EXPERIMENTAL INVESTIGATION AND NUMERICAL MODELLING OF INNOVATIVE FRP REINFORCED CONCRETE BRIDGE DECK PANELS

ÉTUDE EXPÉRIMENTALE ET MODÉLISATION NUMÉRIQUE SUR DES PANNEAUX DE PONT EN BÉTON PRÉCONTRAINT ET ARMÉ DE PRF

Mémoire de maîtrise ès sciences appliquées
Spécialité : génie civil

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Abstract

The possibility of designing steel-free reinforced concrete structures offers an interesting approach to overcome deterioration problems associated with the corrosion of steel reinforcement. This research project investigates steel-free precast prestressed concrete bridge deck systems, with the objectives of increasing the service life of new bridge decks and understanding their structural behaviour.

The project was initiated after a major repair of the bridge deck systems on the Jacques-Cartier Bridge in Montreal, which was carried out during summer 2001–02. The final solution for the replacement of the bridge deck was the use of precast prestressed concrete decks with conventional steel tendons and reinforcing bars. The bridge decks were rebuilt because the existing steel tendons and reinforcing bars were severely damaged over the years by corrosion.

Full-scale concrete bridge panel prototypes reinforced with carbon fibre-reinforced polymer (CFRP) prestressing tendons and glass fibre-reinforced polymers (GFRPs) as stirrups and slab reinforcement were constructed and tested up to failure. The results of these prototypes were compared to those of normal steel reinforced and steel prestressed concrete panel specimens. Each prototype was loaded under three different configurations until cracks first appeared to study the service state. It was then loaded at mid-span until failure to study the ultimate state. The behaviour of the panels was characterized by the mid-span and third point deflections and also by the strains obtained from the gauges fixed on the tendons, concrete, reinforcing bars and stirrups.

Based on the testing of four full-scale panels, the thesis presents comparisons on the serviceability, ultimate strength, and failure modes. The experimental results are also compared to the theoretical equations and to the predictions from two finite element programs. The numerical and theoretical results are shown to be in good agreement with the experiments.
Résumé

La possibilité de dimensionner des éléments en béton armé sans acier offre une intéressante alternative pour résoudre les problèmes de détérioration reliés à la corrosion des armatures en acier. Ce projet de recherche porte sur l'étude de poutres de tabliers de pont préfabriquées en béton précontraint sans acier, avec pour objectif d'augmenter la durée de vie utile des nouveaux tabliers de pont et de comprendre leur comportement structural.

Le projet a pris naissance à la suite d'une réfection majeure des tabliers du pont Jacques-Cartier à Montréal effectuée au cours des étés 2001 et 2002. L'utilisation des tabliers préfabriqués en béton précontraint avec des tendons et des barres en acier conventionnel s'est révélé le choix pour la réfection. Toutefois, les tabliers de pont originaux ont dû être reconstruits, car les tendons et les barres en acier actuels ont été sévèrement endommagés par la corrosion au cours des années.

Des poutres en béton précontraint de tendons en polymères renforcés de fibres de carbone (PRFC), et renforcé d'étiers et d'armatures en polymères renforcés de fibres de verre (PRFV), ont été construites et soumises à des essais de laboratoire; les résultats ont été, par la suite, comparés à des poutres en béton renforcé et précontraint d'acier conventionnel. Chaque spécimen a été chargé sous trois configurations jusqu'à l'apparition des premières fissures pour étudier les limites en service; il a ensuite été chargé à mi-portée jusqu'à la rupture afin d'étudier les limites ultimes. Le comportement des spécimens a été analysé par les flèches à mi-portée et aux tiers de la portée ainsi que par les déformations obtenues des jauges installées sur les tendons, le béton, les armatures et les étiers.

Ce mémoire de recherche présente, sous forme de comparaison, les résistances en service et ultime ainsi que les modes de rupture des essais effectués sur quatre poutres pleine grandeur. Les résultats expérimentaux sont comparés à la fois avec les équations théoriques et avec les prédictions obtenues de deux programmes d'éléments finis. Les résultats numériques et théoriques démontrent une bonne similitude avec les valeurs expérimentales.
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Definitions

Anchor system — anchor pieces or group of anchor pieces to lock the stressed tendon in position so that it will retain its stressed condition.

Anchorage — in pretensioning, a device used to maintain the elongation of a tendon during the time interval between stressing and release.

At jacking — at time tendons are put in tension.

CFRP — carbon fibre-reinforced polymer.

Concrete, structural normal density — wet fresh concrete having a density between 2150 and 2500 $kg/m^3$ in accordance with CSA Standards A23.2.

Creep — the inelastic strain of a material under a sustained load over a period of time.

Development length — length of embedded reinforcement required to develop the nominal capacity of a member.

Effective prestressing force — stress remaining in prestressing tendons after all losses have occurred, excluding effects of dead load and superimposed load.

FRP — fibre-reinforced polymers.

GFRP — glass fibre-reinforced polymers.

Hold-down device — device use to maintain the harped prestressing tendon at the right vertical position during the prestressing.

Initial prestressing force — applied force to stretch the prestressing tendons: in the case of prestressing by pretensioning, the predicted initial prestressing force excluded the necessary force to overcome the anchorage slip and the temperature correction.

Jacking force — temporary force exerted by a device that introduces tension into prestressing tendons.

Precast concrete — concrete elements cast and cured in a location other than their final position in service in accordance with CSA Standards A23.4.

Prestressed concrete — concrete in which a minimal 1.50 MPa internal stresses have been initially introduced so that the subsequent stresses resulting from dead load and
superimposed loads are counteracted to a desired degree. This can be by post-tensioning or pretensioning.

