[a = \frac{143 \times 1000 \text{N}}{(0.85 \times 53.9 \text{MPa} \times 1200 \text{mm})} = 21\text{ mm}\]

\[d_v = 251 - (a/2) - \text{(distance from bottom to resultant of tensile forces)}\]

\[d_v = 251 - (21 / 2) - 5.4\]

\[d_v = 235\text{ mm}\]

from 11.4.7, \(s_e = \hat{d}_v = 235\) mm

from Table 7-4, Assume \(\varepsilon_e = 5.0 \times 10^{-3}\), \(\beta = (1.03/12) = 0.086\), \(\theta = 51^\circ\)

\[V_{cg} = 0.086 \times \sqrt{53.9} \times (1200 \text{mm}) \times (235 \text{mm})\]

\[V_{rg} = V_{cg} = 178\text{ kN}\]

Calculate longitudinal strain in steel at a section \(d_v\) away from loading plate:

Modifying eq. 11-22 (A23.3-94)

\[
\varepsilon_x = \frac{0.5 \times V_f \times \cot \theta + M_f / d_v}{E_s A_s + E_{FRP} \times \text{Width}_{FRP}}
\]
For Carbon, 

\[ E_{FRP}^* \text{ (modulus of elasticity per unit width)} = \frac{\text{Avg. Force per unit width}(Q)}{\text{Rupture Strain}} \]

\[ = \frac{910 \text{ N/mm}}{1.42 \times 10^{-2}} \]

\[ = 64000 \text{ N/mm} \]

Assume \( \varepsilon_x = 2.5 \times 10^{-3} \), \( \beta = (1.48/12) = 0.123 \), \( \theta = 47^\circ \)

\[ V_{cg} = 0.123 \times \sqrt{53.9} \times (1200 \text{ mm}) \times (235 \text{ mm}) \]

\[ V_{rg} = V_{cg} = 255 \text{ kN} \]

From eq. 11-22, \( \varepsilon_x = 3.1 \times 10^{-3} \)

Convergence will occur at a shear of approximately 242 kN.
To achieve a shear of 242 kN, the total applied load \( 2P \) must equal \( 2 \times 242 = 484 \text{ kN} \)
(Wall #3 - Glass Repaired)

from 11.4.3, \( V_{rg} = V_{cg} + V_{sg} \), where \( V_{sg} = 0 \) since there is no transverse reinforcement
from 11.4.3.1, \( V_{cg} = 1.3 \cdot \lambda \cdot f_{e} \cdot \beta \cdot \sqrt{f'c} \cdot b_{w} \cdot d_{v} \)

dropping the material resistance factors, \( V_{cg} = \beta \cdot \sqrt{f'c} \cdot b_{w} \cdot d_{v} \)

\[
\text{Tensile Force in steel} = 4 \cdot 100 \text{mm}^2 \cdot 458 \text{ MPa} = 183 \text{ kN} \\
\text{Tensile Force in GFRP} \approx 400 \text{ N/mm} \cdot 1200 \text{ mm} = 480 \text{ kN} \\
\text{Total Tensile Force} = \text{Compressive Force} = 663 \text{ kN} \\
a = 663 \text{ 000 N} / (0.85 \cdot 53.9 \text{ MPa} \cdot 1200 \text{ mm}) = 12 \text{ mm} \\
d_{v} = 251 - (a/2) - \text{(distance from bottom to resultant of tensile forces)} \\
d_{v} = 251 - (12/2) - 8.9 \\
d_{v} = 236 \text{ mm} \\
\]

from 11.4.7, \( s_{e} = d_{v} = 236 \text{ mm} \)

from Table 7-4 \(^7\), Assume \( \varepsilon_{x} = 5.0 \cdot 10^{-3} \), \( \beta = (1.03/12) = 0.086 \), \( \theta = 51^\circ \)
\[
V_{cg} = 0.086 \cdot \sqrt{53.9} \cdot (1200 \text{ mm}) \cdot (236 \text{ mm}) \\
V_{rg} = V_{cg} = 179 \text{ kN} \\
\]

For Glass,
\( E_{FRP}^* \) (modulus of elasticity per unit width) = Avg. Force per unit width(Q) / Rupture Strain 
\( = 518 \text{ N/mm} / 1.97E-2 = 26000 \text{ N/mm} \)

\[
\varepsilon_{x} = \frac{(0.5 \cdot 179 \cdot \cot 51 + (179 \cdot 339/236)) \cdot 10^3}{200000 \cdot 400 + 26000 \cdot 1200} = 2.96 \cdot 10^{-3} \\
\]

Assume \( \varepsilon_{x} = 3.0 \cdot 10^{-3} \), \( \beta = (1.35/12) = 0.1125 \), \( \theta = 48^\circ \)
\[
V_{cg} = 0.1125 \cdot \sqrt{53.9} \cdot (1200 \text{ mm}) \cdot (236 \text{ mm}) = 234 \text{ kN} \\
\]

From eq. 11-22, \( \varepsilon_{x} = 3.97 \cdot 10^{-3} \)

Convergence will occur at a shear of approximately 215 kN.
To achieve a shear of 215 kN, the total applied load \( 2P \) must equal \( 2 \cdot 215 = 430 \text{ kN} \)
**APPENDIX K**

*Hand Calculations for Moment-Curvature Response of Control Wall Specimen (Wall #1)*

Points of Interest:
I. Positive Cracking Moment  
II. Yield Moment  
III. Just Prior to Repair  
IV. Ultimate Capacity  

*Tension Stiffening will be neglected after cracking*

The Concrete Properties will be assumed to be the same for all 3 specimens:  
\[ f'_c = 53.9 \text{ MPa}, \quad \varepsilon'_c = 1.96\times 10^{-3}, \quad E_c = 33000 \text{ MPa}, \quad f_c = 2.4 \text{ MPa} \]

Steel Properties: \( f_y = 458 \text{ MPa}, \quad f_u = 692 \text{ MPa}, \quad \varepsilon_{du} = 168\times 10^{-3} \)

I. **Positive Cracking Moment**

\[ \varepsilon_{ct} = -5.7 \times 10^{-5} \]
\[ \varepsilon_{st} = -2.85 \times 10^{-5} \]
\[ 140.1 \]
\[ 109.9 \]
\[ \varepsilon_{sb} = 5.7 \times 10^{-5} \]
\[ \varepsilon_o = 7.27 \times 10^{-5} \]

The cracking strain was assigned to the bottom of the concrete. The top strain was then iterated upon until \( N = 0 \) kN.

