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CONCRETE CULVERTS REINFORCED WITH
GLASS FIBRE REINFORCED PLASTICS (GFRP)

by

ATEF AWAD HANNA

A Thesis submitted to the Faculty of Graduate Studies and Research through the Department of Civil and Environmental Engineering in Partial Fulfillment of the Requirements for the Degree of Master of Applied Science at the University of Windsor

Windsor, Ontario, Canada
1995
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ABSTRACT

The use of Glass Fibre Reinforced Plastics (GFRP) in reinforced concrete culverts has been investigated as an alternative rust free material to replace the conventional steel bars. Structures reinforced with GFRP show early development of cracks followed by proportional increase in loading and deflection as a result of the reduced bending rigidity compared to members reinforced with steel rebars. The reduction in bending rigidity is desirable in buried structures due to the fact that their behaviour is governed by the soil-structure interaction in which the bending moment and shear forces are reduced in the culvert walls with the reduction of its bending rigidity.

All design formulas are based on the use of steel bars in the reinforced concrete structures. The current study examines the validity of the design formulas to be used when replacing the steel bars by GFRP. An experimental study was conducted on eight simply supported circular reinforced concrete arches and four simply supported reinforced concrete beams. All models were loaded in two locations with two concentrated line loads. Different strength and failure modes were examined, such as the radial tension capacity, the flexural capacity, the shear load carrying capacity, and the influence of axial forces on the shear load carrying capacity. Also, the bending rigidity for members reinforced with GFRP is compared to the bending rigidity of members reinforced with steel.
TO MY FAMILY
ACKNOWLEDGEMENTS

The author wishes to express his deep appreciation to his advisor Dr. George Abdel-Sayed, who has originally suggested and determined the favourable soil-structure interaction in buried concrete structures when replacing the steel reinforcing bars with the steel reinforcing bars. His guidance, effort, and patient encouragement during the development of this research are greatly appreciated.

The author also wishes to thank Mr. Dieter Liebsch and the members of the technical support centre for their effort in preparing the formwork of the arch models.

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NOMENCLATURE

a  The depth of compression block at ultimate strength

$A_t$  Tension reinforcement area

$A_s'$  Reinforcement area on the compressive side of the section

b  The width of the cross section

d  Distance from the extreme compression face to the centroid of the tension reinforcement

d'  Distance from the extreme compression face to the centroid of the reinforcement near the compression face of the section

$E_c$  The modulus of elasticity for concrete

$E_g$  The modulus of elasticity for GFRP

$E_s$  The modulus of elasticity for steel reinforcing bars

$f_c'$  Concrete compressive strength

$F_c$  A curvature and crack depth factor for circular pipes

$F_d$  The depth factor

$F_N$  The thrust factor

$F_{pr}$  Factor for process and local materials characteristics that affect radial tension strength of the pipe

$F_{pv}$  Factor used to reflect the variation that local material and manufacturing process can have on the tensile strength of concrete
\( f_y \) The yield strength of the reinforcing steel bars

\( I_e \) The effective moment of inertia of the cross section, \( \text{mm}^4 \)

\( I_{cr} \) The moment of inertia for a cracked section, \( \text{mm}^4 \)

\( I_T \) The transformed moment of inertia for an uncracked section, \( \text{mm}^4 \)

\( M_{cr} \) The cracking moment

\( M_u \) Ultimate moment acting on the cross section

\( M_{m} \) Modified bending moment

\( n \) Ratio of modulus of elasticity of reinforcement to modulus of elasticity of concrete (modular ratio)

\( N_u \) Ultimate axial thrust acting on the cross section

\( r_m \) Radius to centroid of pipe wall

\( r_s \) Radius to inside reinforcement

\( S \) Inside span of the pipe

\( V_c \) The ultimate shear strength of the cross section

\( V_u \) Ultimate shear force acting on the cross section

\( w_p \) Weight of the pipe

\( \alpha \) Empirical transition constant

\( \phi \) Capacity reduction factor for shear

\( \rho \) The reinforcement ratio
CHAPTER I

INTRODUCTION

1.1 General

The durability of reinforced concrete culverts is usually affected by the corrosion of its steel reinforcing bars. This problem of corrosion is usually addressed by imposing strict conditions on the width of cracks in the design of concrete culverts. Also, techniques such as, the use of epoxy coated rebars (Clifton et al., 1975; Treece and Jirsa, 1989), and cathodic protections (Schell and Manning, 1985) have been developed and used. However, the reliability of these protections remains questionable and few reports show failures of these measures due to inadequate protection or hostile underground environment. Therefore, the use of fibre reinforced plastics has been suggested as a logical solution to the problem of rusting provided it adequately satisfies the structural and economical consideration for the design.

Glass-Fibre Reinforced Plastics (GFRP) have recently been introduced as reinforcement for concrete structures. However, the interest in using GFRP goes back to the time of steel shortage during World War II as an alternative reinforcement for concrete structures (Schmeckpeper, E.R.). Taking advantage of this interest, in 1941 John G. Jackson applied
for a patent in the use of fibreglass bars to reinforce concrete structural elements. By the
time the patent was granted in 1947 steel became plentiful, and consequently the use of
GFRP as a reinforcement for concrete was not widely adapted by the construction
industry.

Glass fibre reinforced plastic has advantages over conventional steel reinforcement in
specific applications such as concrete structures buried in corrosive environments,
structures where electric or electromagnetic insulation is required, or when high strength
to weight ratio is desired. In general, the GFRP has a low effective modulus of elasticity
in the order of 1/4 of that of the standard steel rods. The bending moment carrying
capacity of a section reinforced with GFRP could be higher than the same section
reinforced with equal area of steel but with lower bending rigidity. This characteristic
is undesirable in usual reinforced concrete structures because of the higher values of
deflection, but it is advantageous for buried structures such as precast concrete pipes.
This can be explained by the fact that the behaviour of the buried culverts is governed
by the soil-structure interaction in which the bending moment and shear forces are
reduced in the culvert wall with the reduction of its bending rigidity. As an example to
illustrate the structural advantage of GFRP over steel reinforcement, finite element
analysis has been conducted by (Abdel-Sayed and Hanna, 1995) using Culvert ANalysis
and DEsign (CANDE) program developed by (Ketona et al, 1976). An 8 ft
(2.44m)diameter circular culvert has been analized with a wall thickness of 6 inch
(150mm) under a soil cover of 8 ft (2.44m) and reinforced; first using No.4 @ 4x4 inch
conventional steel mesh and second using No.4 @ 4x4 inch mesh of GFRP. The bending rigidity of each culvert wall has been evaluated approximately based on the finding of Schmeckpeper, E.R. The results of the analysis are presented in figures 1-1 to 1-4 showing the normal soil pressure on the culvert wall, the bending moment distribution, the shear force distribution, and the thrust distribution on the conduit wall, respectively. These results show the favourable effect of the reduced rigidity on the soil pressure distribution and the corresponding reduction of 40% in the maximum bending moment and shear forces induced in the culvert wall. Furthermore, Figure 1-4 shows that the magnitude of thrust force has practically not changed, i.e., the ratio of M/N and V/N are also reduced leading to better performance of the reinforced concrete wall. In addition, the design using GFRP eliminates the need for crack control which limits the width of crack not to exceed 0.01 inch (0.25 mm) in concrete culverts reinforced with steel in order to protect the reinforcing bars from rusting.

1.2 Objectives

The discussion above highlights the advantages of using GFRP to replace the steel reinforcement in concrete culverts. However, the design formulas applied for steel reinforced concrete may not be applicable for the design of GFRP reinforced concrete culverts. The fact that the GFRP has lower modulus of elasticity leads to different formation of cracks in the concrete. It therefore affects the deformations and load carrying capacity of the conduit wall. The objective of this investigation is to examine the present formulas used for steel reinforced culverts and to propose modifications to allow such
formulas to be applicable for GFRP reinforced culverts.

The contents of this thesis are as follows:

Chapter II  a review of the theoretical formulas used in concrete culverts reinforced with steel and their evaluation for the use in concrete culverts reinforced with GFRP.

Chapter III presents the experimental program conducted in this study with the description of the arch and beam models tested in the laboratory, as well as the instrumentation and materials used.

Chapter IV presents a discussion of results obtained from the experimental and theoretical investigations.

Chapter V contains the summary and conclusion of the study as well as recommendation for future research.
FIG.1-1 SOIL PRESSURE DISTRIBUTION ON REINFORCED CONCRETE CULVERT (N/sq.mm/mm)
FIG 1-2 BENDING MOMENT DISTRIBUTION ON REINFORCED CONCRETE CULVERT (N/mm/mm)
FIG. 3-1 SHEAR FORCE DISTRIBUTION ON REINFORCED CONCRETE CULVERT (kN/m)
CHAPTER II

LITERATURE REVIEW

2.1 General

While extensive studies have been conducted on the behaviour of concrete culverts reinforced with steel bars, the author is unaware of any studies to examine the behaviour of concrete culverts reinforced with GFRP. Herein, it may be recognized that the behaviour of GFRP reinforced curved panels differ from identical panels with steel reinforcing bars. Due to the lower modulus of elasticity of GFRP, the crack size and formation are different from those of steel reinforced panels. The literature survey deals mainly with steel reinforced slabs and culverts and is presented as follows

i) Effective bending rigidity, \( I_E \)

ii) Ultimate bending moment capacity

iii) Shear load carrying capacity

iv) Radial tension carrying capacity
2.2 Effective Bending Rigidity, $I,E_c$

The deformation of reinforced concrete components is governed by its effective bending rigidity, $I,E_c$, in which $E_c$ is the modulus of elasticity of concrete, while $I_e$ is the effective bending rigidity of the transformed section. Herein, $I_e$ is governed by the moment of inertia of both transformed cracked and uncracked section as well as by the magnitude and distribution of the bending moment.

2.2.1 Modulus of Elasticity, $E_c$

Modulus of elasticity is an important property of concrete that is often used in structural design to compute deformations. The initial modulus of elasticity is given by

$$
E_c = 57,000 \sqrt{f_c}, \text{ lb/sq.inch} \nonumber
$$

$$
= 5000 \sqrt{f_c}, \text{ N/sq.mm} \tag{2.1}
$$

2.2.2 Transformed Moment of Inertia for Uncracked Section, $I_T$

Before cracking takes place in a conduit wall shown in figure 2.1, the moment of inertia of the pipe wall is given by the following expression
\[ I_T = \frac{bt^3}{12} + (n-1)A_{so}h_o^2 + (n-1)A_d h_i^2 - A_t y_T^2 \]  
(2.2)

where

\[ A_T = bt + (n-1)A_{so} + (n-1)A_{si} \]  
(2.3)

and \[ n = \text{ratio of modulus of elasticity of reinforcement to modulus of elasticity of concrete (the modular ratio)} \]

\[ y_T = \frac{(n-1)A_{so}h_o - (n-1)A_d h_i}{A_T} \]  
(2.4)

\[ h_i = \frac{t}{2} - t_{bs} - \frac{d_{si}}{2} \]  
(2.5)

\[ h_o = \frac{t}{2} - t_{bs} - \frac{d_{so}}{2} \]  
(2.6)

When the section is cracked, the moment of inertia of the section and the stresses in the reinforcing steel and concrete are developed for two stages.

2.2.3 Cracked Section with \( f_c \leq 0.5 f_c' \)

In this stage, the moment of inertia of the cracked section based on linear strain distribution (figure 2.2(b)) is given by the following expression
\[ I_{cr} = \frac{bt^3}{12} + (n-1)A_{sc}(Y_{cr}' - t_b)^2 + nA_{st}(d - Y_{cr})^2 \]
\[ + ba(Y_{cr}' - \frac{a}{2}) \]  
(2.7)

where

\[ Y_{cr}' = \frac{\left(\frac{ba^2}{2} + (n-1)A_{sc}t_b + nA_{st}d\right)}{A_{cr}} \]  
(2.8)

\[ A_{cr} = a + b + (n-1)A_{sc} + nA_{st} \]  
(2.9)

2.2.4 Cracked Section with $f_c > 0.5 f'_c$

In this case, moment of inertia for the cracked section is calculated based on a non-linear distribution of the stresses and strains (figure 2.2(c)). Heger and Liepins derived the expression for the moment of inertia as in the previous sections 2.2.2 and 2.2.3 based on the equilibrium and compatibility for the forces acting on the cross section and the strain induced on the cross section respectively. Moment of inertia for the cracked section was given by the following expression

\[ I_{cr} = \frac{b}{12}(a_1^3n_c^2a_2^3) + ba(Y_{cr}' - a_2 - \frac{a_1}{2})^2 \]
\[ + bn_c(a_2(Y_{cr}' - \frac{a_2}{2})^2 + (n-1)A_{sc}(Y_{cr}' - t_b)^2 \]
\[ + nA_{st}(d - Y_{cr})^2 \]  
(2.10)
\[ Y_{cr} = \frac{\left( \frac{n_e b a_2}{2} + b a_1 \left( a_2 + \frac{a_1}{2} \right) + (n-1)A_{sc} t_b + nA_{st} d \right)}{A_{cr}} \]  
(2.11)

\[ A_{cr} = n_e b a_2 + b a_1 + (n-1)A_{sc} + nA_{st} \]  
(2.12)

\[ n_e = \frac{E_{cl}^2}{E_{el}} \]  
(2.13)

2.2.5 Effective Moment of Inertia, \( I_e \)

Expressions (2.7) and (2.10) are valid only at the locations of cracks. For uncracked sections between flexural cracks, bond stresses transfer the tensile force from the reinforcing bars to the concrete and the moment of inertia for the sections away from the cracks is somewhat closer to the transformed sectional moment of inertia, \( I_T \). Thus, the effective moment of inertia, \( I_e \), for the conduit wall may be calculated using the equation proposed by Branson, 1963. This equation depends on the magnitude and distribution of the applied bending moment

\[ I_e = \left( \frac{M_c^a}{M_a} \right) I_T + [1 - \left( \frac{M_c^a}{M_a} \right)] I_{cr} \]  
(2.14)

where

\[ I_e = \text{Effective moment of inertia} \]
\[ M_{cr} = f_r \cdot I_T / y_t = \text{cracking moment} \]

\[ f_r = \text{Modulus of rupture} = 7.5 \sqrt{f'_c}, \quad \text{lb/in}^2 \]

\[ y_t = \text{Distance from the neutral axis of uncracked section to extreme fibre} \]

\[ I_T = \text{Transformed moment of inertia for the uncracked section} \]

\[ I_{cr} = \text{Transformed moment of inertia for the cracked section} \]

\[ M_u = \text{Applied moment (} M_u \text{) } M_{cr} \]

\[ \alpha = \text{Empirical transition constant, derived from load-deflection test data based on tests for steel reinforcement and is dependent on the material used, the type of bond as related to the surface roughness and on the reinforcing ratio } \ell. \]

The ACI Building Code 318-89 has incorporated equation (2.14) using a value of 3 for the exponent \( \alpha \). The value of \( \alpha \) is expected to be different when replacing the steel reinforcement with the GFRP.