**Prestressing** — applying compressive stress to a concrete structural member to increase its strength.

**Pretensioning** — method of prestressing in which the tendons are tensioned before the concrete is placed.

**Reinforced concrete** — concrete containing adequate reinforcement (prestressed or not prestressed) and designed on the assumption that concrete and reinforcement act together in resisting forces.

**Reinforcement** — non-prestressed steel and/or FRP bars set into the concrete.

**Relaxation** — loss of tension with time in a stressed tendon maintained at constant length and temperature.

**Shrinkage** — volume reduction of the concrete during the curing period.

**Slab** — element which the width is at least four times the effective height.

**Spacing** — distance between the reinforcement bars axis, of anchorage pieces, or adjacent prestressing tendons.

**Strength of concrete, specified** — compressive strength of concrete used in the design and evaluated in accordance with CSA Standards A23.2.

**Tendon** — steel and/or CFRP high-strength bar or strand used to impart prestress to a structural component.

**Tensile stiffness** — stiffness effect on a member, caused by the contribution of the uncracked concrete between the cracks.

**Tensile strength of prestressing tendon, specified** — tensile strength or rupture load of prestressing tendon by unit area, as specified in the plans and evaluated in accordance with CSA Standards G279.

**Transfer** — act of transferring stress in prestressing tendons from jacks or the pretensioning bed to the concrete member.

**Transverse shear reinforcement** — reinforcement used to resist shear, torsion or lateral forces in a structural member. It is usually ribbed bars that are bent to form U's, L's or rectangular shapes and are placed at right angles or make an angle with the longitudinal reinforcement. The term *stirrups* is usually applied to lateral reinforcement in flexural members.

**Yield strength** — specified minimum yield strength or yield point of steel reinforcement.
## Notation

- \( A \): area of that part of cross section between flexural tension face and centroid of gross section, \( mm^2 \)
- \( A_g \): gross area of section, \( mm^2 \)
- \( A_{GFRP} \): area of GFRP tension reinforcement, \( mm^2 \)
- \( A'_{GFRP} \): area of GFRP compression reinforcement, \( mm^2 \)
- \( A_{PCFRP} \): area of CFRP prestressing tendons in tension zone, \( mm^2 \)
- \( A_{ps} \): area of steel prestressing tendons in tension zone, \( mm^2 \)
- \( A_s \): area of steel tension reinforcement, \( mm^2 \)
- \( A'_s \): area of steel compression reinforcement, \( mm^2 \)
- \( A_{w, min} \): minimum amount of shear reinforcement, \( mm^2 \)
- \( a \): depth of equivalent rectangular stress block, \( mm \)
- \( b \): width of compression face of member, \( mm \)
- \( b_f \): width of the flange, \( mm \)
- \( b_w \): minimum effective web width within depth \( d \), \( mm \)
- \( C \): compressive force in the equivalent rectangular stress block, \( N \)
- \( C'_{GFRP} \): factored force in the compression GFRP reinforcement, \( N \)
- \( C_{cr} \): final creep coefficient
- \( C_{cr}(t) \): time effect on the creep coefficient
- \( C'_{s} \): factored force in the compression steel reinforcement, \( N \)
- \( c \): distance from extreme compression fibre to neutral axis, \( mm \)
- \( d \): distance from extreme compression fibre to the longitudinal tension reinforcement, but need not be less than 0.8\( h \) for prestressed members, \( mm \)
- \( d_b \): nominal diameter of bar or prestressing tendons, \( mm \)
- \( d_c \): thickness of the concrete cover measured from extreme tension fibre to center of bar, \( mm \)
- \( d_i \): depth of each individual tendon, \( mm \)
\( d_s \) distance from the top slab to the centroid of the bottom transverse FRP bars, \( \text{mm} \)

\( d_c \) distance measured perpendicular to the neutral axis between the resultants of the tensile and compressive forces due to flexion, but need not be taken less than 0.72 \( h \), \( \text{mm} \)

\( E_c \) modulus of elasticity of concrete at 28 days, \( \text{MPa} \)

\( E' \) \( = f'_c / \varepsilon_o \)

\( E_{ci} \) modulus of elasticity of concrete at transfer, \( \text{MPa} \)

\( E_f \) elastic modulus of the fibres, \( \text{MPa} \)

\( E_{GFRP} \) modulus of elasticity of \( GFRP \) reinforcement bars, \( \text{MPa} \)

\( E_{long} \) modulus of elasticity of longitudinal reinforcement, \( \text{MPa} \)

\( E_{PCFRP} \) modulus of elasticity of \( CFRP \) prestressing tendons, \( \text{MPa} \)

\( E_{ps} \) modulus of elasticity of steel prestressing tendons, \( \text{MPa} \)

\( E_s \) modulus of elasticity of steel reinforcement bars, \( \text{MPa} \)

\( E_t \) tangent modulus at zero strain, \( \text{MPa} \)

\( E_{FRP} \) modulus of elasticity of FRP stirrups, \( \text{MPa} \)

\( e \) eccentricity of tendon, \( \text{mm} \)

\( f_c \) concrete stress at level considered, \( \text{MPa} \)

\( f'_c \) specified compressive strength of concrete, \( \text{MPa} \)

\( f'_{ct} \) compressive strength of concrete at day, \( t \), \( \text{MPa} \)

\( f_{ct}(t) \) time function

\( f_{cs} \) concrete stress at the tendons centre of gravity, at a considered cross-section, \( \text{MPa} \)

\( f_{cu} \) modulus of compression rupture of concrete, \( \text{MPa} \)