When \( \varepsilon_{ct} = -5.7 \times 10^{-5}, \beta_1 = 0.668, \alpha_1 = 4.31 \times 10^{-2} \)

\[ C_c = \alpha_1 \times f'_c \times \beta_1 \times c \times b = 4.31 \times 10^{-2} \times 53.9 \text{ MPa} \times 0.668 \times 109.9 \text{ mm} \times 1200 \text{ mm} = 204.6 \text{ kN} \]

\[ T_s = E_s \times \varepsilon_{sb} \times A_s = 200000 \text{ MPa} \times 5.7 \times 10^{-5} \times 400 \text{ mm}^2 = 4.56 \text{ kN} \]

\[ C_s = E_s \times \varepsilon_{st} \times A_s = 200000 \text{ MPa} \times 2.85 \times 10^{-5} \times 300 \text{ mm}^2 = 1.71 \text{ kN} \]

\[ T_c = 0.5 \times f_{cr} \times (h-c) \times b = 0.5 \times (2.4 \text{ MPa}) \times (140.1 \text{ mm}) \times (1200 \text{ mm}) = 201.7 \text{ kN} \]
\[ N = T_s + T_c - C_s - C_c = 4.56 + 201.7 - 1.71 - 204.6 \equiv 0 \text{ kN}, \text{ O.K.} \]

Taking sum of the moments about the bottom (clockwise as positive):

\[
M = (4.56 \text{ kN} \times 30 \text{ mm}) + (201.7 \text{ kN} \times 46.7 \text{ mm}) - (1.71 \text{ kN} \times 195 \text{ mm}) - (204.6 \text{ kN} \times (250 - \beta_1 \times c / 2) \text{ mm})
\]

\[ M = 34.4 \text{ kN} \times \text{m} \]
\[ \phi = \epsilon_t / c = 5.7 \times 10^{-5} / 109.9 = 0.52 \times 10^{-6} \text{ rad/mm} \]

**II. Yield Moment**

The strain in the steel equals the yield strain. Iterate top strain until \( N \equiv 0 \text{ kN} \)

Try \( \epsilon_{st} = -0.274 \times 10^{-3}, \beta_1 = 0.675, \alpha_t = 0.198 \)

\( \epsilon_{at} = 0.367 \times 10^{-3}, \epsilon_{sb} = 2.29 \times 10^{-3}, \epsilon_{cb} = 2.64 \times 10^{-3}, c = 23.5 \text{ mm} \)

\[ C_c = \alpha_t \times f_c \times \beta_1 \times c \times b = 203.1 \text{ kN} \]

\[ T_{s1} = E_s \times \epsilon_{sb} \times A_s = 200 \times 1000 \text{ MPa} \times 2.29 \times 10^{-3} \times 400 \text{ mm}^2 = 183.2 \text{ kN} \]

\[ T_{s2} = E_s \times \epsilon_{st} \times A_s = 200 \times 1000 \text{ MPa} \times 0.367 \times 10^{-3} \times 300 \text{ mm}^2 = 22.0 \text{ kN} \]

\[ N = 183.2 + 22 - 203.1 = 2.1 \text{ kN} \equiv 0 \text{ kN}, \text{ O.K.} \]

Taking sum of the moments about the bottom (clockwise as positive):

\[ M = 39.4 \text{ kN} \times \text{m} \]
\[ \phi = \epsilon_t / c = 0.274 \times 10^{-3} / 23.5 = 11.6 \times 10^{-6} \text{ rad/mm} \]

**III. Just Prior To Repair**

\[ \epsilon_{ct} = -0.34 \times 10^{-3} \]
\[ c = 29.4 \text{ mm} \]
\[ \epsilon_{st} = 0.577 \times 10^{-3} \]
\[ \epsilon_{sb} = 3.33 \times 10^{-3} \]
\[ \epsilon_o = 3.83 \times 10^{-3} \]
Strain in the steel was determined from strain gauges. Top strain was iterated until \( N \equiv 0 \) kN

\[
\text{Try } \varepsilon_{\text{et}} = -0.34\text{E-3}, \beta_1 = 0.677, \alpha_1 = 0.241
\]

\[
Cc = \alpha_1 \cdot f'c \cdot \beta_1 \cdot c \cdot b = 215.3 \text{ kN}
\]

\[
Ts_1 = E_s \cdot \varepsilon_{sb} \cdot A_s = 200,000 \text{ MPa} \cdot 2.29\text{E-3} \cdot 400 \text{ mm}^2 = 183.2 \text{ kN (use yield strain)}
\]

\[
Ts_2 = E_s \cdot \varepsilon_{st} \cdot A_s = 200,000 \text{ MPa} \cdot 0.577\text{E-3} \cdot 300 \text{ mm}^2 = 34.6 \text{ kN}
\]

\[
N = 215.3 - 183.2 - 34.6 = -2.5 \text{ kN} \equiv 0 \text{ kN}, \text{ O.K.}
\]