2.3 Ultimate Bending Moment Capacity

In order to avoid sudden and brittle failure and also for general economy in design, structures following the ACI Code are required to be under-reinforced. Figure 2.3 shows the forces acting on a reinforced concrete flexural member at ultimate strength condition when subject to bending and axial compression considering the case of tension failure only. For an under-reinforced section, the flexural capacity \( M_u \) can be obtained by writing moment equilibrium about the point \( y = a / 2 \).
\[ M_u = N_u \left( \frac{h-a}{2} \right) + A f_y \left( d - \frac{a}{2} \right) \pm A' f_y \left( \frac{a}{2} - d' \right) \]  \hspace{1cm} (2.15)

where

\begin{align*}
M_u & = \text{Ultimate moment acting on the cross section} \\
N_u & = \text{Ultimate axial thrust acting on the cross section} \\
h & = \text{Overall thickness of the member} \\
A_e & = \text{Tension reinforcement area} \\
A'_e & = \text{Reinforcement area on the compressive side of the section} \\
f_y & = \text{Specified yield strength of reinforcement} \\
d & = \text{Distance from compression face to the centroid of the tension reinforcement} \\
d' & = \text{Distance from compression face to the centroid of the reinforcement near the compression face of the section} \\
a & = \text{The depth of the compression block at ultimate strength}
\end{align*}

2.4 Shear Load Carrying Capacity

The shear load carrying capacity is affected by the depth of cracks which is related to the reinforcement ratio \( q \) and the modulus of elasticity of the reinforcing bars. Also, the decrease of \( V/N \) ratio has an effect on the crack formation and may lead to increase in the shear load carrying capacity. Shear formulas have been developed by a number of investigators based mainly on test data of structures reinforced with steel bars. The shear capacity for reinforced concrete members without shear reinforcement is given
by a detailed method for members subjected to shear and flexure by the ACI 318-89 clause 11.3.2.1 as

\[ v_c = 1.9\sqrt{f_c'} + 2500 \rho \frac{V_u d}{M_u}, \text{lb/sq.inch} \]  

(2.16)

but not greater than \(3.5\sqrt{f_c'}\), and \(V_u d / M_u\) is not to be taken greater than 1.0. For members subject to axial compression, equation (2.16) may be used with modified bending moment \(M_m\) substituted for \(M_u\) and \(V_u d / M_u\) not then limited to 1.0, clause 11.3.2.2, where

\[ M_m = M_u - N_u \frac{(4h - d)}{8} \]  

(2.17)

where

\begin{align*}
M_u &= \text{Factored moment at a section} \\
h &= \text{Overall thickness of the member} \\
d &= \text{Distance from extreme compression fibre to the centroid of the longitudinal tension reinforcement.} \\
N_u &= \text{Factored axial load acting normal to cross section occurring simultaneously with } V_u; \text{ to be taken positive for compression, negative for tension, and to include effects of tension due to creep and shrinkage.} \\

\text{However, } v_c \text{ shall not be taken greater than}
\end{align*}

\[ v_c = 3.5\sqrt{f_c'} \sqrt{1 + \frac{N_u}{50A_g}}, \text{lb/sq.inch} \]  

(2.18)

where
\[ A_s = \text{Gross area of the cross section} \]

And \( N_s / A_s \) is expressed in psi. When \( M_m \) as computed by equation (2.17) is negative, \( v_c \) shall be computed by equation (2.18).

Extensive analysis based on experimental test data by Heger and MacGrath showed that the reinforcement ratio \( q = A_s / bd \) and the ratio of \( M/Vd \) both have a significant influence on shear strength.

On the other hand, Heger and MacGrath concluded that for members that carry distributed loadings, the governing shear strength usually occurs at sections where \( M/(V\phi d) = 3.0 \).

This is the lowest shear strength at any location in a member with a constant \( q \) and \( bd \).

They proposed the following formulas for computing the shear strength at section where \( M / (V\phi d) \geq 3.0 \)

\[ \phi V_b = \frac{\phi v_b d b}{F_N F_c} \quad (2.19) \]

where

\( \phi \) = Capacity reduction factor for shear as defined in AC 318, or other design standards

\( v_b \) = The nominal basic shear strength at sections where \( M / (V\phi d) \geq 3.0 \)

\[ v_b = (1.1 + 63 \rho) f_c' \leq 2.3 \sqrt{f_c'}, \quad \text{lb/sq.inch} \quad (2.20) \]

Maximum compressive strength for concrete \( f_c' \) not to exceed 7000 psi (48 Mpa).

\( F_N \) = the thrust factor. When \( N_u \) is compressive (i.e. +ve)

when \( N_u \) is tensile (i.e. -ve), use equation (2.22) up to \( N_u / V_u = -1.0 \)
\[ F_N = 1.0 - 0.12 \frac{N_u}{V_u} \geq 0.75 \quad (2.21) \]

\[ F_N = 1.0 - 0.24 \frac{N_u}{V_u} \quad (2.22) \]

\( F_c \) is a curvature and crack depth factor for circular pipes when moment produces tension on the inside of a pipe (radial tension curvature)

\[ F_c = \left( 1 + \frac{d}{2r_m} \right) \frac{1}{F_d F_{pr}} \quad (2.23) \]

when moment produces compression on the inside of a pipe (radial compression curvature)

\[ F_c = \left( 1 - \frac{d}{2r_m} \right) \frac{1}{F_d F_{pr}} \quad (2.24) \]

\( r_m \) is Radius to centroid of pipe wall

\( F_d \) is Depth factor that is described as

\[ F_d = (0.8 + \frac{1.6}{d}) \leq 1.25 \quad (2.25) \]

\( F_{pr} \) is a factor used to reflect the variation that local materials and manufacturing process can have on the tensile strength of concrete and this can be taken 1.0 in most cases.

The shear strength at section where \( M / (V\phi d) < 3.0 \)
\[ V_c = \frac{4 V_b}{M_{n u}} + 1 \]  \hspace{1cm} (2.26)

Where

- \( V_c \) = Nominal shear strength provided by concrete cross section, N per width b
- \( V_u \) = Factored shear force acting on cross section, N per width b
- \( M_{n u} \) = Factored moment as modified for effect of compressive or tensile thrust, N.mm per width b

Another empirical formula was developed by (Zsutty) for the ultimate shear strength of beams without using vertical stirrups as follows:

\[ v_c = 60(f' c \cdot \rho \frac{d}{a})^{\frac{1}{2}} \text{, lb/sq.inch} \]  \hspace{1cm} (2.27)

where \( a = \) Shear span

Rajagopalan and Ferguson found that the shear strength of reinforced concrete beams as given by ACI 318-63 to be unconservative when the ratio of the longitudinal reinforcement, \( \rho \), is small. They proposed the following equation to predict the shear capacity for beams with reinforcing ratio less than 1%:

\[ v_c = (0.8 + 100 \rho) \sqrt{f' c} \text{, lb/sq.inch} \]  \hspace{1cm} (2.28)
2.5 Radial Tension Capacity

Failure due to radial tension in the concrete results in circumferential cracking at the inner tension reinforcement and "Slabbing off" of the concrete. Such failure can be produced by the radial component of the flexural tensile or compressive force in the reinforcement. It is affected by the M/N ratio as well as the type and rigidity of the reinforcing mesh. Heger and MacGrath developed equations to predict the radial tension strength of curved flexural members based on circular pipes loaded in three-edge bearing tests. The maximum ultimate moment that can be developed at the full radial tension strength of the concrete is

\[ M_u = 1.2 \phi \sqrt{f_c} \cdot F_{pr} \cdot b \cdot d \cdot r_s + 0.45 \cdot N_u \cdot d \quad (2.29) \]

where

\[ F_{pr} = \frac{(DL_{ut} + \frac{9w_p}{S})}{1230 \cdot r_s \cdot d \cdot \sqrt{f_c} \cdot S(S + h)} \quad (2.30) \]

where

- \( F_{pr} \) = Factor for process and local materials characteristics that affect radial tension strength of the pipe
- \( r_s \) = Radius to inside reinforcement
- \( DL_{ut} \) = D-load ultimate strength of test pipe where D-load is total test load per foot of pipe length divided pipe inside diameter
- \( w_p \) = weight of pipe
- \( S \) = Inside span of pipe

20
h = Overall wall thickness of the pipe

A close approximation of the maximum area of tension reinforcement that can develop its tensile yield strength before radial tension failure takes place is

\[ \max A_s = \frac{1.33 \, b \, r_s \, f_c' \, F_{pr}}{f_y} \]  

(2.31)
FIG. 2-1  UNCRACKED SECTION OF A PRECAST CIRCULAR CONCRETE PIPE WITH DOUBLE CIRCULAR REINFORCEMENT
FIG. 2-2 CRACKED SECTION OF PRECAST CIRCULAR CONCRETE PIPE
FIG. 2-3 FORCES ACTING ON A REINFORCED CONCRETE FLEXURAL MEMBER AT ULTIMATE STRENGTH CONDITION
CHAPTER III

EXPERIMENTAL PROGRAM

3.1 General

Chapter II discussed the different formulas for the design of concrete conduits with steel reinforcement. Tests are needed to examine the behaviour of circular walls reinforced with Glass Fibre Reinforced Plastics (GFRP) under bending, shear, and axial load.

An experimental program was carried out to study the behaviour of circular arches reinforced with GFRP and compare their behaviour with circular arches reinforced with steel bars. The main objective of this experimental investigation is to determine the validity of the available design formulas as related to the bending rigidity and load carrying capacity.
Tests were carried out on twelve reinforced concrete models. The first group, including models #1 and #2, was directed to study the radial tension capacity for panels reinforced with either steel or GFRP respectively using 540 mm radius circular reinforced concrete arches. The second group, including models #3, #4, #5, and #6 was directed to study the flexural capacity of 1000 mm radius circular reinforced concrete arches with different types of reinforcements. The third group, including model #7 and #8, was directed to study the effect of axial forces, on bending moment, and shear load carrying capacity in reinforced concrete arches with GFRP. Herein, 1000 mm circular reinforced concrete arches are provided with ties at the base of the arch to induce axial forces in the arch. Finally, the last group was directed to study the behaviour of reinforced concrete slabs with steel and GFRP subjected to loads causing shear failure.

3.2 Description of Test Models

Models #1 and #2, are simply supported circular arches with a radius of 540 mm, 700 mm deep, and a thickness of 150 mm. For Model #1, two cages of steel reinforcement, 10M spaced at 100 mm for both circumferential and longitudinal bars, one cage on each face of the arch. For Model #2, two cages of Nefmac reinforcement, one in each face of the arch. Detailed geometry of arch Model #1 and #2 is shown in Figure 3-1. For the second group, Model #3, #4, #5, and #6 geometry is the same as group #1 but the radius of the arch is 1000 mm. For Model #3, steel reinforcement was used in two cages, 10M spaced at 100 mm for the circumferential and longitudinal bars for each cage. For Model #4 and #6, two cages of GFRP (Nefmac) were used, and for Model #5 two
cages, 13M spaced at 100 mm for the circumferential and longitudinal bars for each cage. For Model #4 and #6, two cages of GFRP (Nefmac) were used, and for Model #5 two cages of GFRP "Polyester PSI Fiberbar" were used. 13M spaced at 100 mm for the inside circumferential bars and 10M spaced at 100 mm for the outside circumferential bars while 10M spaced at 100 mm bars were used in the longitudinal direction. Figure 3-2 shows the geometry of Models #3 through #6.

For the third group, geometry is the same as in group two. In this group, each arch Model was tied at the base using 2-φ7 mm high tensile strength steel as shown in Figure 3-3. For Model #7, two cages of Nefmac reinforcement were placed at each face while in Model #8, two cages of Polyester PSI Fiberbar were used as in Model #5.

The last group contained slab Models #9, #10, #11, and #12 as shown in Figure 3-4. Concrete was 350 mm width, 150 mm depth and 1300 mm simply supported span. Model #9 was reinforced with 4-10M normal steel reinforcement. Model #10 was reinforced with Nefmac grids. Model #11 was reinforced with 4-13M "Polyester PSI Fiberbar" and Model #12 was identical to Model #11.
3.3 Material

3.3.1 Concrete

3.3.1.1 Cement

Type 30, CSA high early strength cement was used in the concrete mix of the models. This type of portland cement accelerates the hydration process resulting in rapid hardening and development of strength. This made it possible to test the models 7 days after casting.

3.3.1.2 Coarse Aggregates

The coarse aggregates consisted of hard, clean, crushed, stone of maximum size restricted to 10 mm in diameter, since the narrowest dimension between the sides of the form work was equal to 25 mm and the concrete cover over the reinforcement was 10 mm.