\( f_{FRP,bend} \) specified tensile strength of the straight portion of the FRP bent stirrup, \( \text{MPa} \)

\( f_{GFRP} \) calculated stress of \( GFRP \) reinforcement bar at specified loads, \( \text{MPa} \)

\( f_{GFRPu} \) ultimate tensile stress of \( GFRP \) reinforcement bar, \( \text{MPa} \)

\( f_h \) stress increase due to harping, \( \text{MPa} \)

\( f_{pc} \) effective prestress at the centroid of the section, \( \text{MPa} \)

\( f_{PCFRP} \) stress of \( CFRP \) prestressing tendons, \( \text{MPa} \)

\( f_{pecFRP} \) effective stress in \( CFRP \) prestressing tendons (after allowance for all prestress losses), \( \text{MPa} \)

\( f_{pcs} \) effective stress in steel prestressing tendons (after allowance for all prestress losses), \( \text{MPa} \)
\( f_{psi} \) initial stress in steel tendon after stressing, \( MPa \)

\( f_{psy} \) yield strength of steel prestressing tendons, \( 0.9 f_{psi} \) for low relaxation steel tendons, \( MPa \)

\( f_{psu} \) ultimate tensile strength of steel prestressing tendons, \( MPa \)

\( f_{pcr}^{FP} \) ultimate tensile strength of CFRP prestressing tendons, \( MPa \)

\( f_r \) modulus of cracking of concrete, \( MPa \)

\( f_{sh}(t) \) time function

\( f_{su} \) specified tensile strength of the steel anchorage or the ultimate tensile strength of steel reinforcement bar, \( MPa \)

\( f_t \) uniaxial cut-off tensile stress, \( MPa \)

\( f_y \) specified yield strength of tension steel reinforcement, \( MPa \)

\( f'_y \) specified yield strength of compression steel reinforcement, \( MPa \)

\( H \) average relative humidity, \( \% \)

\( h \) overall depth or thickness of member, \( mm \)

\( h_f \) depth of compression flange, \( mm \)

\( h_i \) distance from the centroid of the tension reinforcement to the neutral axis, \( mm \)

\( h_x \) distance from the extreme flexural tension surface to the neutral axis, \( mm \)

\( I_{cr} \) transformed moment of inertia of cracked section expresses as the moment of inertia of the equivalent concrete section, \( mm^4 \)

\( I_e \) effective moment of inertia for computation of deflection, \( mm^4 \)

\( I_g \) moment of inertia of gross concrete section about the centroid axis, neglecting reinforcement, \( mm^4 \)

\( K_{crh} \) relative humidity coefficient, \( H \)

\( K_r \) reinforcement ratio coefficient, \( r \)

\( K_{shh} \) relative humidity coefficient, \( H \)

\( K_t \) coefficient considering the age of the concrete at transfer, \( t_{co} \)

\( K_v \) coefficient that consider the volume/area ratio exposed to air, \( v \)

\( k \) ratio \( c/d \)

\( k \) stress reduction coefficient

\( l \) length of span, \( mm \)

\( l_d \) development length, \( mm \)

\( l_e \) effective length of tendon, \( mm \)
$M_a$ maximum moment in a element at which the deflection is being computed, $N\cdot mm$

$M_{cr}$ cracking moment, $N\cdot mm$

$M_{DL}$ moment due to dead load at transfer, $N\cdot mm$

$M_{de}$ decompression moment. Moment when the compressive stress on the tensile face of a prestressed member is zero, $N\cdot mm$

$M_f$ moment due to factored loads, $N\cdot mm$

$M_g$ moment due to girder weight, $N\cdot mm$

$M_{LL}$ moment from the applied load on the panel during the testing, $N\cdot mm$

$M_n$ nominal moment capacity, $N\cdot mm$

$M_p$ total moment due to prestressing, $N\cdot mm$

$n$ fitting curve coefficient

$P_{eff}$ effective prestress force after all loses, $N$

$P_t$ prestressing force at transfer, $N$

$P_j$ jacking force, $N$

$R$ curvature radius of the harping saddle, $mm$

$R_t$ tendon radius, $mm$

$r$ radius of curvature of the bend of an FRP stirrup, $mm$

$s$ spacing of shear or torsion reinforcement measured parallel to the longitudinal axis of the member or, sag of prestressing tendons, $mm$

$T$ tensile force in a group of longitudinal bars, $N$

$t_{co}$ age of concrete, when applying loads or, at the moment the creep effect starts, $days$

$V_{CCFRP}$ nominal shear strength provided by concrete with CFRP flexural reinforcement, calculated according to the simplified method for design for shear and torsion in flexural regions, $N$

$V_{CCFRP}$ nominal shear strength provided by concrete with CFRP flexural reinforcement, calculated according to the general method for design for shear and torsion in flexural regions, $N$

$V_{CGS}$ nominal shear strength provided by concrete with steel flexural reinforcement, calculated according to the general method for design for shear and torsion in flexural regions, $N$
\( V_{cs} \) nominal shear strength provided by concrete with steel flexural reinforcement, calculated according to the simplified method for design for shear and torsion in flexural regions, \( N \)

\( V_{GFRP} \) factored shear resistance provided by \( GFRP \) shear reinforcement, calculated according to the simplified method for design for shear and torsion in flexural regions, \( N \)

\( V_{GFRPg} \) factored shear resistance provided by \( GFRP \) shear reinforcement, calculated according to the general method for design for shear and torsion in flexural regions, \( N \)

\( V_f \) factored shear force at section, \( N \)