Taking sum of the moments about the bottom (clockwise as positive):

\[
M = 40.1 \text{ kN*m}
\]

\[
\phi = \frac{\varepsilon_i}{c} = \frac{0.34\text{E-3}}{20.4} = 1.67\text{E-6} \text{ rad/mm}
\]

**IV. Ultimate Capacity**

Strain on top of concrete was equal to the crushing strain. Bottom strain was iterated upon until \( N \equiv 0 \) kN

\[
\varepsilon_{ei} = -3\text{E-3}, \beta_1 = 0.833, \alpha_1 = 0.9
\]

\[
\varepsilon_{et} = 18\text{E-3}, \quad \varepsilon_{ab} = 81.6\text{E-3}, \quad \varepsilon_{eb} = 93.1\text{E-3}, \quad c = 7.8 \text{ mm}
\]

\[
Cc = \alpha_1 \cdot f'c \cdot \beta_1 \cdot c \cdot b = 377.4 \text{ kN}
\]

\[
Ts_1 = E_s \cdot \varepsilon_{sb} \cdot A_s = (\geq 600 \text{ MPa}) \cdot 400 \text{ mm}^2 = 240 \text{ kN}
\]

\[
Ts_2 = E_s \cdot \varepsilon_{st} \cdot A_s = 458 \text{ MPa} \cdot 300 \text{ mm}^2 = 137.4 \text{ kN}
\]

\[
N = 0 \text{ kN}, \text{ O.K.}
\]

Taking sum of the moments about the bottom (clockwise as positive):

\[
M = 59.1 \text{ kN*m}
\]

\[
\phi = \frac{\varepsilon_i}{c} = 3\text{E-3} / 7.8 = 3.85\text{E-4} \text{ rad/mm}
\]
Hand Calculations for Moment-Curvature Response of Carbon Repaired Wall Specimen (Wall #2)

CFRP properties that will be used:

\( Q = \text{Force per unit width per layer of fabric (N/mm/layer)}, \)
\( \varepsilon_u = 14.2 \times 10^{-3}, \text{width of CFRP} = 1200 \text{ mm} \)

*Note: Q can be found by interpolation. One can take the approximate force at a specified strain from the coupon tests in Appendix I. This number is then divided by the coupon width (35.1 mm) to arrive at Q. Figure 6.8 also demonstrates typical values of Q. \( Q_{\text{ultimate}} \) ranged from 850-956 N/mm/layer (see Appendix I).

* Use first 3 points of Wall #1 Analysis

After Repair

Point #4

\[ \varepsilon_{et} = -0.494 \times 10^{-3} \]
\[ \varepsilon_{et} = -0.494 \times 10^{-3} \]
\[ \varepsilon_{st} = 0.71 \times 10^{-3} \]
\[ \varepsilon_{sb} = 4.31 \times 10^{-3} \]
\[ \varepsilon_{fc} = 1.17 \times 10^{-3} \]
\[ \varepsilon_{cb} = 5.0 \times 10^{-3} \]

A strain on the bottom of the concrete was arbitrarily selected. The top strain was then iterated upon until \( N = 0 \text{ kN} \).

\( \varepsilon_{et} = -0.494 \times 10^{-3}, \beta_1 = 0.682, \alpha_1 = 0.339 \)

\( C_c = \alpha_1 \times f'c \times \beta_1 \times \sigma_c \times b = 338 \text{ kN} \)
\( T_{s1} = E_s \times \varepsilon_{sb} \times A_s = 200 \text{ 000 MPa} \times 2.29 \times 10^{-3} \times 400 \text{ mm}^2 = 183.2 \text{ kN} \) (use yield strain)
\( T_{s2} = E_s \times \varepsilon_{st} \times A_s = 200 \text{ 000 MPa} \times 0.71 \times 10^{-3} \times 300 \text{ mm}^2 = 42.6 \text{ kN} \)
\( T_{\text{FRP}} = Q \times (\text{N/mm/Layer}) \times \text{Width of FRP (mm)} = 94.8 \times 1200 = 113.7 \text{ kN} \)

\( N = 1.5 \text{ kN} \equiv 0 \text{ kN}, \text{ O.K.} \)
Taking sum of the moments about the bottom (clockwise as positive):

\[ M = 68.1 \text{kN} \times \text{m} \]
\[ \phi = \epsilon_i / c = 0.494 \times 10^{-3} / 22.6 = 2.19 \times 10^{-6} \text{ rad/mm} \]

**Point #5**

A strain on the bottom of the concrete was arbitrarily selected. The top strain was then iterated upon until \( N \equiv 0 \text{kN} \).

When \( \varepsilon_{at} = -0.767 \times 10^{-3}, \beta_1 = 0.692, \alpha_1 = 0.492 \)
\[ \varepsilon_{at} = 0.93 \times 10^{-3}, \quad \varepsilon_{ab} = 6.04 \times 10^{-3}, \quad \varepsilon_{cb} = 7.0 \times 10^{-3}, \quad \varepsilon_{fc} = 3.17 \times 10^{-3}, \quad c = 24.8 \text{ mm} \]

\[ Cc = \alpha_1 \times f'c \times \beta_1 \times c \times b = 546.1 \text{kN} \]
\[ T_{s1} = E_s \times \varepsilon_{sb} \times A_s = 200 \times 10^3 \text{ MPa} \times 2.29 \times 10^{-3} \times 400 \text{ mm}^2 = 183.2 \text{kN} \text{ (use yield strain)} \]
\[ T_{s2} = E_s \times \varepsilon_{st} \times A_s = 200 \times 10^3 \text{ MPa} \times 0.93 \times 10^{-3} \times 300 \text{ mm}^2 = 55.8 \text{kN} \]
\[ T_{FRP} = Q \times (\text{N/mm/Layer}) \times \text{Width of FRP (mm)} = 256.8 \times 1200 = 308.1 \text{kN} \]

\[ N = 1.0 \text{kN} \equiv 0 \text{kN}, \text{ O.K.} \]

Taking sum of the moments about the bottom (clockwise as positive):

\[ M = 115.6 \text{kN} \times \text{m} \]
\[ \phi = \epsilon_i / c = 0.767 \times 10^{-3} / 24.8 = 3.09 \times 10^{-6} \text{ rad/mm} \]

**Point #6**

A strain on the bottom of the concrete was arbitrarily selected. The top strain was then iterated upon until \( N \equiv 0 \text{kN} \).