3.3.1.3 Fine Aggregate

Coarse sand from lake Erie was used in the concrete mix. This sand, available in the structural laboratory was assumed to be free of chemicals, coating of clay, or other fine materials that may affect hydration reaction and bond of the cement paste.

3.3.1.4 Mixing Water

Natural tap water having no impurities was used in the concrete mix. Water cement ratio of about 0.45 was selected to achieve the required concrete strength of 30
MPa. A concrete mixer of 0.14 m³ charging capacity and electrically operated was used for mixing the required concrete. Two batches were required for each model of #1 and #2, four batches were required for each of the model #3 through #8 while only one batch was needed for each slab model. Two cylinders were taken for each model in group #1, four cylinders were taken for each model in groups #2 and #3 while two cylinders were taken for each slab model. Figure 3-5 shows some cylinders which were tested on the day of model testing to determine the concrete compressive strength. Appendix A-1 contains the concrete mix design. The concrete compressive strength for all models are shown in table 3-1.

3.3.2 GFRP

3.3.2.1 Nefmac

The longitudinal and transverse bars in Nefmac FRP composite grids are fabricated using a process in which glass fibre filaments are impregnated with vinyl ester resin, and then woven in two or three dimensional patterns to form the reinforcing grid. The finished grids are then pressed between heated steel plates which flatten the upper and lower surfaces of the bars. Figure 3-6 shows the placement of Nefmac FRP grids inside the formwork for the circular arch modes. Figure 3-7(a) shows the Nefmac FRP grids, the bars are smooth on top and bottom with irregular sides. The grid used in this experimental program was produced with equal sized longitudinal and transverse bars equally spaced in both directions at 150 mm from centreline to centreline of the bars. The manufacturer, designates the Nefmac grid as G10, provided the properties of material
which was tested at the laboratories of the National Research Council (NRC) as follows:

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross Sectional Area</td>
<td>78.8 mm²</td>
</tr>
<tr>
<td>Ultimate Load</td>
<td>55.0 kN</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>600 MPa</td>
</tr>
<tr>
<td>Modulus of Elasticity</td>
<td>30 GPa</td>
</tr>
</tbody>
</table>

### 3.3.2.2 POLYESTER PSI FIBERBAR

Polyester PSI Fiberbar is made up of 70% E-glass and 30% polyester resin. Longitudinal fibres of glass are thoroughly coated in a bath of resin and drawn to merge together into a circular die. The smooth round bar emerging through this die is then covered with a nexus veil to protect the longitudinal glass fibres and to increase the bond strength. Finally the bar is wrapped with additional strands of E-glass at a 45° angle, the resulting spiral indentation produces a further increase in bond strength in the same way that deforming steel bars increases their bond strength. Figure 3-7(b) shows a typical shape of the Polyester PSI Fiberbar which is produced in standard bar sizes from #2 bar with 1/4 in. diameter (6.3 mm) to #9 bar with 1.125 in. diameter (28.6 mm).

The manufacturer provided the experimental test data for two different sizes, namely, #3 bars with 3/8 in. diameter (9.5 mm) and #4 bars with 1/2 in. diameter (12.7 mm). The physical properties of Polyester PSI Fiberbar which are found to be as follows:

a) Properties of the composite bars:

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile Modulus</td>
<td>41.3 GPa (6 x 10⁶ psi)</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>551.6 MPa (80,000 psi)</td>
</tr>
</tbody>
</table>
b) Properties of the resin matrix

\[
\begin{align*}
\text{Tensile Modulus} &= 3.172 \text{ GPa} (0.46 \times 10^6 \text{ psi}) \\
\text{Tensile strength} &= 77.2 \text{ MPa} (11,200 \text{ psi})
\end{align*}
\]

c) Properties of the fibre reinforcement

\[
\begin{align*}
\text{Tensile Modulus} &= 72.4 \text{ GPa} (10.5 \times 10^6 \text{ psi}) \\
\text{Tensile strength} &= 3447.5 \text{ MPa} (500,000 \text{ psi})
\end{align*}
\]

3.3.3 High Tensile Strength Wires

High tensile strength wires were used as ties at the base of Model #7 and Model #8 to study the effect of the axial forces acting on the cross sections. Each of the wires has a diameter of 7 mm (0.276 in), with ultimate strength of 1760 MPa (225,000 psi).

3.3.4 Auxiliary Materials

The following items were used during the construction of the models:

1. 3/4 in. (19.1 mm) thick plywood for the form work.

2. Aluminum sheets of 2 mm (0.08 in) thickness were curved and used as an inside and outside perimeter of the Arch models.

3. Thin steel wires were used to tie the curved bars to the straight bars in both Arch model containing steel bars and Polyester PSI Fibrebar bars.
3.4 Instrumentation

3.4.1 Strain Gauges on High Tensile Strength Wires

In order to measure the strain on the ties at the base of the arch models No. 7 and No. 8, foil strain gauges of type N11-FA-10-120-11, having a length of 10 mm (0.39 in) and an average resistance of 119.8 ohms and a gauge factor of 2.13 ± 1% were used. The surface of the steel was smooth using sandpaper No. 600. All dust was removed and the surface was cleaned with Acetone. The surface was then ready for the installation of the strain gauges by applying a conditioner followed by a neutralizer. The gauges were mounted using an M-Bond AE10 adhesive with a 200 catalyst as bonding agent. After applying pressure on the gauge for one minute, the wires were soldered to the strain gauges using lead wires. The wires were then connected to the automatic strain indicator.

3.4.2 Mechanical Dial Gauges

Mechanical dial gauges with travel sensitivity 0.025 mm (0.001 in) were used to measure the deflection of the models as well as the movement of the roller support under the external applied load. The locations of dial gauges are shown in figure 3-9, and are seen in positions in figures 3-10 and 3-11.

3.5 Test Equipment

3.5.1 Hydraulic Jack

A hydraulic jack was used for the application of the load figure 3-12. The jack of capacity 440,000 N (100,000 lb) was supported by a rigid portal frame system.
3.5.2 Universal Flat Load Cell

A Universal flat load cell model FL 100 U (C)-25GKT, S/N: 0011724-1 having a capacity of 440 kN (100 Kips) was used to determine the magnitude of the applied concentrated load. Figure 3-12 shows the universal load cell attached to the hydraulic jack. The calibration of the universal load cell is given in figure 3-13.

3.5.3 Strain Indicator

The strain indicator Model P-3500 manufactured by Vishay Intertecnhology was used to record the strains from strain gauges as well as to record the strains on the load cell and then convert these strains using the calibration curve to loads. The Model P-3500 strain indicator is a portable and battery-powered precision instrument. Strain gauges were connected via the front panel binding posts. The P-3500 Model will accept gauge factors of 0.500 to 9.900 and gauge factor is settable to an accuracy 0.001 by a front panel ten-turn potentiometer. Gauge factor is displayed by the front panel readout. (Figure 3-14)

3.6 Preparation of the Test Models

1. For the different Arch models, the base of the forms were drawn on a sheet of plywood 19.1 mm (3/4 in) thick and then cut into the required dimensions. The sides were made of aluminum sheets 2 mm (0.08 in) thick and curved and then screwed to the base.

2. The required reinforcement for the different test models were placed inside the
form and C-clamps were used to maintain the two sides of the form parallel to each other. Both ends of the Arch models were fastened to a 19.1 mm (3/4 in) thick plywood to ensure stability while casting the concrete as shown in fig. 3-15.

3. The required weights of concrete components were prepared for each batch. First, the coarse aggregates, the fine aggregate, and the cement were dry mixed in the concrete mixer for about 3 minutes then the water was added in stages while mixing. Four standard cylinders were taken during the casting process, one from each batch. Appropriate compaction was applied by using an electrical hand vibrator. After casting the models and cylinders, all surfaces were given a smooth final finish by hand trowelling (Figure 3-16).

4. During casting, a slump test was conducted on the wet concrete by filling a semi cone, open from both ends, with concrete in three layers and each layer was compacted using a standard rod, the semi cone was pulled slowly upward and the distance from the top of the cone to the top of the settled concrete was measured to be 75 mm (3 in) which is the value of slump. (Figure 3-17)

5. Few hours after casting, wet burlap sheets (shown in figure 3-18) were placed on the concrete surface. Curing continued for two or three days until the concrete surface seemed sufficiently wet. The concrete cylinders were taken out of the aluminum moulds one day after casting and submerged under water in a container in the curing room.

6. After two days, the form sides were removed and the model was sprayed by water and left for another five days to gain the desired strength. Then the model was
moved to the place of testing and the strain gauges were put on the concrete surface as described in section 3.4.1.

For models #1 and #2, as shown in figures 3-19 and 3-20, the rollers set on heavy steel I-beam sections braced with vertical stiffeners along the web to ensure stability, while the other Arch models, rollers set directly on the testing table because of the difference in the Arch rise between the first group of Arch models and the other two groups of the Arch models. Figure 3-21

7. The mechanical dial gauges were placed in position under the two concentrated line loads, and at mid span, to measure the deflection. Also a mechanical dial gauge was placed on the side to measure the horizontal movement of the roller during the loading process.

3.7 Experimental Test Procedure

The testing process was carried out to study the behaviour of advanced composite materials in reinforced concrete Arches. The scope of the experiments was focused on determining:

1. deflected shape.

2. the cracking load, the propagation of cracks with the increase of load, and the crack pattern before and at failure.

3. the ultimate capacity of the tested models.
Table 3.1 Concrete Compressive Strength for Tested Models (MPa)

<table>
<thead>
<tr>
<th></th>
<th>Concrete Compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cylinder (1)</td>
</tr>
<tr>
<td>Arch Model #1</td>
<td>35</td>
</tr>
<tr>
<td>Arch Model #2</td>
<td>40.9</td>
</tr>
<tr>
<td>Arch Model #3</td>
<td>50.3</td>
</tr>
<tr>
<td>Arch Model #4</td>
<td>42.6</td>
</tr>
<tr>
<td>Arch Model #5</td>
<td>40.3</td>
</tr>
<tr>
<td>Arch Model #6</td>
<td>46.8</td>
</tr>
<tr>
<td>Arch Model #7</td>
<td>42.8</td>
</tr>
<tr>
<td>Arch Model #8</td>
<td>46.3</td>
</tr>
<tr>
<td>Slab Model #9</td>
<td>44.6</td>
</tr>
<tr>
<td>Slab Model #10</td>
<td>25.1</td>
</tr>
<tr>
<td>Slab Model #11</td>
<td>27.4</td>
</tr>
<tr>
<td>Slab Model #12</td>
<td>32.4</td>
</tr>
</tbody>
</table>
FIG. 3-1 GEOMETRY AND REINFORCEMENT OF MODELS (1) AND (2)
FIG.3-2 GEOMETRY AND REINFORCEMENT OF MODELS (3), (4), (5), AND (6)

Transverse section in model (3)
FIG. 3-3 GEOMETRY AND REINFORCEMENT OF ARCH MODELS (7) AND (8)
FIG. 3-4  GEOMETRY AND REINFORCEMENT FOR SLAB MODELS (9), (10), (11), AND (12)
FIG. 3-5  SOME TESTED CONCRETE CYLINDERS
FIG. 3-6  NEFMAC GRID PLACEMENT IN THE FORMWORK
FIG. 3-7 ADVANCED COMPOSITE MATERIALS (ACM) USED IN THE TEST MODELS
(a) Dial gauge locations for the arch models

(b) Dial gauge locations for the slab models

Fig. 3-8 Dial gauge arrangement for the different arch and slab models
FIG. 3-9 DIAL GAUGES IN POSITION FOR THE DIFFERENT ARCH MODELS
FIG. 3-10 DIAL GAUGES IN POSITION FOR THE DIFFERENT SLAB MODELS
FIG. 3-11  THE HYDRAULIC JACK AND UNIVERSAL LOAD CELL (440 KN CAPACITY)
FIG. 3-12 CALIBRATION OF THE UNIVERSAL LOAD CELL (LOAD VS STRAIN)
FIG. 3-13  THE P-3500 MODEL STRAIN INDICATOR
FIG. 3-14  THE FORMWORK FOR THE DIFFERENT ARCH MODELS
FIG. 3-15 CASTING OF CONCRETE
FIG. 3-16  THE SLUMP TEST
FIG. 3-17 CONCRETE CURING AFTER CASTING
FIG. 3-18  TYPICAL ROLLER SUPPORT
FIG. 3-19 TYPICAL HINGED SUPPORT
CHAPTER IV

ANALYSIS OF RESULTS

4.1 General:

In this chapter, the results achieved from the experimental program are presented and compared with the theoretical ones. Tests were conducted on twelve different models which were categorized as four groups. The first group was designed to test the radial tension failure in the arch models, while the second group was designed to study the flexural failure. The third group was designed to study the effect of axial forces (thrust) on the failure load, and finally, the fourth group was designed to study the shear failure in slabs. For each arch or slab model, the load was applied in increments up to failure. Crack pattern, cracking load, the shape of failure, ultimate load, and deflection of the model are all presented. Finally, a comparative study is presented to show the behaviour of the different test models under the same static loading conditions.
4.2 Arch Model #1

Arch Model #1 was a simply supported reinforced concrete arch with 1% steel reinforcement in each face (each cage), an inside radius of 540 mm, a span of 1065 mm, and a thickness of 150 mm. The flexural capacity of the model is 32.5 kN.m, the resisting moment of the cross section at the shear failure of the model is 41.7 kN.m based on eq. (2.19) by Heger and MacGrath, and the capacity of the model at radial tension failure is 26.1 kN.m (Appendix B). Therefore, it is expected that the governing failure criteria will be radial tension.