\( V_{PCFRP} \) component in the direction of the applied shear of the effective prestressing forces for \( CFRP \) prestressing tendons or for variable depth members, the sum of the components of flexural compression and tension in the direction of the applied shear, positive if resisting applied shear, \( N \)

\( V_{ps} \) component in the direction of the applied shear of the effective prestressing forces for steel prestressing tendons or for variable depth members, the sum of the components of flexural compression and tension in the direction of the applied shear, positive if resisting applied shear, \( N \)

\( V_r \) factored shear resistance, calculated according to the simplified method for design for shear and torsion in flexural regions, \( N \)

\( V_{rg} \) factored shear resistance, calculated according to the general method for design for shear and torsion in flexural regions, \( N \)

\( V_s \) factored shear resistance provided by steel shear reinforcement, calculated according to the simplified method for design for shear and torsion in flexural regions, \( N \)

\( V_{sg} \) factored shear resistance provided by steel shear reinforcement, calculated according to the general method for design for shear and torsion in flexural regions, \( N \)

\( y_{cr} \) distance to tension fibre being considered from centroid of cracked section, \( mm \)

\( y_g \) distance from the extreme compression fibre to the centroid of the gross section, \( mm \)

\( \alpha \) angle between inclined stirrups or bent-up bars and the longitudinal axis of the member, \( ^\circ \)
\( \alpha_1 \) ratio of average stress in rectangular compression block to the specified concrete strength

\( \beta \) angle of inclination of the transverse reinforcement to the longitudinal axis of an element, °

\( \beta_d \) reduction coefficient used in calculating deflection

\( \beta_{l} \) ratio of depth of rectangular compression block to depth to the neutral axis, a/c

\( \Delta_d \) delayed deflection, mm

\( \Delta_g \) deflection due to the girder weight, mm

\( \Delta_L \) deflection from live loads, mm

\( \Delta_p \) deflection due to the eccentricity of the effective prestressing force after losses, mm

\( \Delta_{po} \) instantaneous elastic deflection due to the eccentricity of the prestressing force, mm

\( \Delta_{w}\) total short-term deflections that occur just after the prestressing force is released to the concrete, mm

\( \Delta_{ws}\) deflection from the permanent dead loads, mm

\( \varepsilon_c \) compressive strain in concrete

\( \varepsilon_{cu} \) ultimate compressive strain in concrete

\( \varepsilon_d \) strain used to decompress the concrete

\( \varepsilon_f \) strain available for flexure

\( \varepsilon_{GFRP} \) strain in tension GFRP reinforcement bar

\( \varepsilon_{GFRPu} \) ultimate strain in GFRP reinforcement bar at stress \( f_{GFRPu} \)

\( \varepsilon_i \) elastic shortening of the concrete

\( \varepsilon_o \) concrete strain corresponding to the maximum stress \( f'_c \)

\( \varepsilon_{PCFRP} \) strain in CFRP prestressing tendon at stress \( f_{PCFRP} \)

\( \varepsilon_{PCFRPu} \) ultimate strain in CFRP prestressing tendon at stress \( f_{PCFRPu} \)

\( \varepsilon_{pe,CFRP} \) strain used for the prestressing of CFRP tendon

\( \varepsilon_{pr} \) any loss of strain capacity due to sustained loads

\( \varepsilon_{psy} \) strain in steel prestressing tendon at stress \( f_{psy} \)

\( \varepsilon_{psu} \) ultimate strain in steel prestressing tendon

\( \varepsilon_{sh} \) shrinkage strain or strain hardening point of steel reinforcement

\( \varepsilon_{su} \) ultimate strain of steel reinforcement bar

\( \varepsilon_v \) effective strain of the stirrups
\( \varepsilon_x \)  
longitudinal strain of flexural tension chord of the member, positive when tensile

\( \varepsilon_y \)  
yield strain of tension steel reinforcement

\( \varepsilon_s \)  
principal tensile strain in cracked concrete due to factored load

\( \xi \)  
initial prestressing ratio

\( \theta \)  
angle of inclination of diagonal compressive stresses to the longitudinal axis of the member, \(^\circ\)

\( \lambda \)  
concrete density coefficient (1.0 for normal density concrete)

\( \lambda \)  
material constant

\( \rho \)  
reinforcement ratio

\( \rho_b \)  
balanced reinforcement ratio

\( \rho_s \)  
ratio of the cross-sectional area of the longitudinal FRP reinforcement to the effective cross-sectional area of the beam

\( \rho_{v, FRP} \)  
ratio of the total cross-sectional area of the legs of an FRP stirrup to the product of the width of the beam and the spacing of stirrups

\( \sigma_N \)  
stress in the concrete due to axial loads, \(MPa\)

\( \sigma_o \)  
effective stress of the stirrups, \(MPa\)

\( \nu \)  
Poisson ratio

\( \nu_f \)  
shear stress intensity, \(MPa\)

\( \gamma_c \)  
density of concrete, \(kg/m^3\)

\( \phi_c \)  
resistance factor for concrete

\( \phi_{PCFRP} \)  
resistance factor for carbon-fibre-reinforced polymer (CFRP)

\( \phi_{GFRP} \)  
resistance factor for glass-fibre-reinforced polymer (GFRP)