When \( \varepsilon_{at} = -1.53 \times 10^{-3}, \beta_1 = 0.725, \alpha_1 = 0.796 \)
\[ \varepsilon_{at} = 1.43 \times 10^{-3}, \quad \varepsilon_{ab} = 10.3 \times 10^{-3}, \quad \varepsilon_{cb} = 12 \times 10^{-3}, \quad \varepsilon_{fc} = 8.17 \times 10^{-3}, \quad c = 28.4 \text{ mm} \]

\[ Cc = \alpha_1 \times f'c \times \beta_1 \times c \times b = 1060 \text{kN} \]
\[ T_{s1} = E_s \times \varepsilon_{sb} \times A_s = 200 \times 10^3 \text{ MPa} \times 2.29 \times 10^{-3} \times 400 \text{ mm}^2 = 183.2 \text{kN} \text{ (use yield strain)} \]
\[ T_{s2} = E_s \times \varepsilon_{st} \times A_s = 200 \times 10^3 \text{ MPa} \times 1.43 \times 10^{-3} \times 300 \text{ mm}^2 = 86 \text{kN} \]
\[ T_{FRP} = Q \times (\text{N/mm/Layer}) \times \text{Width of FRP (mm)} = 661.7 \times 1200 = 794.1 \text{kN} \]

\[ N = 3.3 \text{kN} \equiv 0 \text{kN}, \text{ O.K.} \]

Taking sum of the moments about the bottom (clockwise as positive):

\[ M = 232 \text{kN} \times \text{m} \]
\[ \phi = \epsilon_i / c = 1.53 \times 10^{-3} / 28.4 = 53.9 \times 10^{-6} \text{ rad/mm} \]
**Point #7**

The rupture strain for the fibre composite was selected. The top strain was then iterated upon until \( N \geq 0 \) kN.

When \( \varepsilon_{st} = -2.5E-3, \ \beta_1 = 0.79, \ \alpha_1 = 0.928 \)

\( \varepsilon_{st} = 1.97E-3, \ \varepsilon_{ab} = 15.5E-3, \ \varepsilon_{ob} = 18.03E-3, \ \varepsilon_c = 14.2E-3, c = 30.6 \) mm

\[
Cc = \alpha_1 \cdot f_c \cdot c \cdot \beta_1 \cdot c \cdot b = 1451 \text{ kN}
\]

\( Ts_1 = Es \cdot \varepsilon_{sb} \cdot A = 500 \text{ MPa} \cdot 400 \text{ mm}^2 = 200 \text{ kN} \) (due to strain-hardening)

\( Ts_2 = Es \cdot \varepsilon_{st} \cdot A = 200000 \text{ MPa} \cdot 1.97E-3 \cdot 300 \text{ mm}^2 = 118.2 \text{ kN} \)

\( T_{FRP} = Q \cdot (\text{N/mm/Layer}) \cdot \text{Width of FRP (mm)} = 945 \cdot 1200 = 1134 \text{ kN} \)

\( N = 1.2 \text{ kN} \geq 0 \text{ kN} \), O.K.

Taking sum of the moments about the bottom (clockwise as positive):

\( M = 316.7 \text{ kN} \cdot \text{m} \)

\( \phi = \varepsilon_1 / c = 2.5E-3 / 30.6 = 81.7E-6 \text{ rad/mm} \)

* Note: Although it is more desirable to use values of Force per unit width per layer of CFRP when finding forces, this is not possible in computer applications. As a result, an average stress value and modulus of elasticity for the coupons was found to be 1050 MPa and 90 000 MPa, respectively, by assuming 0.9 mm thickness of one layer of CFRP. Stress values for FRP should be avoided since they are affected by epoxy thickness.
Hand Calculations for Moment-Curvature Response of Glass Repaired Wall Specimen (Wall #3)

GFRP properties that will be used:
- \( Q = \text{Force per unit width per layer of fabric (N/mm/Layer)} \),
- \( \varepsilon_u = 19.7 \times 10^{-3} \), width of GFRP = 1200 mm

*Note: \( Q \) can be found by interpolation. One can take the approximate force at a specified strain from the coupon tests in Appendix I. This number is then divided by the coupon width (35.1 mm) to arrive at \( Q \). Figure 6.8 also demonstrates typical values of \( Q \). \( Q_{\text{ultimate}} \) ranged from 490-568 N/mm/Layer (see Appendix I).