Crack initially started as a bending crack at the section of maximum bending moment, then towards the ultimate load, two different cracks appeared along the top and the bottom cages of reinforcement. The top crack is a product of the compressive force in the top reinforcement producing the radial component which is approximately equal to the component at the tension steel causing failure along the bottom cage. At ultimate load, the initial bending crack intersected with the radial tension crack surface leading to the complete collapse of the model as shown in fig.4.1. Fig.4.2 shows the load deflection curve at different locations of the arch model. Dial gauge No.1 and No.3 are almost identical due to the symmetry of the arch model. At cracking, the arch model loses some of its bending rigidity due to the development of cracks, but it continues to provide resistance to the applied load until failure. The maximum deflection prior to failure was 10.18 mm. Fig.4.3 shows a comparison between the theoretical and experimental deflected shape of the model to be within 10% average difference at a total applied load of 124.60 kN and using a bending rigidity of $E_I c_r$ (Appendix B). The theoretical deflected shape of
the arch model has been obtained using FINESSE program. FINESSE is a structural analysis software for two & three dimensional trusses and frames. For the problem at hand, the arches have to be modeled as two dimensional frames.

4.3 Arch Model #2

Arch model #2 had the same dimensions as arch model #1, except the replacement of steel reinforcement by NEFMAC grids. The flexural capacity of the model is 31.0 kN.m, the resisting moment of the cross section at the shear failure of the model is 38.9 kN.m based on eq. (2.19), and the capacity of the model in radial tension failure is 29.8 kN.m. Therefore, it is expected that the governing failure criteria will be radial tension failure (Appendix B). Bending cracks started at the section of maximum bending moment same as in arch model #1, and when approaching the ultimate load, a large crack developed along the top cage of reinforcement leading to a radial tension failure but with no crack at the bottom cage of reinforcement as in arch model #1 (see figure 4.4). It may be noted that the cracking formulates at about 25% of the failure load of the specimen. Figure 4.5 shows the load deflection curve of the model. Dial gauges No.1 and No.2 were removed shortly after cracking to avoid damaging those gauges if sudden failure takes place. It was noticed that after cracking, the model continued to resist the applied load with a reduced bending rigidity until failure load. The failure load of arch model #1 and #2 are very close to one another, this shows that the radial tension capacity of arch models is independent of the material of reinforcing bars but rather by the arrangement and curvature of the tensile and compressive forces induced by the bending moment.
Fig. 4.6 shows a comparison between the theoretical and experimental deflected shape of the model at a total applied load of 45.50 kN and using a bending rigidity of $E_I\ell_r$.

4.4 Arch Model #3

Arch Model #3 was a simply supported reinforced concrete arch with 1% steel reinforcement in each face (each cage), an inside radius of 1000 mm, a span of 1900 mm, and a thickness of 150 mm. The flexural capacity of the model is 33.5 kN.m, the resisting moment of the cross section at the shear failure of the model is 110.9 kN.m based on eq. (2.19), the capacity of the model at radial tension failure is 60.8 kN.m. Therefore, it is expected that the governing failure criteria will be tension failure (Appendix B). Figure 4.7 shows the crack pattern which started at 21% of the ultimate load achieved by the model. With the increase of the load, a large bending crack near the location of the load started to widen and propagate upwards towards the location of the load on the top fibre of the model leading to tension failure at the ultimate load. Another crack appeared near the centreline of the arch model (figure 4.8). That crack was a result of the imperfection due to the improper placement of the steel reinforcing bars of the inside cage with 50 mm concrete cover instead of 25 mm which led to a section with a reduced bending capacity of 30.3 kN.m instead of the initially designed bending capacity of 33.3 kN.m. Figure 4.9 shows the load deflection curve for the arch model, a maximum horizontal displacement of 24.6 mm was recorded at the roller support. Figure 4.10 shows the deflection distribution for the arch model, theoretical deflection is as much as double the experimental deflection because of the use of $\ell_r$ to calculate the theoretical deflection, the
actual bending rigidity at this stage of loading is $E_A I_A$, and based on the calculations illustrated in Appendix B, the exponential $\alpha$ in eq.(2.14) was found to be 1.60. This value is different from the value of 3.0 adopted by the ACI 318-89 Code. The reason may be due to the fact that the value of 3.0 is applicable in slabs where axial forces are negligible but when axial forces become major factor, such as in culverts, this value is lower than 3.0.

### 4.5 Arch Model #4

Arch model #4 is identical to arch model #3, except the replacement of steel reinforcement by NEFMAC grids. The flexural capacity of the model is 31.0 kN.m, the resisting moment of the cross section at the shear failure of the model is 90.8 kN.m based on eq. (2.19) by Heger and MacGrath, and the capacity of the model at radial tension failure is 42.3 kN.m. Therefore, it is expected that the governing failure criteria will be tension failure.

Bending cracks were observed as shown in figure 4.11 and one of these cracks extended to the top fibre causing the expected tension failure of the specimen. Failure of the model was sudden as shown in figure 4.12. Figure 4.13 shows the load deflection curve for the model, dial gauges No.1, No.2, and No.3 were removed shortly after cracking to avoid damaging them at the failure of the model. The choice of NEFMAC grids was based on $(A_o \cdot f_o)_{steel} = (A_o \cdot f_o)_{NEFMAC}$, therefore, it was expected that the flexural capacity of both arch models #3 and #4 would be the same. The experimental ultimate load for arch model #4 was found to be 23% less than the theoretical ultimate load expected at the tension failure.
of the arch model. Maximum horizontal displacement prior to failure was 43.3 mm compared to 24.6 mm for arch model #3. Figure 4.14 shows the deflection distribution for the arch model, theoretical deflection is 80% higher than the experimental deflection because of the use of \( L_r \) to calculate the theoretical deflection, the actual bending rigidity at this stage of loading is \( E_c L_r \), and based on the calculations illustrated in Appendix B, the exponential \( \alpha \) in eq.(2.14) was found to be 6.3. This value is much higher than the value of 1.6 for arch model #3. This may be attributed to the larger size and magnitude of cracks in the arch model reinforced with NEFMAC than the arch model reinforced with steel bars which, in turn, affects the effective bending rigidity of arch model \( E_c L_r \).

4.6 Arch Model #5

Arch model #5 is identical to arch model #3 and #4, except the reinforcement was the use of Polyester PSI Fiberbars, 10M spaced at 100mm centre to centre for the outside cage and 13M spaced at 100 mm centre to centre for the inside cage. The flexural capacity of the model is 41.4 kN.m, the resisting moment of the cross section at the shear failure of the model at the location of the load is 47.1 kN.m based on eq. (2.27) by Zsutty, and the capacity of the model at radial tension failure is 55.6 kN.m. Therefore, it is expected that the governing failure criteria will be tension failure. Unfortunately, the development of the photographs of this model was not successful but the crack pattern was very similar to the crack pattern of arch model #4 and the failure mode was due to shear. This means that the shear load carrying capacity formulas used in structures reinforced with steel cannot be used in structures reinforced with GFRP without a
reduction factor applied to the different formulas to account for the mechanical properties of GFRP which are different from the mechanical properties of steel reinforcement. The cracking load was recorded at 21.8% of the ultimate load of the model. Maximum deflection prior to failure was 34.9 mm compares to 24.6 mm for arch model #3 reinforced with steel and 43.3 mm for arch model #4 reinforced with NEFMAC grids. Figure 4.16 shows the deflection distribution for the arch model, the average theoretical deflection is as much as 2.5 times the average experimental deflection because of the use of \( I_r \) to calculate the theoretical deflection, the actual bending rigidity at this stage of loading is \( E_c I_r \), and based on the calculations illustrated in Appendix B, the exponential \( \alpha \) in eq.(2.14) was found to be 2.6. This value is higher than the value of 1.6 for arch model #3. This may be attributed to the larger size and magnitude of cracks in the arch model reinforced with Polyester bars than the arch model reinforced with steel bars which affects the effective bending rigidity of arch model \( E_c I_r \).

4.5 Arch Model #6

Arch model #6 is identical to arch model #4. Failure criteria was expected to be tension failure. Bending cracks were observed as shown in figure 4.17 and one of these cracks extended to the top fibre causing the expected tension failure of the specimen. Failure of the model was sudden as shown in figure 4.18. Dial gauges No.1, No.2, and No.3 were removed shortly after cracking to avoid damaging them at the failure of the model as shown in figure 4.19, but it can be noticed that they follow the pattern as dial gauge No.4 at the roller support. Behaviour of arch model #6 was very similar to arch
model #4, failure was in tension in both models and occurred at the location of maximum bending moment near the location of the applied load, crack pattern was very close to one another, and the failure load was 57.2 kN for model #4 while it was 57.0 kN for model #6. Maximum deflection prior to failure was 43 mm in both arch model #4 and #6. As expected, a good agreement was observed between arch models #4 and #6. Fig 4.20 shows a comparison between the experimental and theoretical deflected shape of the model at a total applied load of 35.6 kN and using \( I_e \) as the bending rigidity of the cross section, the average theoretical deflection is as much as 2.45 times the average experimental deflection. This means that the effective bending rigidity of the arch model at this stage of loading is 2.45 times the bending rigidity of the cracked section.

4.8 Arch Model #7

Arch model #7 is identical to arch models #4 and #6, except the presence of two high tensile strength steel wires at the base of the arch to study the behaviour of the arch model when thrust force is present. The model was loaded until the ties snapped due to the weak end connections, ties were replaced and end anchorages, usually used in the end anchorage of the prestressing systems, were used and test of the model was designated as model #7A. When the ties failed, several bending cracks were noticed as shown in figure 4.21, the depth of those cracks was a little longer than the mark 10 (at a total applied load of 44.5 kN) shown in the figure. When the load was applied to arch model #7A, all those cracks propagated upward towards the outside fibre, then prior to failure of the model, a large shear crack near the location of the applied load was developed and
this crack was the cause of failure for the model in shear as shown in figure 4.22. The ultimate load at failure was 267.0 kN, and shear load at failed section of the model was 97.5 kN. According to eq.(2.16) by ACI code, the shear load carrying capacity of the model is 87.0 kN, while the shear load carrying capacity using eq.(2.19) by Heger and MacGrath is 93.5 kN. Both equations (2.16) and (2.19) are suitable to represent arch model #7A, a reduction factor is not needed to be applied in both equations to account for the use of GFRP instead of steel reinforcement. Figure 4.23 shows the load deflection curve for arch model #7, the model continued to resist the applied load with little reduction in bending rigidity due to the presence of compressive forces in the arch membrane as a result of the ties action at the base of the arch. Figure 4.25 shows the applied load vs the strain in the ties, the increase of the axial force of the ties can be noticed after cracking due to the reduction of the effective bending rigidity $E_cI_c$. Figure 4.26 shows the load deflection curve for arch model #7A, the slope of the curve which represents the bending rigidity $EI$ of the model was very close to $EI$ of arch model #7 after cracking. This is due to the formation of cracks at the end of testing model #7 when the ties failed. Figure 4.27 shows a comparison between the experimental and theoretical deflected shape of the model at a total applied load of 178.0 kN and using $I_c$ as the bending rigidity of the cross section, the difference was found to be within 7% for the average deflected shape of the arch model. This means that the effective bending rigidity of the arch model is very close to bending rigidity of the cracked section of the arch model.
4.9 Arch Model #8

Arch model #8 is identical to arch model #5, except the presence of two high tensile strength steel wires at the base of the arch to study the behaviour of the arch model when thrust force is present. The load was applied up to about 60% of the ultimate load at failure of the model to study the effect of repetitive loading on the model. Figure 4.29 to figure 4.32 show the load deflection relationship at different locations of the arch model, the model was cracked at about 14% of the ultimate load at failure of the specimen, a sudden increase in deflection was noticed at cracking followed by a linear relation up to the release of the loading. When the load was released, the model did not return to its origin, but a permanent deformation of about 4 mm average was developed. When the load was re-applied on the model, there was a similar behaviour to the initial application of loading with about 40% increase in the effective bending rigidity EI compared to the effective bending rigidity after cracking when the loading was initially applied. Crack pattern was very similar to arch model #7, and prior to failure of the model, a large crack along the shear surface of the model at the location of the applied load was developed and this crack was the cause of failure for the model in shear same as in arch model #7. The ultimate load at failure was 311.5 kN, and shear load at failed section of the model was 115.70 kN. According to eq.(2.16), the shear load carrying capacity of the model is 92.5 kN, while the shear load carrying capacity using eq.(2.19) is 101.0 kN. Both equations (2.16) and (2.19) are suitable to represent arch model #8, a reduction factor is not needed to be applied in both equations to account for the use of GFRP instead of steel reinforcing bars. Figure 4.33 shows a comparison between the
experimental and theoretical deflected shape of the model at a total applied load of 178.0 kN and using $l_{cr}$ as the bending rigidity of the cross section, the difference was found to be within 6% for the average deflected shape of the arch model. Figure 4.34 shows the relationship between the applied load and the strain in the ties, it is noticed that the axial force of the ties was not affected by the repetitive loading applied on arch model #8.

4.10 Slab Model #9

Slab model #9 is a simply supported reinforced concrete slab with 1% steel reinforcing bars, 350 mm width, 1200 mm span length, and 150 mm concrete depth with 25 mm concrete cover. The shear load carrying capacity of the model using eq.(2.19) is 39.9 kN, and using eq.(2.28) is 41.1 kN, while the load to cause tension failure is 53.7 kN, therefore, it is expected model will fail due to shear. Cracks started as bending cracks at the region of maximum bending moment as shown in figure 4.35, then prior to the ultimate load, a shear crack was developed and caused the slab to collapse in shear (see figure 4.36). Cracking load was about 8% of the ultimate load at failure. Figure 4.37 shows a linear load deflection relationship before cracking, then another linear relationship after cracking with a lower bending rigidity $EI$. Figure 4.38 shows a comparison between the experimental and theoretical deflected shape of the model at a total applied load of 133.50 kN and using $l_{cr}$ as the bending rigidity of the cross section, the difference was found to be within 3% for the average deflected shape of the slab model.
4.11 Slab Model #10

Slab model #10 is identical to slab model #9, except the replacement of steel reinforcement by NEFMAC grids. The shear load carrying capacity of the model using eq.(2.19) is 25.7 kN, and using eq.(2.28) is 24 kN, while the load to cause tension failure is 51.5 kN, therefore, it is expected that the model will fail due to shear. Cracks started as a tensile crack at the location of the applied load as shown in figure 4.39, then a shear crack developed in the shear span of the slab and propagated to reach the top fibre of the slab at the location of the applied load to cause the model to fail in shear at an ultimate total applied load of 84.5 kN (see figure 4.40). The cracking load was about 23% of the ultimate load at failure of the specimen. Figure 4.41 shows the load deflection curve for the slab model; note the linear relationship before cracking, followed by another linear relationship representing the effective bending rigidity $E_s I_s$ of the slab after cracking. Figure 4.42 shows a comparison between the experimental and theoretical deflected shape of the model at a total applied load of 48.95 kN and using $I_r$ as the bending rigidity of the cross section, the difference was found to be within 15% for the average deflected shape of the slab model. The shear load at failure is 33.3 kN, therefore, the different shear load carrying capacity formulas used for steel reinforcing bars could be used effectively in slabs reinforced with polyester PSI Fiberbars with no reduction factor needed.