\( \phi_{ps} \)  
resistance factor for steel prestressing tendons

\( \phi_s \)  
resistance factor for steel reinforcement bars

\( \chi \)  
constant for tensile strain definition

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Chapter 1

Introduction

1.1 Deterioration and Rehabilitation of Structures

For more than 100 years, structural engineers have had the possibility to design bridge structures with reinforced concrete. This method offers several advantages. All kinds of shapes can be cast, they can be easy and fast to fabricate, relatively cheap for construction and can resist all kind of loads (flexural, shear, axial and torsional). These are the reasons why we see many reinforced concrete structures all around the world. Inside the concrete matrix, steel bars are placed to increase the capacity of the components. Steel bars perform well, but they have their limits, in particular in northern climates. The extensive use of de-icing salts on roads during winter can be a major problem for the steel bars. When the combination of salts and humidity infiltrate the micro-cracks and cracks in the concrete, in a short time they initiate the corrosion of the steel bars. The corrosion of steel considerably affects the capacity and the durability of concrete structures reinforced with conventional steel. The most affected structures are bridge structural components and outside car parks, because they are directly attacked by the aggressive agents in the environment. Corrosion problems result in enormous costs. Each year in Quebec, bridges with corroded steel bars cost $100 million in repair. The corrosion also puts in danger the safety of the road users. In March 2003, on Highway 15 in Montreal, a one cubic meter of concrete broke loose from an overpass and crashed on a car during the peak hour traffic. Fortunately, nobody was injured [CHARRON, 2003].

In Canada, the cost of repairing and rehabilitating structures is evaluated at $60 billion which represents over 30,000 bridges that are severely damaged. In the United States, more than 200,000 bridges required an intervention or a reconstruction [MUFTI, 2004]. This
situation represents considerable sums of money. Fortunately, structural engineers now have the possibility to use the new technology of fibre-reinforced polymer (FRP) materials to increase the service life of structures and also to reduce the maintenance and repair costs for future generations.

1.2 Use of FRPs in Concrete Structures

Although the use of FRPs goes back up to the early 1940's, it is only in the 1980's that researchers considered these innovative materials as substitution for steel bars in reinforced concrete structures. The current techniques to improve the durability of steel reinforcements have not led to an adequate and economic solution for the corrosion problem. To solve this problem, FRPs were recognized quite recently as an alternative to conventional steel reinforcement in concrete structures [Newman et al., 2004]. Their high strength, high modulus of elasticity and non-corrosive properties allow them to be used where aggressive agents are present in the environment. Numerous studies, researches and demonstration projects were realized in Canada, in Europe, in Japan and in the United States in the 1990's to understand and demonstrate the advantages of these new materials [Burgoine, 1999; Dolan, 1999; Erki, 1999; Fukuyama, 1999; Karbhari and Seible, 1999].

The field installation of steel bars is not an easy task because of the high density of the material. One of the great advantages of carbon fibre-reinforced polymer (CFRP) and glass fibre-reinforced polymer (GFRP) bars is in their low material density, which allows them to be easily manipulated on the field. Demonstration projects are more and more present in many countries. Some of the North America projects are described in the following.

- In Canada, the new generation of concrete bridges reinforced and prestressed with FRPs was first seen in 1993. Since, a two continuous spans highway bridge of 22.83 m and 19.23 m, respectively, in length was built in Calgary [Rizkalla and Tadros, 1994]. On each span, 13 bulb-T precast prestressed concrete beams were fabricated for the structural components of this bridge. Two types of FRP tendons were used. Four beams were prestressed with 15.2 mm diameter carbon fibre composite cable (CFCC), and two other beams were prestressed with 10 mm diameter CFRP Leadline tendons. The height of each beam is 1.1 m. The six concrete beams prestressed with FRPs are instrumented to collect long-term information on the
behaviour of the structure.

- In Manitoba, a reinforced concrete bridge was built with FRPs in 1997 [RIZKALLA et al., 1998]. This structure, the Taylor Bridge, measures 165 m in length and is divided into five equal spans. Each span consists of eight I shaped precast prestressed concrete beams with a height of 1.829 m. For the overall number of beams, two were prestressed with 10 mm diameter CFRP tendons, and the two others were prestressed with 15.2 mm diameter CFCC tendons. The remaining beams were reinforced and prestressed with conventional steel. The prestressing force applied to both types of FRP represents 60% to 63% of the manufacturer’s specified tensile strength. The four concrete beams with FRP materials are instrumented to follow their long-term behaviour and also to be able to make fast interventions if an abnormal change occurs in the structure.

- The first bridge built with CFRP tendons in the United States was constructed in 1998 [GRACE et al., 2002]. This bridge, the Bridge Street structure, measures 62 m in length. The structural elements consist of four special precast concrete beams in a double-tee (DT) shape reinforced and prestressed with CFRP. Only the stirrups are stainless steel. The beam height is 1.22 m and the stem width is 0.304 m. The total slab width is 2.12 m. The beam spans are respectively 21.314 m, 20.349 m and 21.429 m each. The concrete design compressive strength after 28 days was 52 MPa. For the prestressing, 10×10 mm diameter CFRP Leadline tendons were used with a prestressing force of 91 kN for each tendon. This bridge is also instrumented to monitor the long-term behaviour of this structure.

- In Quebec, the bridge decks of the Wotton and Joffre Bridges were partially reinforced with FRP [EL-SALAKAWY et al., 2002]. Half of the bridge deck of the Wotton Bridge is reinforced with FRP bars and the other half is reinforced with conventional steel bars. The bridge decks were instrumented to compare the behaviour of both types of reinforcement.

These demonstration projects illustrate very well the feasibility of using FRP materials in new bridge structures. However, practical engineers are still reluctant to employ these materials, mainly because they lack proper information on how to use these new innovative materials. It is difficult to implant a new technology, in view of the fact that for more than a century the use of steel is exclusive in reinforced concrete structures. The efforts
Figure 1.1 Corrosion problem on various structural elements

provided by the research community to increase the acceptance of these new materials in new designs are exposed in Section 2.2.