* Use first 3 points of Wall #1 Analysis

After Repair

Point #4

\[ \varepsilon_{ct} = -0.435 \times 10^{-3} \]
\[ c = 20.1 \text{ mm} \]
\[ \varepsilon_{st} = 0.755 \times 10^{-3} \]
\[ \varepsilon_{sb} = 4.33 \times 10^{-3} \]
\[ \varepsilon_{fc} = 1.17 \times 10^{-3} \]
\[ \varepsilon_{cb} = 5.0 \times 10^{-3} \]

A strain on the bottom of the concrete was arbitrarily selected. The top strain was then iterated upon until \( N \equiv 0 \) kN.
When \( \varepsilon_{ct} = -0.435 \times 10^{-3} \), \( \beta_1 = 0.68 \), \( \alpha_1 = 0.302 \)

\( C_c = \alpha_1 \cdot f'c \cdot \beta_1 \cdot c \cdot b = 267 \) kN
\( T_{s1} = E_s \cdot \varepsilon_{sb} \cdot b = 200000 \text{ MPa} \cdot 2.29 \times 10^{-3} \cdot 400 \text{ mm}^2 = 183.2 \) kN (use yield strain)
\[ T_{S2} = E_s \epsilon_{st} A_s = 200,000 \text{ MPa} \times 0.755E-3 \times 300 \text{ mm}^2 = 45.3 \text{ kN} \]
\[ T_{FRP} = Q (N/mm/Layer) \times \text{Width of FRP (mm)} = 33.1 \times 1200 = 39.7 \text{ kN} \]

\[ N = 267 - (183.2 + 45.3 + 39.7) = -1.2 \text{ kN} \leq 0 \text{ kN} \text{, O.K.} \]

Taking sum of the moments about the bottom (clockwise as positive):

\[ M = 50.6 \text{ kN} \times m \]
\[ \phi = \epsilon_t / c = 0.435E-3 / 20.1 = 21.6E-6 \text{ rad/mm} \]

**Point 5**

A strain on the bottom of the concrete was arbitrarily selected. The top strain was then iterated upon until \( N \leq 0 \text{ kN} \).

When \( \epsilon_{at} = -0.603E-3, \beta_1 = 0.686, \alpha_1 = 0.403 \)
\( \epsilon_{at} = 1.06E-3, \epsilon_{ab} = 0.066E-3, \epsilon_{eb} = 7.0E-3, \epsilon_{fc} = 3.17E-3, c = 19.9 \text{ mm} \)

\[ C_c = \alpha_1 \times f'c \times \beta_1 \times c \times b = 355.8 \text{ kN} \]
\[ T_{S1} = E_s \epsilon_{sb} A_s = 200,000 \text{ MPa} \times 2.29E-3 \times 400 \text{ mm}^2 = 183.2 \text{ kN} \text{ (use yield strain)} \]
\[ T_{S2} = E_s \epsilon_{st} A_s = 200,000 \text{ MPa} \times 1.06E-3 \times 300 \text{ mm}^2 = 63.8 \text{ kN} \]
\[ T_{FRP} = Q (N/mm/Layer) \times \text{Width of FRP (mm)} = 89.7 \times 1200 = 107.7 \text{ kN} \]

\[ N = 1.1 \text{ kN} \leq 0 \text{ kN} \text{, O.K.} \]

Taking sum of the moments about the bottom (clockwise as positive):

\[ M = 68.6 \text{ kN} \times m \]
\[ \phi = \epsilon_t / c = 0.435E-3 / 20.1 = 30.3E-6 \text{ rad/mm} \]

**Point 6**

A strain on the bottom of the concrete was arbitrarily selected. The top strain was then iterated upon until \( N \leq 0 \text{ kN} \).

When \( \epsilon_{at} = -1.04E-3, \beta_1 = 0.703, \alpha_1 = 0.624 \)
\( \epsilon_{at} = 1.81E-3, \epsilon_{ab} = 10.4E-3, \epsilon_{eb} = 12E-3, \epsilon_{fc} = 8.17E-3, c = 20.1 \text{ mm} \)

\[ C_c = \alpha_1 \times f'c \times \beta_1 \times c \times b = 570.3 \text{ kN} \]
\[ T_{S1} = E_s \epsilon_{sb} A_s = 200,000 \text{ MPa} \times 2.29E-3 \times 400 \text{ mm}^2 = 183.2 \text{ kN} \text{ (use yield strain)} \]
\[ T_{S2} = E_s \epsilon_{st} A_s = 200,000 \text{ MPa} \times 1.81E-3 \times 300 \text{ mm}^2 = 108.9 \text{ kN} \]
\[ T_{FRP} = Q (N/mm/Layer) \times \text{Width of FRP (mm)} = 231.3 \times 1200 = 277.5 \text{ kN} \]

\[ N = 0.7 \text{ kN} \leq 0 \text{ kN} \text{, O.K.} \]
Taking sum of the moments about the bottom (clockwise as positive):

\[ M = 112 \text{kN} \cdot \text{m} \]
\[ \phi = \epsilon_t / c = 1.045 \times 10^{-3} / 20.1 = 52 \times 10^{-6} \text{ rad/mm} \]

**Point #7**

The rupture strain in the fibre composite was selected. The top strain was then iterated upon until \( N = 0 \text{kN} \).

When \( \epsilon_{ct} = -2.2 \times 10^{-3}, \beta_1 = 0.766, \alpha_1 = 0.917 \)
\( \epsilon_{ct} = 3.43 \times 10^{-3}, \epsilon_{ab} = 20.3 \times 10^{-3}, \epsilon_{cb} = 23.53 \times 10^{-3}, \epsilon_{c} = 19.7 \times 10^{-3}, c = 21.5 \text{ mm} \)

\[
C_c = \alpha_1 \cdot f \cdot c \cdot \beta_1 \cdot c \cdot b = 976.8 \text{kN}
\]
\[
T_{S_1} = E_s \cdot \epsilon_{sb} \cdot A_s = 500 \text{ MPa} \cdot 400 \text{ mm}^2 = 200 \text{kN} \text{ (due to strain-hardening)}
\]
\[
T_{S_2} = E_s \cdot \epsilon_{st} \cdot A_s = 458 \text{ MPa} \cdot 300 \text{ mm}^2 = 137.4 \text{kN}
\]
\[
T_{FRP} = Q \cdot (\text{N/mm/Layer}) \cdot \text{Width of FRP (mm)} = 534 \cdot 1200 = 640.8 \text{kN}
\]

\( N = 1.4 \text{kN} \equiv 0 \text{kN} \), O.K.