4.12 Slab Model #11

Slab model #11 is identical to slab model #9, except the replacement of steel reinforcing bars by 4-13M polyester PSI Fiberbars. The shear load carrying capacity of
the model using eq.(2.19) is 31.2 kN, and using eq.(2.28) is 31.4 kN, while the load to
cause tension failure is 65.6 kN, therefore, it is expected that the model will fail due to
shear. Cracks started as tensile cracks at the location of maximum bending moment
(figure 4.43), then a major shear crack was developed and led to the failure of the slab
due to shear at an ultimate load of 77.4 kN(figure 4.44). The cracking load was about
11% of the ultimate load at failure of the specimen. Figure 4.45 shows the load deflection
curve for the slab model, note the linear relationship before cracking, followed by another
linear relationship representing the bending rigidity EI of the slab after cracking. Figure
4.46 shows a comparison between the experimental and theoretical deflected shape of the
model at a total applied load of 68.5 kN and using l_e as the bending rigidity of the cross
section, the difference was found to be within 16% for the average deflected shape of the
slab model.

The shear load at failure is 38.7 kN, therefore, the different shear load carrying capacity
formulas used for steel reinforcing bars could be used effectively in slabs reinforced with
polyester PS1 Fiberbars with no reduction factor needed.

4.13 Slab Model #12

Slab model #12 was identical to slab model #11 to ensure a good assessment on
the use of polyester PS1 Fiberbars as reinforcing bars in concrete slabs. Crack pattern,
ultimate load at failure, and failure criteria were all similar to those of slab model #11 as
shown in figure 4.47. Figure 4.48 shows the load deflection curve for the slab model; note
the linear relationship before cracking, then followed by another linear relationship
representing the bending rigidity EI of the slab after cracking. Figure 4.49 shows a comparison between the experimental and theoretical deflected shape of the model at a total applied load of 63.6 kN and using $I_c$ as the bending rigidity of the cross section; the difference was found to be about 18% for the average deflected shape of the slab model. The shear load at failure is 38.7 kN same as slab model #11; therefore, the different shear load carrying capacity formulas used for steel reinforcing bars could be used effectively in slabs reinforced with polyester PSI Fiberbars.

4.14 Observations and Conclusions:

4.14.1 Effective Moment of Inertia

In this section, a study is conducted to compare the effective moment of inertia when using GFRP to the effective moment of inertia when steel reinforcing bars are used. Figure 4.50 shows that the effective moment of inertia $I_e$ for arch model #2 reinforced with NEFMAC grids is about 25% of $I_e$ for arch model #1 reinforced with steel rebars. Figure 4.51 shows that $I_e$ for arch model #4 reinforced with NEFMAC grids is about 27% of $I_e$ for arch model #3 reinforced with steel rebars. Figure 4.63 shows that $I_e$ for slab model #10 reinforced with NEFMAC grids is about 25% of $I_e$ for slab model #9 reinforced with steel rebars. Figure 4.51 shows that the effective moment of inertia $I_e$ for arch model #5 reinforced with polyester PSI Fiberbars is about 70% of $I_e$ for arch model #3 reinforced with steel rebars. Figure 4.63 shows that $I_e$ for slab model #11 or slab model #12 reinforced with polyester PSI Fiberbars is about 33% of $I_e$ for slab model #9 reinforced with steel rebars. It can be concluded that the effective moment of inertia for
A concrete member reinforced with NEFMAC grids is about 25% of the same member reinforced with steel bars, while the effective moment of inertia for a curved concrete member reinforced with polyester PSI Fiberbars is about 70% of the same member reinforced with steel rebars and a slab reinforced with Polyester bars is about 33% of the same slab reinforced with steel bars. For curved members reinforced with steel bars, the value of the exponent $\alpha$ in eq. (2.14) is 1.60, and for curved members reinforced with Polyester PSI Fiberbars, the exponent $\alpha$ is 2.6, while it is 6.3 for curved members reinforced with NEFMAC grids. Figure 4.55 to figure 4.58 show a comparison between arch model #6 reinforced with NEFMAC grids and arch model #7 reinforced with NEFMAC grids and tied at the base, the effective moment of inertia $I_e$ for arch model #6 is about 20% of the effective moment of inertia $I_e$ for arch model #7, i.e. the presence of the axial compression in the arch model reinforced with NEFMAC grids increased the effective moment of inertia $I_e$.

Figure 4.59 to figure 4.62 show a comparison between arch model #5 reinforced with polyester PSI Fiberbars and arch model #8 reinforced with polyester PSI Fiberbars and tied at the base, the effective moment of inertia $I_e$ for arch model #5 is about 30% of the effective moment of inertia $I_e$ for arch model #8, the presence of the axial compression in the arch model reinforced with polyester PSI Fiberbars increased the effective moment of inertia $I_e$. 

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4.14.2 Behaviour of Arch Models in Radial Tension

As outlined in sections 4.2 and 4.3, the failure mode for both arch models #1 and #2 is in radial tension, and according to equations (2.29), (2.30), and (2.31), the radial tension carrying capacity of a reinforced concrete curved member does not depend on the type of material used in the reinforcement whether steel or fibre reinforced plastics (FRP). This was tested by recording the ultimate load at failure for both arch models, the ultimate load at failure for arch model #1 reinforced with steel was 133.5 kN, while the ultimate load at failure of arch model #2 reinforced with NEFMAC grids was 137.8 kN. This means that the equations predicting the radial tension capacity of a concrete curved member reinforced with steel bars (2.29), (2.30), and (2.31) need no modifications when used in predicting the radial tension carrying capacity in a concrete curved member reinforced with FRP.

4.14.3 Behaviour of the Arch Models in Flexure

The failure mode for three of the eight arch models was in tension, first, arch model #3 reinforced with steel bars, then arch model #4 reinforced with NEFMAC grids, and arch model #6 which is identical to arch model #4. The analysis for the different arch models were outlined earlier in sections 4.4, 4.5, and 4.7, it was expected that all arch models #3, #4, and #6 have the same flexural capacity based on the information provided by the manufacturer that the NEFMAC grids provided are equivalent to the use of 10M steel reinforcing bars spaced at 100 mm centre to centre. The ultimate moment at failure for arch model #3 is 30.3 kN.m, while it is 20.45 kN.m for arch model #4, and 20.35 for
arch model #6. Therefore, Both arch models #4 and #6 failed at an experimental load 23% lower than their analytically obtained load carrying capacity as well as 30% lower the ultimate at failure for arch model #3. According to eq.(2.26), the material used in reinforcing the concrete members does not influence the ultimate moment acting on the cross section, therefore, a conclusion can be made that the formula used in predicting the capacity of reinforced concrete culverts with steel bars may require some modifications in order to be used in computing the capacity of reinforced concrete culverts with NEFMAC grids.

4.14.4 Behaviour of Concrete Sections Reinforced with GFRP in Shear

Shear was the failure criteria for arch model #5 reinforced with polyester PSI Fiberbars, arch model #7 reinforced with NEFMAC grids and tied at the base, arch model #8 reinforced with polyester PSI Fiberbars and tied at the base, and all slab models. The different formulas by Heger and MacGrath (eq. 2.19), Zsutty (eq. 2.27), and Rajagopalan and Ferguson (eq. 2.28) which were developed to predict the shear load carrying capacity for concrete members reinforced with steel may be used when using GFRP instead of steel reinforcement with a reduction factor. One can notice that these equations include the reinforcement ratio $\rho$ which accounts for the effect of the longitudinal steel reinforcement on the shear load carrying capacity of the concrete section. The shear load carrying capacity of a reinforced concrete member depends on the depth of cracks which in turn depend on the axial rigidity of the reinforcing bars defined as the area times the modulus of elasticity. The modulus of elasticity of GFRP was about 35% of the modulus
of elasticity of steel reinforcement, therefore, a reduction factor \( K_e = 0.35 \) was applied to the reinforcement ratio \( \rho \) in equations (2.16), (2.20), (2.27), and (2.28).

For arch model #5, at the failure of the specimen, the ultimate shear load \( V_u \) was 60.0 kN, it was expected to be 82.9 kN using eq.(2.16), 70.4 kN using eq.(2.19), 64.6 kN using eq. (2.27), and 69.2 kN using eq.(2.28), therefore, the equations were not applicable to arch model #5. With the reduction factor \( K_e \), the shear load carrying capacity using eq.(2.16) was 81.8 kN, eq.(2.19) was 56.7 kN, using eq.(2.27) was 45.6 kN, and using eq.(2.28) was 47.6 kN, therefore, all equations, with the application of the reduction factor \( K_e = 0.35 \), except eq.(2.16), can represent the shear load carrying capacity for arch model #5. A conclusion can be made that the formulas used for computing the shear load carrying capacity of concrete members reinforced with steel can be used in members reinforced with polyester PSI Fiberbars with a reduction factor \( K_e \) applied to the reinforcement ratio \( \rho \), where

\[
K_e = \frac{(\text{Modulus of elasticity})_{GFRP}}{(\text{Modulus of elasticity})_{STEEL}}
\]

For arch model #7, the shear load carrying capacity was enhanced by the presence of the axial compressive thrust force in the arch model. The effect of axial load on the shear resistance is accounted for in eq.(2.20) by Heger and MacGrath, and eq.(2.16) by the ACI Building code by adjusting \( V_u \), if axial compression is present, \( V_u \) may be increased, while if axial tension is present, \( V_u \) must be reduced. At the failure of the specimen, the
ultimate shear load $V_s$ was 97.45 kN, it was expected to be 87.0 kN using eq.(2.16), 93.45 kN using eq.(2.20). With the reduction factor $K_E$, the shear load carrying capacity using eq.(2.16) was 83.2 kN, eq.(2.20) was 81.1 kN, therefore, the application of the reduction factor $K_E = 0.35$ improved the performance of equations (2.16) and (2.20) to represent the shear load carrying capacity for arch model #7. Finally, a conclusion can be made here that the formulas used to compute the shear load carrying capacity for concrete members reinforced with steel and a combined shear and axial loads are acting on the section can be used effectively when replacing the steel bars by GFRP with the reduction factor $K_E = E_{\text{GFRP}} / E_{\text{steel}}$.

All slab models failed in shear as expected and explained in section 4.10 to section 4.13. A comparison for the fourth group of slabs is shown in figure 4.63, the different formulas to predict the shear load at failure for all slabs underestimate their shear load carrying capacity, therefore, a reduction factor $K_E = 0.35$ is not needed in this case.

Finally, a conclusion can be made, although a reduction factor was not needed to be applied to the different shear formulas for the slab models, it was needed for the formulas representing the arch models failed in shear. Therefore, it is suggested that the reduction factor $K_E$ be applied to the reinforcement ratio $\rho$ that appears in most shear load carrying capacity equations.
TABLE 4.1 SUMMARY OF TEST RESULTS FOR ALL ARCH MODELS AND SLAB MODELS

<table>
<thead>
<tr>
<th>MODEL No.</th>
<th>CRACKING LOAD (kN)</th>
<th>FAILURE LOAD (kN)</th>
<th>TYPE OF FAILURE</th>
</tr>
</thead>
<tbody>
<tr>
<td>ARCH MODEL #1</td>
<td>44.50</td>
<td>133.5</td>
<td>RADIAL TENSION</td>
</tr>
<tr>
<td>ARCH MODEL #2</td>
<td>35.60</td>
<td>137.8</td>
<td>RADIAL TENSION</td>
</tr>
<tr>
<td>ARCH MODEL #3</td>
<td>17.80</td>
<td>85.0</td>
<td>FLEXURAL</td>
</tr>
<tr>
<td>ARCH MODEL #4</td>
<td>26.70</td>
<td>46.4</td>
<td>FLEXURAL</td>
</tr>
<tr>
<td>ARCH MODEL #5</td>
<td>26.70</td>
<td>122.3</td>
<td>SHEAR</td>
</tr>
<tr>
<td>ARCH MODEL #6</td>
<td>22.25</td>
<td>56.9</td>
<td>FLEXURAL</td>
</tr>
<tr>
<td>MODEL No.</td>
<td>CRACKING LOAD (kN)</td>
<td>FAILURE LOAD (kN)</td>
<td>TYPE OF FAILURE</td>
</tr>
<tr>
<td>------------------</td>
<td>--------------------</td>
<td>-------------------</td>
<td>-----------------</td>
</tr>
<tr>
<td>ARCH MODEL #7</td>
<td>22.25</td>
<td>304.4</td>
<td>SHEAR</td>
</tr>
<tr>
<td>ARCH MODEL #8</td>
<td>44.50</td>
<td>322.6</td>
<td>SHEAR</td>
</tr>
<tr>
<td>SLAB MODEL #9</td>
<td>10.25</td>
<td>133.5</td>
<td>SHEAR</td>
</tr>
<tr>
<td>SLAB MODEL #10</td>
<td>11.24</td>
<td>48.95</td>
<td>SHEAR</td>
</tr>
<tr>
<td>SLAB MODEL #11</td>
<td>8.68</td>
<td>77.40</td>
<td>SHEAR</td>
</tr>
<tr>
<td>SLAB MODEL #12</td>
<td>11.25</td>
<td>77.40</td>
<td>SHEAR</td>
</tr>
</tbody>
</table>
FIG. 4-1  RADIAL TENSION FAILURE OF ARCH MODEL (1)
FIG. 4-2 LOAD VS DEFLECTION
ARCH MODEL (1)