1.3 Problematic

One of the major problems in reinforced and prestressed concrete structures is the corrosion of the steel bars and the prestressing steel tendons due to the humidity and the penetration of chlorides from de-icing salts in the concrete (Fig. 1.1).

An example of this problem is the bridge deck of the Jacques-Cartier Bridge in Montreal (Figs. 1.2 & 1.3). The bridge deck, which was 70 years old, was replaced in the summer 2001–02 for various reasons. Other than the age of the structure, the increase in weight and traffic as well as the extensive use of de-icing salts have affected the structure in such a way that it was not responding any more to the current standards. The damage caused by corrosion was the main factor that prompted the entire bridge deck replacement. In the new design, the engineers took various measures to increase the service life
of the new bridge deck structure [ZAKI and MAILHOT, 2003]. (Nevertheless, the final solution still used steel as reinforcement and prestressing.) Thus, the steel corrosion in the concrete may still be a concern in a number of years, and be a cause for further repairs of the Jacques-Cartier Bridge deck, and for many other similar structures. In the past twenty years, research on FRP materials applied to civil engineering structures has led to codes and design manuals. This now offers the possibility to substitute conventional steel with FRP materials in reinforced concrete structures ([CAN/CSA-S6-00, 2000], [CAN/CSA S806-02, 2002], [ISIS CANADA M-03, 2001]).

The main purpose of this research project was to investigate the structural behaviour of FRP reinforced and prestressed concrete bridge deck panels and to compare their performance with similar panels reinforced and prestressed with conventional steel. By taking the same geometry as the bridge deck panels of the Jacques-Cartier Bridge, the experimental panels were tested and compared to evaluate if the material substitution is a viable solution to overcome the steel corrosion problem.

1.4 Methodology

The research project is divided in two main parts: an experimental program and a numerical program.
The experimental program studies the behaviour of high-performance concrete bridge deck panels prestressed and reinforced entirely with FRP materials. Four specimens were instrumented and subjected to flexural and shear loads. Two of the four specimens had a DT cross-section and the two other had a single-tee (ST) cross-section. In each configuration, one panel was reinforced and prestressed with conventional steel, and the other one was reinforced and prestressed entirely with FRP materials. More details on the panel properties are presented in Section 4.2. Specimens reinforced and prestressed with steel will serve as reference in the testing program. The load–deflection behaviour, the crack distributions, the stress and strain distributions in the section as well as the failure modes have been studied. The anchor systems used to prestress the CFRP tendons are also presented.

In addition, the experimental values have been compared with the predictions obtained from two nonlinear finite element programs. The commercial software, ADINA, and the free software, Response 2000®, were used to examine the influence of various parameters, such as the number or the type of tendons and/or reinforcement, the level of prestressing and the concrete compressive strength.

The results of this research have been submitted for publication in the Canadian Journal of Civil Engineering [TARDIF et al., 2005a]. Presentation have also been made at the International Workshop on Innovative Bridge Deck Technologies in Winnipeg, Manitoba [TARDIF et al., 2005b], at the COBRAE Bridge Engineering with Polymer Composites Conference in Dübendorf, Switzerland [TARDIF et al., 2005c], at the Third Middle East
Symposium on Structural Composites for Infrastructure Applications (MESC-3) in Aswan, Egypt [ZAKI et al., 2002], at the PCI Bridge Conference in Orlando, United States [ZAKI et al., 2003a], at the Annual Conference of the Canadian Society for Civil Engineering in Moncton, New-Brunswick [ZAKI et al., 2003b] and at the “11e Colloque sur la progression de la recherche québécoise sur les ouvrages d’art” in Quebec, Quebec [DEMERS et al., 2004].
Chapter 2

Literature Review

2.1 Introduction

A literature review involving different aspects was initially performed to obtain an overview of previous research using FRPs as internal prestressing in concrete structures. First, standards and design manuals with FRPs were examined at to see if this kind of investigation had already been carried out elsewhere. Then, an overview of previous research was realized on research projects that studied the behaviour of concrete elements reinforced and/or prestressed with FRP materials. Later on in this chapter, the FRP properties from design manuals or manufacturers are presented. Finally, a state-of-the-art on the different types of anchorage systems for CFRP tendons is described.

2.2 Standards and Design Manuals with FRP

In order for FRP materials to be accepted in new reinforced concrete structures, researchers have written standards and design manuals to allow engineers to use these new materials in their designs. Their efforts have now been rewarded. As a result, several design books have already included chapters to give the principal guidelines for the design of various structural elements which use FRP materials as reinforcement and/or prestressing.

In Canada:
The ISIS Canada research network is one of the pioneers with regard to research on FRP materials. This research group has prepared a design manual for concrete structures reinforced with FRP [ISIS CANADA M-03, 2001]. Recently, ISIS Canada issued a new design manual for concrete elements prestressed with FRP [ISIS CANADA M-05, 2005]. These
design manuals are continuously being modified and updated as soon as the results of new research become available. For structural components of Canadian bridges, a technical committee, grouping together national and international experts on FRP materials, prepared Section 16 of the *Canadian Highway Bridge Design Code* (CHBDC) [Newhook et al., 2002; Bakht et al., 2000; Tadros, 2000; CAN/CSA-S6-00, 2000]. For buildings, a new Canadian standard was published in 2002 for the design and construction of concrete elements reinforced with FRP [CAN/CSA S806-02, 2002]. For a better comprehension of the issues when designing reinforced and prestressed concrete structures using FRP with the *CSA S806-02* Standard, a simplistic explanation of the different considerations was published in the *Canadian Civil Engineering* magazine [Razaqpur et al., 2004].