Taking sum of the moments about the bottom (clockwise as positive):

\[ M = 203.7 \text{kN} \cdot \text{m} \]
\[ \phi = \epsilon_t / c = 2.2 \times 10^{-3} / 21.5 = 102 \times 10^{-6} \text{ rad/mm} \]

* Note: Although it is more desirable to use values of Force per unit width per layer of GFRP when finding forces, this is not possible in computer applications. As a result, an average stress value and modulus of elasticity for the coupons was found to be 600 MPa and 31 800 MPa, respectively, by assuming 0.9 mm thickness of one layer of CFRP. Stress values for FRP should be avoided since they are affected by epoxy thickness.
APPENDIX L

MATERIAL PROPERTIES FOR BEAM SECTIONS
CONCRETE

*TYPICAL COMPRESSIVE STRESS-STRAIN CURVES*
### RAW DATA FROM CONCRETE CYLINDER COMPRESSION TESTS

<table>
<thead>
<tr>
<th>CYLINDER</th>
<th>X-SECT. AREA (sq.mm)</th>
<th>MAX. FORCE (N)</th>
<th>f&lt;sub&gt;c&lt;/sub&gt;</th>
<th>e&lt;sub&gt;c&lt;/sub&gt;</th>
<th>ELASTIC MODULUS (MPa)</th>
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<tbody>
<tr>
<td><strong>06/24/97 Beam 1</strong></td>
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<td>45.2</td>
<td>1.80E-03</td>
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<td>45.2</td>
<td>30271</td>
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<td>43.1</td>
<td>29528</td>
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<td>1.96E-03</td>
<td>30072</td>
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<td><strong>Haunch</strong></td>
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<td></td>
<td></td>
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<tr>
<td>1HA</td>
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<td>41.2</td>
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* Tested in a different frame - no strain information
STEEL

AMERICAN NO. 3

* TYPICAL TENSILE STRESS-STRAIN CURVE

* RAW DATA FROM STEEL COUPON TENSILE TESTS

<table>
<thead>
<tr>
<th>SAMPLE</th>
<th>X-SECT. AREA (sq.mm)</th>
<th>YIELD FORCE (kN)</th>
<th>YIELD STRESS (MPa)</th>
<th>ULTIMATE FORCE (kN)</th>
<th>ULTIMATE STRESS (MPa)</th>
<th>RUPTURE STRAIN</th>
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</thead>
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<tr>
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<td>508</td>
<td>55.7</td>
<td>781.2</td>
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<tr>
<td>2</td>
<td>71.3</td>
<td>36.1</td>
<td>506</td>
<td>55.2</td>
<td>774.2</td>
<td>0.129</td>
</tr>
<tr>
<td>AVG.</td>
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<td>36.2</td>
<td>507</td>
<td>55.5</td>
<td>778</td>
<td>0.121</td>
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</table>
NO. 30

* TYPICAL TENSILE STRESS-STRAIN CURVE

* RAW DATA FROM STEEL COUPON TENSILE TESTS

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<thead>
<tr>
<th>SAMPLE</th>
<th>X-SECT. AREA (sq.mm)</th>
<th>YIELD FORCE (kN)</th>
<th>YIELD STRESS (MPa)</th>
<th>ULTIMATE FORCE (kN)</th>
<th>ULTIMATE STRESS (MPa)</th>
<th>RUPTURE STRAIN</th>
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<td>491</td>
<td>455</td>
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NO. 25

* TYPICAL TENSILE STRESS-STRAIN CURVE

* RAW DATA FROM STEEL COUPON TENSILE TESTS

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<th>SAMPLE</th>
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<th>YIELD FORCE (kN)</th>
<th>YIELD STRESS (MPa)</th>
<th>ULTIMATE FORCE (kN)</th>
<th>ULTIMATE STRESS (MPa)</th>
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<td>490</td>
<td>344</td>
<td>688</td>
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APPENDIX M

LOAD VERSUS TOTAL CRACK WIDTH FOR BEAM SECTIONS

*Captions indicate number of cracks crossing LVDT's
*LVDT'S were installed at 45° while major cracks appeared at ~40°.
* After repair point on Beam #2 (1180 kN), number of cracks could no longer be seen.
APPENDIX N

Hand Calculations for Shear Capacity of Control Beam (Beam #1)
*Refer to CSA STANDARD A23.3-94

Simplified Method
Right Side of Beam (Section B-B in figure 7.1)

from 11.3.4, \( V_r = V_c + V_s \)

from 11.3.5.2 (less transverse reinforcement than minimum),

\[
V_c = \left( \frac{260}{1000 + d} \right) \phi_s \frac{f'c}{\lambda} b_w d
\]

dropping the material resistance factors,\( V_c = \left( \frac{216}{1000 + d} \right) \sqrt{44.7} \times 550 \text{ mm} \times 935 \text{ mm} \)

\( V_c = 384 \text{ kN} \)

from 11.3.7, \( V_s = \frac{\phi_s A_v f_y d}{s} \)

dropping the material resistance factors, \( V_s = \frac{142.6 \times 507 \times 935}{400} = 169 \text{ kN} \)

from 11.3.4, \( V_r = V_c + V_s = 384 + 169 = 553 \text{ kN} \)

From the shear diagram, \( V = 0.474P \)

\( 553 = 0.474P \)