LOAD (KN)
0  20  40  60  80  100  120  140

DEFLECTION (mm)
0  2  4  6  8  10  12  14  16  18  20  22  24

DIAL#1  DIAL#2  DIAL#3  DIAL#4
FIG. 4-3 DEFLECTION DISTRIBUTION FOR ARCH MODEL (1)
AT LOAD \( P = 124.6 \) KN
FIG. 4-4 RADIAL TENSION FAILURE OF ARCH MODEL (2)
FIG. 4-6 DEFLECTION DISTRIBUTION FOR ARCH MODEL (2)
AT LOAD \( P = 45.50 \text{ kN} \)
FIG. 4-7 THE CRACK PATTERN FOR ARCH MODEL (3)
FIG. 4-8 ARCH MODEL (3) AT FAILURE
FIG. 4-9 LOAD VS DEFLECTION
ARCH MODEL (3)

LOAD (kN)

DEFLECTION (mm)

- DIAL#1 - DIAL#2 - DIAL#3 - DIAL#4
FIG. 4-10 DEFLECTION DISTRIBUTION FOR ARCH MODEL (3)
AT LOAD P = 44.5 KN
FIG. 4-11 THE CRACK PATTERN FOR ARCH MODEL (4)
FIG. 4-12 ARCH MODEL (4) AT FAILURE
FIG. 4-14  DEFLECTION DISTRIBUTION FOR ARCH MODEL (4)
AT LOAD  P = 35.60 KN
Fig. 4-15 Load vs Deflection Arch Model (5)
FIG. 4-16  DEFLECTION DISTRIBUTION FOR ARCH MODEL (5)
AT LOAD  P = 62.30 KN
FIG. 4-17 THE CRACK PATTERN FOR ARCH MODEL (6)
FIG. 4-18 ARCH MODEL (6) AT FAILURE
FIG. 4-20 DEFLECTION DISTRIBUTION FOR ARCH MODEL (6)
AT LOAD $P = 35.60$ KN
FIG. 4-21 THE CRACK PATTERN FOR ARCH MODEL (7A)
FIG. 4-22 ARCH MODEL (7A) AT FAILURE
FIG. 4-23 LOAD VS DEFLECTION
ARCH MODEL No. 7

LOAD (kN) vs DEFLECTION (mm)

Legend:
- Dial#1
- Dial#2
- Dial#3
- Dial#4

The graph shows the relationship between load (in kN) and deflection (in mm) for four different dial gauges labeled 1, 2, 3, and 4. The deflection points are marked at intervals along the arch model.
FIG. 4-24 DEFLECTION DISTRIBUTION FOR ARCH MODEL (7)
AT LOAD P = 62.30 KN
FIG. 4-27  DEFLECTION DISTRIBUTION FOR ARCH MODEL (7A)
AT LOAD  P = 178.0 KN
FIG. 4-28 LOAD VS STRAIN IN THE TIES FOR ARCH MODEL (7/A)
FIG. 4-29 LOAD VS DEFLECTION
ARCH MODEL (8), DIAL#1
FIG. 4-30 LOAD VS DEFLECTION
ARCH MODEL (8), DIAL #2

LOAD (KN)

DEFLECTION (mm)

0 2 4 6 8 10 12 14 16 18 20 22 24 26

0 50 100 150 200 250 300 350
FIG. 4-31 LOAD VS DEFLECTION ARCH MODEL (8), DIAL #3
FIG. 4-33 DEFLECTION DISTRIBUTION FOR ARCH MODEL (B)
AT LOAD P = 178.0 KN
FIG. 4-34 LOAD VS STRAIN IN THE TIES FOR ARCH MODEL (8)
FIG. 4-35 THE CRACK PATTERN FOR SLAB MODEL (9)
FIG. 4-36  SLAB MODEL (9) AT FAILURE
FIG. 4-37 LOAD VS DEFLECTION
SLAB MODEL (9)
FIG. 4-38 DEFLECTION DISTRIBUTION FOR SLAB MODEL (9)
AT LOAD $P = 133.50$ KN
FIG. 4-39 THE CRACK PATTERN FOR SLAB MODEL (10)
FIG. 4-40  SLAB MODEL (10) AT FAILURE
FIG. 4-42 DEFLECTION DISTRIBUTION FOR SLAB MODEL (10)
AT LOAD P = 49.0 KN
FIG. 4-43 THE CRACK PATTERN FOR SLAB MODEL (11)
FIG. 4-44 SLAB MODEL (11) AT FAILURE
FIG. 4-46 DEFLECTION DISTRIBUTION FOR SLAB MODEL (11)
AT LOAD P = 69.5 KN
FIG. 4-47 THE CRACK PATTERN AND FAILURE FOR SLAB MODEL (12)
FIG. 4-48 LOAD VS DEFLECTION
SLAB MODEL (12)

LOAD (KN)

DEFLECTION (mm)

1 2 3

DIAL#1 ← DIAL#2 ← DIAL#3
FIG. 4-49 DEFLECTION DISTRIBUTION FOR SLAB MODEL (12) AT LOAD P = 63.6 KN
FIG. 4-51 LOAD VS DEFLECTION COMPARISON FOR THE SECOND GROUP AT DIAL #1
FIG. 4-52 LOAD VS DEFLECTION COMPARISON FOR THE SECOND GROUP AT DIAL #2

LOAD (KN)

DEFORMATION (mm)

STEEL  NEFMAC  POLYESTER BARS
FIG. 4-53 LOAD VS. DEFLECTION COMPARISON FOR THE SECOND GROUP AT DIAL #3
FIG. 4-54 LOAD VS DEFLECTION COMPARISON
FOR THE SECOND GROUP AT DIAL #4
FIG. 4-55 EFFECT OF AXIAL LOADS ON THE LOAD VS DEFLECTION FOR NEFMAC, DIAL #1
FIG. 4.56 EFFECT OF AXIAL LOADS ON THE LOAD VS DEFLECTION FOR NEFMAC, DIAL #2

ARCH MODEL (6) → ARCH MODEL (7)
FIG. 4-57 EFFECT OF AXIAL LOADS ON THE LOAD VS DEFORMATION FOR NEFMAC, DIAL #3

ARCH MODEL (G) — ARCH MODEL (T)

LOAD (KN)

DEFLECTION (mm)
FIG. 4-58 EFFECT OF AXIAL LOADS ON THE LOAD VS DEFLECTION FOR NEFMAC, DIAL #4

LOAD (KN)

DEFLECTION (mm)

ARCH MODEL (6)  ARCH MODEL (7)
FIG.4-59 EFFECT OF AXIAL LOADS ON THE LOAD-DEFLECTION FOR POLYESTER BARS, @1

LOAD (kN)

DEFLECTION (mm)

ARCH MODEL (5)  ARCH MODEL (8)
FIG. 4-60 EFFECT OF AXIAL LOADS ON THE LOAD-DEFLECTION FOR POLYESTER BARS, @2

LOAD (kN)

DEFLECTION (mm)

ARCH MODEL (5) — ARCH MODEL (8)
FIG. 4-61 EFFECT OF AXIAL LOADS ON THE LOAD-DEFLECTION FOR POLYESTER BARS, $\odot 3$
FIG. 4.62 EFFECT OF AXIAL LOADS ON THE LOAD-DEFLECTION FOR POLYESTER BARS, @4
CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions:

The main objective of the present thesis was to investigate the behaviour of concrete culverts reinforced with Glass Fibre Reinforced Plastics (GFRP) and examine the validity of the design formulas used in concrete culverts reinforced with steel reinforcing bars to be used in concrete culverts reinforced with GFRP. The experimental study was carried out by testing nine arch models and four beam models under two concentrated line loads. The following conclusions are based on the limited number of tests of the present study;

1. The radial tension capacity of a concrete culvert reinforced with GFRP can be computed using the formulas used in predicting the radial tension capacity of concrete culverts reinforced with steel bars.

2. The formulas used to calculate the flexural capacity of concrete culverts reinforced with steel may require some modifications in order to be used in computing the flexural capacity of concrete culverts reinforced with GFRP.
3. The formulas used in predicting the shear load carrying capacity for concrete culverts reinforced with steel can be used in concrete culverts reinforced with GFRP with a reduction factor $K_E \left( = \frac{E_{GFRP}}{E_{steel}} \right)$ applied to the reinforcement ratio $\rho$ of the GFRP.

4. The reduction factor $K_E$ is not needed to be applied to the reinforcement ratio $\rho$ of the GFRP in the shear load carrying capacity formulas when high axial component is present in the reinforced concrete culvert walls (arch models #7 and #8) as well as the case of beams reinforced with GFRP.

5. The effective bending rigidity $EI_e$ of concrete arch models and slab models reinforced with NEFMAC grids is about 25% of the effective bending rigidity $EI_e$ of concrete arch models and slab models, respectively, of same bending moment carrying capacity and reinforced with steel bars. The effective bending rigidity $EI_e$ of concrete beam models reinforced with polyester PSI Fiberbars is about 33% of the effective bending rigidity $EI_e$ of concrete beam models reinforced with steel bars. The value of the exponent $\alpha$ used in computing $I_e$ for curved members reinforced with steel bars is 1.6, and for curved members reinforced with NEFMAC grids is 6.3, while it is 2.6 for curved members reinforced with Polyester PSI Fiberbars.
5.2 Recommendations for Future Research:

The present study outlined a limited number of tests which show the validity of using GFRP in reinforced concrete culverts. However, more experimental and analytical work is needed to identify the limits and magnitude of the reduction factor $K_e$ and the exponent $\alpha$ used in computing the effective moment of inertia $I_e$.

Also, more research work is needed to identify the modifications required in order to use the flexural capacity formula when replacing steel bars with GFRP.
APPENDIX A

CONCRETE MIX DESIGN
A.1.1 General

The concrete mix was designed based on the guide lines presented in the ACI manual of concrete practice (ACI, 1989)

A.1.2 Materials

<table>
<thead>
<tr>
<th>Component</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>High Early Strength (type 30)</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
<td>Dry mass = 1600 kg/m³ (99.9 lb/ft³)</td>
</tr>
<tr>
<td></td>
<td>Moisture content = 2 %</td>
</tr>
<tr>
<td></td>
<td>Maximum size = 9.5 mm (3/8 in.)</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>Moisture content = 4 %</td>
</tr>
<tr>
<td></td>
<td>Fineness modulus (FM) = 2.56</td>
</tr>
<tr>
<td>Water</td>
<td>Natural tap water</td>
</tr>
</tbody>
</table>

A.1.3 Desired Characteristics

- Compressive strength ($f'_c$) = 30 MPa (4360 psi)
- Slump = 3 inch (75 mm)
- Air content = 0 %

A.1.4 Design

Step (1) - Choice of Slump Value:

- Maximum slump = 4 inch (102 mm), Minimum slump = 2 inch (50 mm)
- Slump value = 3 inch (75 mm)
Step (2) - Maximum Size of Aggregate:

Maximum size = 3/8 in. (10 mm), to produce highest strength at a given water cement ratio.

Step (3) - Estimation of Mixing Water Content:

According to the "ACI Manual of Concrete Practice 1989", and using table 5.3.3, for slump = 3-4 in., and nominal maximum size of aggregate = 3/8 in.

\[ W = 228 \, \text{kg/m}^3 \, (385 \, \text{lb/yd}^3) \]

Step (4) - Selection of Water-Cement ratio:

According to table 5.3.4(a). W/C = 0.54

Step (5) - Calculation of Cement Content:

Cement = \( \frac{228}{0.56} = 422 \, \text{kg/m}^3 \, (712 \, \text{lb/yd}^3) \)

Step (6) - Estimation of Coarse Aggregate Content:

Using table 5.3.6, for fineness modulus of 2.56 and maximum size of 3/8 in (10 mm), the coarse aggregate content = 0.484. As the dry mass of coarse aggregate = 1600 kg/m\(^3\) (2700 lb/yd\(^3\)), the dry weight of coarse aggregate = 0.484 x 1600 = 775 kg/m\(^3\) (1308 lb/yd\(^3\)).