**In the United States:**
The *American Concrete Institute* (ACI) has also published a number of guidelines for the design and construction of concrete elements reinforced with FRP [ACI 440.1R-03, 2003]. In addition, Dolan et al. [2001] published three excellent reports on the current state of FRPs in structures and also made specific recommendations on specifications for AASHTO type beams. After carrying out several tests on concrete beams prestressed with CFRP tendons, researchers and engineers prescribed and validated equations for the flexural design of members [Grace and Singh, 2003; Dolan and Swanson, 2002; Burke and Dolan, 2001; Dolan and Burke, 1996]. At the end of 2004, ACI published one of the first manuals that provided guidelines for the design of concrete structures prestressed with FRP tendons [ACI 440.4R-04, 2004]. Other guidelines have also been written for reinforced concrete slabs with FRP [Bradbbery, 2001].

**In Japan:**
The use of FRPs is very prominent in Japan and guidelines for designing reinforced concrete structures with FRPs are well established [Sonobe et al., 1997].

Finally, a comparative study between various Canadian, European and Japanese researches on concrete elements prestressed with FRPs was carried out in order to validate each country's design philosophy [Gilstrap et al., 1997]. These standards and design manuals are used to guide structural engineers when they employ FRP in their designs.
2.3 Previous Research

To predict the behaviour of concrete elements reinforced and/or prestressed with FRPs, several research projects have been carried out and many others will undoubtedly be initiated in the future. Two major fields to be investigated to evaluate the potential of FRP as reinforcement and/or prestressing are: the behaviour under service loads, and the behaviour in shear and flexure at ultimate states. First, previous research on non-prestressed beams and slabs reinforced with FRP are presented here, followed by results on serviceability and ultimate limit states for prestressed concrete beams.

2.3.1 Concrete beams and slabs reinforced with FRPs

Without going into great detail, a presentation of some studies which were carried out on concrete beams and slabs using FRPs as internal reinforcement without prestressing is given. Four reinforced concrete beams with FRP bars were compared to four other concrete beams reinforced with conventional steel bars [BENMOKRANE et al., 1996]. Specimens measured 3.3 m long, 0.2 m width, 0.3 m and 0.5 m in height. Four point flexural tests were performed on eight different beams. The stress distributions for the elements reinforced with FRP bars demonstrated a perfect bond between the bars and concrete. Results on similar flexural tests on concrete beams reinforced with FRPs have also been presented by MASMoudi et al. [1998].

The behaviour of continuous concrete slabs reinforced with FRPs has also been a subject of research. A parametric study consisted of investigating the boundary conditions, the reinforcement ratios, the type of FRP reinforcement and the effects when reinforcement is added in the superior bed [HASSAN et al., 2000]. Eight one-way concrete slabs reinforced with FRP bars of 3.5 m by 1.0 m and between 0.15 m and 0.2 m in thickness were fabricated for this specific research. For these slabs, the flexural behaviour, the crack distributions and the failure modes were analyzed [MICHALUK et al., 1998].

A vast study on available FRP bars was conducted to use FRP bars as shear reinforcement. The study resulted in writing shear design guidelines for concrete beams reinforced with FRP stirrups [SHEHATA et al., 2000]. Ten concrete beams reinforced with FRPs, as well as 52 panel specimens specifically designed to study the bending behaviour were fabricated and tested. The parameters considered in the study were the type of FRP stirrups, the stirrup diameters, the bending radius, the stirrup anchorage configuration and the stirrup angle.
Next, we present research projects that were realized to understand the general behaviour of concrete beams prestressed with CFRP tendons.

2.3.2 Concrete beams prestressed with CFRP tendons: Serviceability Limit States

For concrete beams reinforced with FRP bars and prestressed with FRP tendons, the design is governed mostly by the service limit states rather than the ultimate states. This allows designers to obtain a certain ductility while limiting deflections and cracks. This means that the service loads must be well defined to adequately predict the deflections. A simplified method was proposed by Abdelrahman and Rizkalla [1999] for the deflection calculation of concrete beams partially prestressed with CFRP tendons by using the effective moment of inertia. This simplified method has been compared to experimental values in their research. The proposed equations represent well the behaviour of the beams under static and repetitive loads. A similar rational method was also proposed in another research project [Razaqpur et al., 2000]. This method is based on the moment-curvature integration relationship and is represented by a trilinear curve behaviour. To obtain a certain amount of ductility, researchers formulated a new way to describe the ductility in concrete structure reinforced and prestressed with FRPs. The ratio of the energy absorbed to the total energy was found to be a good criterion to characterize the ductility in such structures [Grace and Abdel-Sayed, 1998].

2.3.3 Concrete beams prestressed with CFRP tendons: Flexure and Shear

For the same level of prestressing, all experimental test results arrive at the same conclusions when the comparison is made between a beam prestressed with steel tendons and another beam prestressed with CFRP tendons. The cracking load is very similar in both cases; however, the behaviour is different at the failure load. Because CFRP tendons have a higher tensile strength than steel tendons and their behaviour is linear elastic until failure, for the case of beams prestressed with CFRP tendons, a higher failure load combined with a lower deflection is obtained compared to that for beams prestressed with conventional steel tendons. Beams prestressed with steel tendons exhibit more ductility, which allows them to reach higher deflections before failure.

A summary of different experimental concrete beams prestressed with CFRP tendons is given below. The main purpose of this section is not to present all the experimental
results, but to provide a general overview of the main characteristics of the specimens and main conclusions.