\( P = 1167 \text{ kN} \)

Simplified Method
Left Side of Beam (Section A-A in figure 7.1)

from 11.3.4, \( V_r = V_c + V_s \)

from 11.3.5.1 (more transverse reinforcement than minimum),


\[ V_c = 0.2 \lambda \phi_c \sqrt{f'c} b_w d \]

dropping the material resistance factors, \( V_c = 0.166 \sqrt{44.7} \times 550 \text{ mm} \times 935 \text{ mm} \)

\[ V_c = 571 \text{ kN} \]

dropping the material resistance factors, \( \nu_s = \frac{\phi_s A_v f_y d}{s} \)

from 11.3.7, \( \nu_s = \frac{285 \times 507 \times 935}{400} = 338 \text{ kN} \)

from 11.3.4, \( V_c = \nu_c + \nu_s = 571 + 338 = 909 \text{ kN} \)

From the shear diagram, \( V = 0.526P \)

\[ 909 = 0.526P \]

\[ P = 1728 \text{ kN} \]

**General Method**

**Right Side of Beam (Section B-B in figure 7.1)**

from 11.4.3, \( \nu_{rg} = \nu_{cg} + \nu_{sg} \)

from 11.4.3.1, \( \nu_{cg} = 1.3 \lambda \phi_c \beta \sqrt{f'c} b_w d_v \)

dropping the material resistance factors, \( \nu_{cg} = \beta \sqrt{f'c} b_w d_v \)

where \( d_v \) = distance between the resultants of the tensile and compressive forces due to flexure

Tensile Force in steel = \( 4200 \text{ mm}^2 \times 492 \text{ MPa} = 2066 \text{ kN} \)

Tensile Force = Compressive Force = 2066 kN

\[ a = 2 \times 066 000 \text{ N} / (0.85 \times 44.7 \text{ MPa} \times 550 \text{ mm}) = 99 \text{ mm} \]

\[ d_v = 1000 - (a/2) - \text{(bottom cover to reinforcement)} \]

\[ d_v = 1000 - (99 / 2) - 65 \]

\[ d_v = 886 \text{ mm} \]

from 11.4.7, \( s_x \approx 872 \text{ mm} \)

Since transverse reinforcement is less than minimum, can use Figure 11-2 (A23.3-94)

Assume \( \varepsilon_x = 1.5 \times 10^{-3}, \beta = 0.10, \theta = 57^\circ \)

\[ \nu_{cg} = 0.10 \sqrt{44.7} (550 \text{ mm}) (886 \text{ mm}) = 326 \text{ kN} \]

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from 11.4.3.2, \[ V_{sg} = \frac{A \nu \cdot f_y \cdot d \nu \cdot \cot \theta}{s} \] \[ = \frac{142.6 \cdot 507 \cdot 886 \cdot \cot 57}{400} = 104 \, \text{kN} \]

\[ V_{rg} = V_{cg} + V_{sg} = 326 + 104 = 430 \, \text{kN} \]

Calculate longitudinal strain in steel at a section \( d_v \) away from loading plate:
Modifying eq. 11-22 (A23.3-94)

\[ \varepsilon_x = \frac{0.5 \cdot V_f \cdot \cot \theta + M_f / d_v}{E_s A_s} \]

\[ \varepsilon_x = \frac{(0.5 \cdot 430 \cdot \cot 57 + (430 \cdot 1064 / 886)) \cdot 10^3}{200000 \cdot 4200} = 0.78 \cdot 10^{-3} \]

Assume \( \varepsilon_x = 1.0 \cdot 10^{-3} \), \( \beta = 0.125 \), \( \theta = 55^\circ \)

\[ V_{cg} = 0.125 \cdot \sqrt{44.7} \cdot (550 \, \text{mm}) \cdot (886 \, \text{mm}) = 407 \, \text{kN} \]

\[ V_{sg} = \frac{142.6 \cdot 507 \cdot 886 \cdot \cot 55}{400} = 112 \, \text{kN} \]

\[ V_{rg} = V_{cg} + V_{sg} = 407 + 112 = 519 \, \text{kN} \]

From eq. 11-22, \( \varepsilon_x = 0.96 \cdot 10^{-3} \)
As a result, \( V \approx 519 \, \text{kN} \)

From the shear diagram, \( V = 0.474P \)

\[ 519 = 0.474P \]

\[ P = 1095 \, \text{kN} \]

**General Method**

**Left Side of Beam (Section A-A in figure 7.1)**

from 11.4.3, \( V_{rg} = V_{cg} + V_{sg} \)

from 11.4.3.1, \( V_{cg} = 1.3 \cdot \lambda \cdot f_c \cdot \beta \cdot \sqrt{f'c} \cdot b_w \cdot d_v \)

dropping the material resistance factors, \( V_{cg} = \beta \cdot \sqrt{f'c} \cdot b_w \cdot d_v \)
where \( dv \) = distance between the resultants of the tensile and compressive forces due to flexure

Tensile Force in steel = \( 4200 \text{mm}^2 \times 492 \text{ MPa} = 2066 \text{ kN} \)
Tensile Force = Compressive Force = 2066 kN

\[ a = 2066 \times 1000 \text{ N} / (0.85 \times 44.7 \text{ MPa} \times 300 \text{ mm}) = 181 \text{ mm} \]
\[ d_v = 1300 - (a/2) - (\text{bottom cover to reinforcement}) \]
\[ d_v = 1300 - (181 / 2) - 65 \]
\[ d_v = 1144 \text{ mm}, \text{ let } d_v=1000 \text{ mm due to irregular cross-section} \]

Since transverse reinforcement is more than minimum, must use Figure 11-1 (A23.3-94)

\[ \frac{V_f}{bw \times dv} = 909 \text{ kN} / (550 \times 1000) = 1.65 \text{ MPa} \]