Step (7) - Estimation of fine aggregate content:

Using the volume computation approach

Water \( = \frac{228}{(1 \times 1000)} = 0.228 \, \text{m}^3 \)

Cement \( = \frac{422}{(3.15 \times 1000)} = 0.134 \, \text{m}^3 \)

Coarse aggregate \( = \frac{775}{(2.68 \times 1000)} = 0.289 \, \text{m}^3 \)

Total \( = 0.651 \, \text{m}^3 \)
Fine aggregate  =  1 - 0.651 = 0.349 m³

Dry fine aggregate  =  0.349x1000x2.64 = 920 kg

The weight of fresh concrete = 228+422+775+920 = 2345 kg/m³

Step (8) - The Final Estimated Batch:

(a) Water  =  228 kg/m³ (14.3 lb/ft³)
(b) Cement  =  422 kg/m³ (26.3 lb/ft³)
(c) Coarse Aggregate  =  775 kg/m³ (48.4 lb/ft³)
(d) Fine Aggregate  =  920 kg/m³ (57.5 lb/ft³)

Step (9) - Calculation of the Total Concrete Volume:

The total volume of concrete needed for the first group of arch models and two 6 inch (152.4 mm) by 12 inch (304.8 mm) test cylinders:

Volume of arch model  =  0.143 m³ (5.05 ft³)
Volume of cylinders  =  0.011 m³ (0.388 ft³)
Total volume  =  0.143 + 0.011 = 0.154 m³ (5.438 ft³)

For the second & third group of arch models

Volume of arch model  =  0.348 m³ (12.29 ft³)
Volume of four cylinders  =  0.022 m³ (0.777 ft³)
Total volume  =  0.348 + 0.022 = 0.370 m³ (13.07 ft³)

For the fourth group of slab models

Volume of slab model  =  0.068 m³ (2.40 ft³)
Volume of two cylinders  =  0.011 m³ (0.388 ft³)
Total volume  =  0.068 + 0.011 = 0.079 m³ (2.77 ft³)
Step (10) - Final Batch for the Needed Volume:

For the First Group of arch models

Water = 0.154 * 228 = 35.11 kg (77.43 lb)

Cement = 0.154 * 422 = 64.99 kg (143.33 lb)

Coarse aggregate = 0.154 * 775 = 119.35 kg (263.21 lb)

Fine aggregate = 0.154 * 920 = 141.68 kg (312.45 lb)

For the second & third group of arch models

Water = 0.37 * 228 = 84.36 kg (186.05 lb)

Cement = 0.37 * 422 = 156.14 kg (344.34 lb)

Coarse aggregate = 0.37 * 775 = 286.75 kg (632.39 lb)

Fine aggregate = 0.37 * 920 = 340.4 kg (750.71 lb)

For the fourth group of slab models

Water = 0.079 * 228 = 18.01 kg (39.72 lb)

Cement = 0.079 * 422 = 33.34 kg (73.53 lb)

Coarse aggregate = 0.079 * 775 = 61.22 kg (135.01 lb)

Fine aggregate = 0.079 * 920 = 72.68 kg (160.29 lb)

Final Ratios:

Water/Cement ratio = 0.54

Aggregate/Cement ratio = 4.02

Fine Aggregate/Coarse Aggregate = 1.19
APPENDIX B

SAMPLE CALCULATIONS
Arch Model (1)

Reinforcement:
7 - 10 M steel bars
Two cages of reinforcement.

Cross section:
b = 700 mm, d = 120 mm, t = 150 mm

Concrete properties:
f'_c = 31.6 MPa, E_c = 26.6 GPa

Calculations:

\[ \rho = \frac{700}{700 \times 120} = 0.0083 \]

\[ a = Kd = \frac{A_s f_y}{0.85 f'_c \cdot b} = \frac{700 \times 414}{0.85 \times 31.6 \times 760} = 15.4 \text{ mm} \]

\[ n = \frac{E_s}{E_c} = \frac{200}{26.6} = 7 \]

\[ I_{cr} = \frac{b (kd)^3}{3} + n A_s \left( d - kd \right)^2 + n A_s' \left( d' - kd \right)^2 \]
\[ = \frac{700(18.6)^3}{3} + 7 \times 700(120 - 18.6)^2 + 7 \times 700(25 - 18.6)^2 \]
\[ = 1,501,466 + 50,381,604 + 200,704 = 52,083,774 \text{ mm}^4 \]

\[ A = 700 \times 18.6 = 13,020 \text{ mm}^2 \]
Flexural Capacity:

Compression failure:

\[
\rho_b = 0.85 \beta_1 \frac{f_c'}{f_y} \frac{\varepsilon_c}{\varepsilon_c} \frac{E_s}{E_s + f_y} \\
= 0.85 \times 0.85 \times \frac{31.6}{414} \times 0.003 \times \frac{200000}{0.003 \times 200000 + 414} \\
= 0.032 > 0.0083
\]

Therefore, a compression failure is not likely to occur.

Tension failure: (neglecting the force in the top steel reinforcement)

\[
M_r = A_s f_y \left( d - \frac{a}{2} \right) \\
= 700 \times 414 \left( 120 - \frac{18.6}{2} \right) \times 10^{-6} \\
= 32.5 \text{ kN.m}
\]

Shear capacity:

Using eq.(2.20) by Heger and MacGrath

\[
V = 0.083 \times b \times d \sqrt{f_c'} \left( 1.1 + 63 \times \rho \right) \frac{F_d F_N}{F_c}
\]

\[
F_N = 1 \\
F_d = 0.8 + \frac{41}{d} = 0.8 + \frac{41}{120} = 1.14 \\
F_c = 1 + \frac{d}{2r} = 1 + \frac{120}{2 \times 615} = 1.1
\]

\[
V = 0.083 \times (700) \times (120) \sqrt{31.6} \left( 1.1 + 63 \times 0.0083 \right) \frac{1.14}{1.1} \times 10^{-3} \\
= 66.2 \text{ kN}
\]

Using eq.(2.27) by Zsitty
\[ V = 60 \frac{b}{a} \left( f_c, \rho \frac{d}{a} \right)^\frac{1}{3} \]
\[ = 60 \times 27.55 \times 4.7 \left( 4580 \times 0.0084 \times \frac{4.7}{12.4} \right)^\frac{1}{3} \times \frac{4.45}{10^3} \]
\[ = 83.8 \text{ kN} \]

Using eq.(2.28) by Rajagopalan and Ferguson

\[ V = (0.8 + 100 \rho) b d \sqrt{f_c} \]
\[ = (0.8 + 100 \times 0.0084) \times 27.55 \times 4.7 \times \sqrt{4580} \times \frac{4.45}{10^3} \]
\[ = 63.95 \text{ kN} \]

The shear load carrying capacity of the model is 63.95 kN.

Radial Tension Capacity:
According to ACI 318-89, clause 7-8.7.1.4

\[ M_r = K b r_s \left( d - \frac{a}{2} \right) \sqrt{f_c} \]
\[ = 0.111 \times 700 \times 540 \times \left( 120 - \frac{18.6}{2} \right) \sqrt{31.6} \]
\[ = 26.1 \text{ kN.m} < 32.5 \text{ kN.m} \]

Thus, the radial tension failure criterion is more critical than the tension failure criterion.

Reaction at support due to the radial tension failure = 26.1 / 0.325 = 80.3 kN
Shear component due to the radial tension failure = 80.3 \sin 30^\circ = 40.15 \text{ kN} < 63.95 \text{ kN}

Therefore radial tension failure is likely to govern.
Arch Model (2)

\[ d = 120 \text{ mm} \]
\[ A_g = 5 \times 78.8 = 394 \text{ mm}^2 \]
\[ \rho = \frac{(394)}{(700 \times 120)} = 0.0047 \]
\[ f_u = 700 \text{ MPa} \]

\[ a = \frac{A_s f_y}{0.85 f_c' b} = \frac{394 \times 700}{0.85 \times 30 \times 700} = 15.45 \text{ mm} \]

**Flexural Capacity**

Compression failure:

\[ \rho_b = 0.85 \beta_1 \frac{f_c'}{f_y} \frac{E_s}{E_c + f_y} \]
\[ = 0.85 \times 0.85 \times \frac{40}{700} \times 0.003 \times \frac{70000}{0.003 \times 70000 + 700} \]
\[ = 0.0095 > 0.0047 \]

Therefore, a compression failure is not likely to occur.

Tension failure: (neglecting the force in the top reinforcement)

\[ M_r = A_s f_y (d - \frac{a}{2}) \]
\[ = 394 \times 700 \left(120 - \frac{15.45}{2}\right) \times 10^{-6} \]
\[ = 31.0 \text{ kN.m} \]
Shear load carrying capacity

Using eq.(2.20) by Heger and MacGrath

\[ V = 0.083 \, b \, d \, \sqrt{f_c'} \, (1.1 + 63 \, \rho) \frac{F_d F_N}{F_c} \]

\[ F_N = 1 \]

\[ F_d = 0.8 + \frac{41}{d} = 0.8 + \frac{41}{120} = 1.14 \]

\[ F_c = 1 + \frac{d}{2r} = 1 + \frac{120}{2 \times 615} = 1.1 \]

\[ V = 0.083 \times (700) \times (120) \times \sqrt{40} \times (1.1 + 63 \times (0.0047) \times \frac{1.14}{1.1}) \]

\[ = 64100 \, N = 64.1 \, kN \]

Using eq.(2.27) by Zsutty

\[ V = 60 \, b \, d \, (f_c' \rho \frac{d}{a})^{\frac{1}{3}} \]

\[ = 60 \times 27.55 \times 4.7 \times (5800 \times 0.0047 \times \frac{4.7}{12.4})^{\frac{1}{3}} \times \frac{4.45}{10^3} \]

\[ = 74.7 \, kN \]

Using eq.(2.28) by Rajagopalan and Ferguson

\[ V = (0.8 + 100 \, \rho) \, b \, d \, \sqrt{f_c'} \]

\[ = (0.8 + 100 \times 0.0047) \times 27.55 \times 4.7 \times \sqrt{5800} \times \frac{4.45}{10^3} \]

\[ = 55.7 \, kN \]

The shear load carrying capacity of the model is 55.7 kN.

Radial Tension Capacity:

According to ACI 318-89, clause 7-8.7.1.4

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\[ M_r = K b r_s \left( d - \frac{a}{2} \right) \sqrt{f'_{c}} \]
\[ = 0.111 \times 700 \times 540 \times \left( 120 - \frac{15.45}{2} \right) \frac{\sqrt{40}}{10^6} \]
\[ = 29.8 \text{ kN.m} < 31.0 \text{ kN.m} \]

Thus, the radial tension failure criterion is more critical than the tension failure criterion.

Reaction at support due to the radial tension failure = \(29.8 / 0.325 = 91.7 \text{ kN}\)

Shear component due to the radial tension failure = \(91.7 \sin 30^\circ = 45.85 \text{ kN}\)
\(< 55.7 \text{ kN}\)

Therefore, radial tension failure is likely to govern.

**Arch Model (3)**

- \(b = 700 \text{ mm}\)
- \(d = 120 \text{ mm}\)
- \(f'_{c} = 46.6 \text{ MPa}\)
- \(\rho = 0.0083\)

\[ a = \frac{A_s f_y}{0.85 f'_{c} b} = \frac{700 \times 414}{0.85 \times 46.6 \times 700} = 10.5 \text{ mm} \]

**Flexural Capacity** (neglecting the force in the top reinforcement)

\[ M_r = A_s f_y \left( d - \frac{a}{2} \right) \]
\[ = 700 \times 414 \left( 120 - \frac{10.5}{2} \right) \times 10^{-6} \]
\[ = 33.3 \text{ kN.m} \]
Shear Capacity

\[ V = 0.083 \ b \ d \ \sqrt{f'_c} \ (1.1 + 63 \ p) \]
\[ = 0.083 \ (700) \ (120) \ \sqrt{46.6} \ (1.1 + 63 \times 0.0083) \]
\[ = 77550 \ \text{N} = 77.55 \ \text{kN} \]

\[ M_n = \frac{V}{\cos 11.5^\circ} \cdot \text{shear span} \]
\[ = \frac{77.55}{0.98} \cdot 0.715 = 56.6 \ \text{kN.m} \]

Radial Tension Capacity

\[ M_r = K \ b \ r_s \ (d - \frac{a}{2}) \sqrt{f'_c} \]
\[ = 0.111 \times 700 \times 1000 \times (120 - \frac{10.5}{2}) \sqrt{46.6} \]
\[ = 60.8 \ \text{KN.m} \]

Therefore, tension failure is likely to govern.

At mid span of the arch model

\[ P = 40 \ \text{kN} \]
\[ \Delta_{\text{exp}} = 2.8 \ \text{mm} \]
\[ \Delta_{\text{theoretical}} = \Delta_{cr} = 6 \ \text{mm} \]
\[ I_{cr} = 41,830,000 \ \text{mm}^4 \ (d = 100 \ \text{mm due to the improper placement of the steel bars}) \]

Therefore, \[ I_s = I_{cr} \cdot \frac{\Delta_{cr}}{\Delta_{exp}} = 41,830,000 \times 6 / 2.8 = 89,635,700 \ \text{mm}^4 \]
\[ M_{cr} / M = 17.8 / 40 = 0.445 \]

Using eq.(2.2), the transformed moment of inertia for the uncracked section

\[ I_T = 217,875,000 \ \text{mm}^4 \]

Using eq.(2.14) for the effective moment of inertia

\[ 89,635,700 = (0.445)^a \times 217,875,000 + [1 - (0.445)^a] \times 41,830,000 \]

Using the trial and error method \[ \alpha = 1.60 \]
Arch Model (4)

\[ b = 700 \text{ mm} \]
\[ d = 120 \text{ mm} \]
\[ f'_c = 42.3 \text{ MPa} \]
\[ \rho = 0.0047 \]
\[ A_s = 394 \text{ mm}^2 \]

\[ a = \frac{A_s f_y}{0.85 f'_c \cdot b} = \frac{394 \times 600}{0.85 \times 42.3 \times 700} = 9.4 \text{ mm} \]

**Flexural Capacity** (neglecting the force in the top reinforcement)

\[ M_r = A_s f_y (d - \frac{a}{2}) \]
\[ = 394 \times 600 (120 - \frac{9.4}{2}) \times 10^{-6} \]
\[ = 27.3 \text{ kN.m} \]

**Shear Capacity**

\[ V = 0.083 b d \sqrt{f'_c} (1.1 + 63 \rho) \]
\[ = 0.083 (700)(120) \sqrt{42.3} (1.1 + 63 \times 0.0047) \]
\[ = 63500 \times 63.5 \text{ kN} \]

\[ M_s = \frac{V}{\cos11.5^\circ} \cdot \text{shear span} \]
\[ = \frac{63.5}{0.98} \times 0.715 = 46.3 \text{ kN.m} \]
Radial Tension Capacity

\[ M_r = K b r_s \left( d - \frac{a}{2} \right) f'_c \]
\[ = 0.111 \times 700 \times 1000 \times \left( 120 - \frac{10.95}{2} \right) \sqrt{42.3} \]
\[ = 57.8 \text{ kN.m} \]

Tension failure is likely to govern.

At mid span of the arch model
\[ P = 35.60 \text{ kN} \]
\[ \Delta_{exp} = 13.5 \text{ mm} \]
\[ \Delta_{theoretical} = \Delta_{cr} = 26.1 \text{ mm} \]
\[ I_{cr} = 700 \left( 10.96 \right)^3 / 3 + 2 \times 394 \times (120 - [10.96]/2)^2 + 2 \times 394 \times (25 - 10.96)^2 \]
\[ = 10,566,615 \text{ mm}^4 \]

Therefore, \( I_e = I_{cr} \cdot \Delta_{cr} / \Delta_{exp} = 10,566,615 \times 26.1 / 13.5 = 20,428,790 \text{ mm}^4 \)
\[ M_{cr} / M = 22.25 / 35.60 = 0.625 \]

Using eq.(2.2), the transformed moment of inertia for the uncracked section
\[ I_T = 198,845,000 \text{ mm}^4 \]

Using eq.(2.14) for the effective moment of inertia
\[ 20,428,790 = (0.625)^a \times 198,845,000 + [1 - (0.625)^a] \times 10,566,615 \]

Using the trial and error method
\[ \alpha = 6.30 \]
Arch Model (5)

\[ A_s = 7 \times 88.6 = 620 \text{ mm}^2 \]

\[ b = 700 \text{ mm} \]

\[ d = 120 \text{ mm} \]

\[ \rho = \frac{620}{700 \times 120} = 0.0074 \]

\[ a = \frac{A_s f_y}{85 f_c b} = \frac{620 \times 600}{0.85 \times 41.2 \times 700} = 15.2 \text{ mm} \]

Flexural Capacity (neglecting the force in the top reinforcement)

\[ M_r = A_s f_y \left( d - \frac{a}{2} \right) \]

\[ = 620 \times 600 \left( 120 - \frac{15.2}{2} \right) \times 10^{-6} \]

\[ = 41.8 \text{ kN.m} \]

Radial Tension Capacity

\[ M_r = K b r_s \left( d - \frac{a}{2} \right) \sqrt{f_c} \]

\[ = 0.111 \times 700 \times 1000 \times \left( 120 - \frac{15.2}{2} \right) \sqrt{41.2} \]

\[ = 56.1 \text{ kN.m} \]

Shear Capacity

Using eq.(2.20) by Heger and MacGrath

\[ V = 0.083 b d \sqrt{f_c} \left( 1.1 + 63 \rho \right) \]

\[ = 0.083 \times (700) \times (120) \times \sqrt{41.2} \times (1.1 + 63 \times 0.0074) \]

\[ = 70400 \text{ N} = 70.4 \text{ kN} \]
Using eq.(2.16)

\[ V_c = (1.9 \sqrt{f'_{c}} + 2500 \rho \frac{V_u d}{M_u}) b d \]

\[ = (1.9 \sqrt{6000} + 2500 \times 0.0074 \times \frac{120}{715}) \times \frac{27.55 \times 4.7}{10^3} \]

\[ = 19.45 \text{ kips} = 86.6 \text{ kN} \]

Using eq.(2.27) by Zsutty

\[ V = 60 b d (f'_{c} \rho \frac{d}{a})^{\frac{1}{3}} \]

\[ = 60 \times 27.55 \times 4.7 \times (6000 \times 0.0074 \times \frac{4.7}{28.1})^{\frac{1}{3}} \times \frac{4.45}{10^3} \]

\[ = 67.4 \text{ kN} \]

Using eq.(2.28) by Rajagopalan and Ferguson

\[ V = (0.8 + 100 \rho) b d \sqrt{f'_{c}} \]

\[ = (0.8 + 100 \times 0.0074) \times 27.55 \times 4.7 \times \sqrt{6000} \times \frac{4.45}{10^3} \]

\[ = 68.7 \text{ kN} \]

The shear load carrying capacity is 67.4 kN

\[ M_n = \frac{V}{\cos 11.5^\circ} \text{ shear span} \]

\[ = \frac{67.4}{0.98} \times 0.715 = 49.1 \text{ KN.m} \]

When applying the reduction factor \( K_e = 0.35 \) to the shear resistance equations

Eq.(2.16) \( V_c = 85.4 \text{ kN} \)

Eq.(2.20) \( V_c = 56.7 \text{ kN} \)

Eq.(2.27) \( V_c = 47.6 \text{ kN} \)

Eq.(2.28) \( V_c = 47.5 \text{ kN} \)

The shear load carrying capacity is 47.5 kN \( \Rightarrow M_n = 34.65 \text{ kN.m} \)

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Therefore, shear failure is likely to govern.

At mid span of the arch model

\[ P = 60.0 \text{ kN} \]

\[ \Delta_{exp} = 10.0 \text{ mm} \]

\[ \Delta_{theoretical} = \Delta_{cr} = 24.0 \text{ mm} \]

\[ I_{cr} = \frac{700}{3} \left( \frac{(15.2)^3}{3 + 2 \times 620} \times (120 - \frac{[15.2]/2}{2})^2 + 2 \times 620 \times (25 - 15.2)^2 \right) = 16,565,950 \text{ mm}^4 \]

Therefore, \[ I_e = I_{cr} \times \Delta_{cr} / \Delta_{exp} = 16,565,950 \times 24 / 10 = 39,758,300 \text{ mm}^4 \]

\[ M_{cr} / M = 26.7 / 60.0 = 0.445 \]

Using eq.(2.2), the transformed moment of inertia for the uncracked section

\[ I_T = 203,075,000 \text{ mm}^4 \]

Using eq.(2.14) for the effective moment of inertia

\[ 39,758,300 = (0.445)^a \times 203,075,000 + [1 - (0.445)^a] \times 16,565,950 \]

Using the trial and error method

\[ \alpha = 2.60 \]
**Arch Model (7)**

Using eq.(2.19)

\[
\phi V_b = \frac{\phi v_b b d}{F_N F_C}
\]

\[
v_b = (1.1 + 60 \times 0.0047) \sqrt{6000} = 107 \text{ lb/sq.inch}
\]

\[
F_N = 1 - 0.12 \frac{N_u}{V_u} = 1 - 0.12 \frac{39.6}{21.9} = 0.78
\]

\[
F_c = \left(1 - \frac{d}{2r_m}\right) \frac{1}{F_d F_{pv}}
\]

\[
F_d = 0.8 + \frac{1.6}{4.7} = 1.14 < 1.25
\]

\[
F_e = \left(1 - \frac{4.7}{2 \times 42.3}\right) \frac{1}{1.14 \times 1} = 0.83
\]

\[
V_b = \frac{107 \times 4.7 \times 27.55}{0.78 \times 0.83 \times 10^3} = 21.4 \text{ kips} = 95.2 \text{ kN}
\]

Compares to the shear load at failure of the model \(V_u = 97.4 \text{ kN}\)

Using eq.(2.16) by ACI 318-89
\[ V_c = \left( 1.9 \sqrt{f_c} + 2500 \frac{V_u d}{M_m} \right) b d \]

\[ M_m = M_u - N_u \left( \frac{4h - d}{8} \right) \]
\[ = 206.7 - 39.6 \left( \frac{4 \times 6 - 4.7}{8} \right) = 111.2 \text{ kN.m} \]

\[ V_c = \left( 1.9 \sqrt{6000} + 2500 \times 0.0047 \times \frac{21.9 \times 4.7}{111.2} \right) \frac{27.55 \times 4.7}{10^3} \]
\[ = 20.47 \text{ kips} = 91.1 \text{ kN} \]

Using the reduction factor \( K_E = 0.35 \) in eq.(2.16)

\[ V_c = 87.0 \text{ kN} \]

**Arch Model (8)**

Using eq.(2.19)

\[ \phi V_b = \frac{\phi \nu_b b d}{F_N F_C} \]

\[ \nu_b = (1.1 + 60 \times 0.0074) \sqrt{6000} = 120 \text{ lb/sq.inch} \]

\[ F_N = 1 - 0.12 \frac{44.3}{26} = 0.80 \]

\[ F_d = 0.8 + \frac{1.6}{4.7} = 1.14 < 1.25 \]

\[ F_c = \left( 1 - \frac{4.7}{2 \times 42.3} \right) \frac{1}{1.14 \times 1} = 0.83 \]

\[ 150 \text{ mm} \]

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\[ V_b = \frac{120 \times 4.7 \times 27.55}{0.8 \times 0.83 \times 10^3} = 23.4 \text{ kips} = 104.1 \text{ kN} \]

Compares to the shear load at failure of the model \( V_u = 115.7 \text{ kN} \)

Using eq.(2.16) by ACI 318-89

\[ V_c = (1.9 \sqrt{f_c'} + 2500 \rho \frac{V_u d}{M_m}) b d \]

\[ M_m = M_u - N_u \left( \frac{4h - d}{8} \right) \]
\[ = 286.7 - 44.3 \left( \frac{4 \times 6 - 4.7}{8} \right) = 178.7 \text{ kN.m} \]

\[ V_c = (1.9 \sqrt{6000} + 2500 \times 0.0074 \times \frac{26 \times 4.7}{178.7} \frac{27.55 \times 4.7}{10^3} \]
\[ = 20.7 \text{ kips} = 92.1 \text{ kN} \]
Slab Model (9)

\[ b = 350 \text{ mm} \]
\[ d = 120 \text{ mm} \]
\[ A_s = 400 \text{ mm}^2 \]
\[ \rho = \frac{400}{(350 \times 120)} = 0.0095 \]

Flexural Capacity

\[ a = \frac{A_s f_y}{0.85 f'_c b} = \frac{400 \times 414}{0.85 \times 45 \times 350} = 12.4 \text{ mm} \]

\[ M_r = A_s f_y \left( d - \frac{a}{2} \right) \]
\[ = 400 \times 414 \left( 120 - \frac{12.4}{2} \right) \times 10^{-6} \]
\[ = 18.8 \text{ kN.m} \]

Shear Capacity

Eq.(2.20): \[ V = 0.083 \times 350 \times 120 \sqrt{45} \left( 1.1 + 63 \times 0.0095 \right) \times 10^{-3} = 39.9 \text{ kN} \]

Eq.(2.28): \[ V = 0.083 \left( 0.8 + 100 \times 0.0095 \right) \sqrt{45} \times 350 \times 120 \times 10^{-3} = 41.1 \text{ kN} \]

ACI Code: \[ V = 0.083 \times 2 \times 350 \times 120 \sqrt{45} \times 10^{-3} = 46.9 \text{ kN} \]

The moment resistance \( M_n \) when shear failure occurs

\[ M_n = 39.9 \times 0.35 = 13.96 \text{ kN.m} \]

Therefore, a shear failure is likely to govern.
**Slab Model (10)**

\[ b = 350 \text{ mm} \]
\[ d = 120 \text{ mm} \]
\[ A_g = 3 \times 78.8 = 236.4 \text{ mm}^2 \]
\[ \rho = \frac{236.4}{(350 \times 120)} = 0.0056 \]

**Flexural Capacity**

\[ a = \frac{A_z f_y}{0.85 f_c' b} = \frac{236.4 \times 700}{0.85 \times 25.5 \times 350} = 21.8 \text{ mm} \]

\[ M_t = A_z f_y \left( d - \frac{a}{2} \right) \]
\[ = 236.4 \times 700 \left( 120 - \frac{21.8}{2} \right) \times 10^{-6} \]
\[ = 18.05 \text{ kN} \cdot \text{m} \]

**Shear Capacity**

*Eq.(2.20):* \[ V = 0.083 \times 350 \times 120 \sqrt{25.5} (1.1 + 63 \times 0.0056) \times 10^{-3} = 25.7 \text{ kN} \]

*Eq.(2.28):* \[ V = 0.083 (0.8 + 100 \times 0.0056)\sqrt{25.5} \times 350 \times 120 \times 10^{-3} = 24.1 \text{ kN} \]

*ACICode:* \[ V = 0.083 \times 2 \times 350 \times 120 \sqrt{25.5} \times 10^{-3} = 35.3 \text{ kN} \]

The moment resistance \( M_n \) when shear failure occurs

\[ M_n = 24.1 \times 0.35 = 8.41 \text{ kN} \cdot \text{m} \]

Therefore, a shear failure is likely to govern.

The shear load at failure is 33.3 kN, therefore, no reduction factor is needed in this case.
Slab Model (11)

\[ b = 350 \text{ mm} \]
\[ d = 120 \text{ mm} \]
\[ A_s = 4 \times 88.6 = 354.4 \text{ mm}^2 \]
\[ \rho = \frac{(354.4)}{(350 \times 120)} = 0.0084 \]

Flexural Capacity

\[ a = \frac{A_s f_y}{0.85 f_c^* b} = \frac{354.4 \times 600}{0.85 \times 30 \times 350} = 23.8 \text{ mm} \]

\[ M_r = A_s f_y \left( d - \frac{a}{2} \right) \]
\[ = 354.4 \times 600 \left( 120 - \frac{23.8}{2} \right) \times 10^{-6} \]
\[ = 22.98 \text{ kN.m} \]

Shear Capacity

\[ Eq.(2.20): V = 0.083 \times 350 \times 120 \sqrt{30} \left( 1.1 + 63 \times 0.0084 \right) \times 10^{-3} = 31.2 \text{ kN} \]

\[ Eq.(2.28): V = 0.083 \left( 0.8 + 100 \times 0.0084 \right) \sqrt{30} \times 350 \times 120 \times 10^{-3} = 31.4 \text{ kN} \]

\[ ACICode: V = 0.083 \times 2 \times 350 \times 120 \sqrt{30} \times 10^{-3} = 38.3 \text{ kN} \]

The moment resistance \( M_n \) when shear failure occurs

\[ M_n = 31.2 \times 0.35 = 10.92 \text{ kN.m} \]

Therefore, a shear failure is likely to govern.

The shear load at failure is 38.7 kN, therefore, no reduction factor is needed in this case.
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