\( V_f=909 \text{ kN} \) was predicted from the simplified method

Assume \( \varepsilon_x = 1.5 \times 10^{-3}, \beta = 0.16, \theta = 41^\circ \)

\[ V_{cg} = 0.16 \times \sqrt{44.7} \times (550 \text{ mm}) \times (1000 \text{ mm}) = 588 \text{ kN} \]

from 11.4.3.2, \[ V_{sg} = \frac{A_v \times f_y \times dv \times \cot \theta}{s} = \frac{285 \times 507 \times 1000 \times \cot 41}{400} = 416 \text{ kN} \]

\[ V_{rg} = V_{cg} + V_{sg} = 588 + 416 = 1004 \text{ kN} \]

Calculate longitudinal strain in steel at a section \( d_v \) away from loading plate:
Modifying eq. 11-22 (A23.3-94)

\[ \varepsilon_x = \frac{0.5 \times V_f \times \cot \theta + M_f / d_v}{E_s \times A_s} \]

\[ \varepsilon_x = \frac{(0.5 \times 1004 \times \cot 41 + (1004 \times 750 / 1000)) \times 10^3}{200000 \times 4200} = 1.58 \times 10^{-3} \]

As a result, \( V \approx 1004 \text{ kN} \)
From the shear diagram, \( V = 0.526P \)
\[ 1004 = 0.526P \]
\[ P = 1909 \text{ kN} \]
Hand Calculations for Shear Capacity of Repaired Beam (Beam #2)
*Refer to CSA STANDARD A23.3-94 and Prestressed Concrete Structures*

*Simplified Method*

**Right Side of Beam (Section B-B in figure 7.1)**

from 11.3.4, \[ V_r = V_c + V_s \]

from 11.3.5.1 (more transverse reinforcement than minimum),

\[ V_c = 0.2 \times \lambda \times \phi_c \times \sqrt{f'_c} \times b_w \times d \]

dropping the material resistance factors, \[ V_c = 0.166 \times \sqrt{45.7} \times 550 \text{ mm} \times 935 \text{ mm} = 577 \text{ kN} \]

from 11.3.7, \[ V_s = \frac{\phi_s \times A_v \times f_y \times d}{s} \]

dropping the material resistance factors, \[ V_s = \frac{142.6 \times 507 \times 935}{400} = 169 \text{ kN} \]

*Total Area of FRP = 1830 mm wide * 0.9 mm thick * 2 sides = 3294 mm²*

*FRP covers an area equivalent to 5 stirrups spaced at approximately 400 mm. Therefore, each stirrup = 3294/5 = 659 mm²*

Assume FRP capacity can be approximated in the same manner as steel stirrups:

from 11.3.7, dropping the material resistance factors,

\[ V_{FRP} = \frac{\phi_{FRP} \times A_v \times f_y \times d}{s} = \frac{659 \times 1050 \times 935}{400} = 1617 \text{ kN} \]

from 11.3.4, \[ V_r = V_c + V_s + V_{FRP} = 577 + 169 + 1617 = 2363 \text{ kN} \]

From the shear diagram, \[ V = 0.474P \]

\[ 2363 = 0.474P \]

\[ P = 4985 \text{ kN} \]
**General Method**

**Right Side of Beam (Section B-B in figure 7.1)**

from 11.4.3, \( V_{rg} = V_{cg} + V_{sg} \),

from 11.4.3.1, \( V_{cg} = 1.3 \cdot \lambda \cdot \phi_c \cdot \beta \cdot \sqrt{f'c} \cdot b_w \cdot d_v \)

dropping the material resistance factors, \( V_{cg} = \beta \cdot \sqrt{f'c} \cdot b_w \cdot d_v \)

where \( d_v \) was found previously to be \( = 886 \text{ mm} \)

Since transverse reinforcement is more than minimum, must use Figure 11-1 (A23.3-94) or Table 7-3.

\[
\frac{V_f}{b_w \cdot d_v} = 2363 \text{ kN} / (550 \cdot 886) = 4.85 \text{ MPa}
\]

\( (V_f = 2363 \text{ kN} \) was predicted from the simplified method \)

\[
\frac{V_f}{f'c} = \frac{4.85}{45.7} = 0.106
\]

Using \( \varepsilon_x = 2.5 \cdot 10^{-3} \), then \( \beta = 0.0967, \theta = 38^\circ \)

\[
V_{cg} = 0.0967 \cdot \sqrt{45.7} \cdot (550 \text{ mm}) \cdot (886 \text{ mm}) = 319 \text{ kN}
\]

from 11.4.3.2, \( V_{sg} = \frac{A_v \cdot f_y \cdot d_v \cdot \cot \theta}{s} = \frac{142.6 \cdot 507 \cdot 886 \cdot \cot 38}{400} = 205 \text{ kN} \)

Assume FRP capacity can be approximated in the same manner as steel stirrups:

\[
V_{FRP} = \frac{A_v \cdot f_y \cdot d_v \cdot \cot \theta}{s} = \frac{659 \cdot 1050 \cdot 886 \cdot \cot 38}{400} = 1962 \text{ kN}
\]

\( V_{rg} = V_{cg} + V_{sg} + V_{FRP} = 319 + 205 + 1962 = 2486 \text{ kN} \)

\( \varepsilon_x \) can be found to be \( 5.4 \cdot 10^{-3} \). However, \( \varepsilon_x \) need not be taken greater than yield strain of steel. As a result, \( V = 2486 \text{ kN} \).

From the shear diagram, \( V = 0.474P \)

\[
2486 = 0.474P \\
P = 5245 \text{ kN}
\]