- Eight ST shaped specimens of 6.2 m in length, 0.33 m in height and a stem width of 0.105 m were investigated in an experimental program by Abdelrahman and Rizkalla [1997]. The slab width varied from 0.6 m to 0.2 m, according to the specimens. The beams were subjected to different loading tests with various CFRP tendon configurations and also prestressing forces which varied from 50% to 70% of the specified tensile strength.

- Two Type 2 AASHTO beams 12.19 m long were tested in flexure by a four point loading test set-up [Stoll et al., 2000]. This type of concrete beam with a height of 0.914 m, had a lower flange width of 0.457 m, a top flange width of 0.305 m and a slab thickness of 0.152 m. Eight CFRP tendons were prestressed at 77% of the specified tensile strength for the first beam and at 45% for the second beam. It is reported that three of the eight tendons ruptured during prestressing and another tendon failed during casting of the concrete. The results agreed well with the theoretical calculations, but more investigations were required to reach the design prestressing force in the CFRP tendons.

- Four ST shaped beams of 6.3 m in length, with a height of 0.33 m, a slab width of 0.45 m and a stem width of 0.105 m were tested by Abdelrahman et al. [1995]. These beams were reduced by a scale factor of 3.3 compared to the full-scale beams of the Calgary Bridge. In two beams, two 15.2 mm diameter CFCC tendons were used. In the two other beams, four 8.0 mm diameter CFRP tendons were used. For all beams, the prestressing force represented 60% of the manufacturer’s specified tensile strength. It was reported that it was necessary to pay a special attention to the level of prestressing, because a change in the mode of failure could occur. Two modes of failure are possible: crushing of the concrete in compression before rupture of the tendons, or a tensile rupture of the prestressing tendons. To resolve the problem of brittle ruptures, this research led to establishing a new method for evaluating the ductility of concrete beams prestressed with FRP tendons.

- Tests were carried out by Braimah et al. [2003] on concrete beams prestressed with CFRP tendons to verify their long-term behaviour. The concrete ST beams measured 4.4 m in length, 0.3 m in height, with a slab width of 0.5 m and a web
width of 0.13 m. The prestressing force varied between 50% and 70% of the specified tensile strength. Three beams were prestressed with CFRP tendons and one beam was prestressed with conventional steel tendons for comparison. The results demonstrated that the beams prestressed with CFRP tendons achieved a better long-term behaviour than beams prestressed by conventional steel tendons.

- A study was performed on a DT concrete beam prestressed with CFRP tendons and post-tensioned with CFCC tendons. This was done to validate the design of five beams constructed on the new Street Bridge in Michigan [Grace et al., 2003]. The beam was 20.88 m long, the slab width was 2.12 m, the height of the beam was 1.37 m and the width of each stem was 0.28 m. Thirty CFRP tendons of 10 mm and 12.5 mm in diameter were distributed in each stem. For the post-tensioning, four 40 mm diameter CFCC tendons were needed. The tests allowed the measurement of the strain distribution along the length and the height of the beam, the transfer length for the prestressed tendons, the curvature and deflection, the cracking load, the post-tensioning force in the CFCC tendons, the flexural ultimate load and the failure mode.

- Five DT concrete beams prestressed with CFRP tendons, two having a 15° skewed angle and three having a 30° skewed angle at there end, were fabricated and tested [Grace and Abdel-Sayed, 2000]. These beams were subjected to static, dynamic, repetitive, and eccentric loads. In the test program, the beams were loaded up to failure. Prestressing and post-tensioning were achieved using CFRP tendons. The beams measured 6.706 m in length, 0.368 m in height, a slab width of 1.016 m and a web width of 0.064 m.

- To be able to make comparisons, three concrete beams prestressed with CFRP tendons and three others prestressed with conventional steel tendons were loaded to study the flexural behaviour [Dolan and Swanson, 2002]. The ST shaped specimens measured 10 m in length, by 0.46 m in height, by 1.22 m for the slab width and 0.11 m for the web width. For three elements, the tendons were rectilinear and in three others, the tendons were in a parabolic shape. The prestressing force in the concrete beams prestressed with CFRP tendons was 60% of the specified tensile strength. The experimental values corresponded well with the theoretical equations.

- Five ST prestressed concrete shaped specimens of 9.3 m in length, by 0.55 m in
height, by 0.5 m for the slab width, with a web width of 0.2 m, were subjected to flexural loadings with various configurations of CFRP tendons, and also of prestressing force which varied from 58% to 62% of the specified tensile strength [FAM et al., 1997]. In the design, rectilinear and draped CFRP tendons were present. The flexural behaviour, the failure modes, the shear behaviour, the effect on the web reinforcement ratio and the effect of different elastic modulus for the reinforcement were analyzed.

The dowel action for FRP tendons in prestressed concrete beams was the object of some other studies [SALIB et al., 2002; PARK and NAAMAN, 1999]. Other studies on relaxation, creep and fatigue of CFRP tendons have also been conducted to understand their behaviour in prestressed concrete structures [SAADATMANESH and TANNOUS, 1999].

2.4 FRP Properties

The properties of various FRP products can be found in various documents such as the ISIS Canada manual on the design of concrete structures reinforced with FRPs [ISIS CANADA M-03, 2001]. These same properties can be directly obtained from manufacturers, although each of them has its own method to determine the mechanical and chemical characteristics of their products [MITSUBISHI CHEMICAL CORPORATION, 1996; HUGHES BROTHERS, 2002]. Considering this fact, many researchers developed a uniform methodology to determine the exact properties of FRP products [BENMOKRANE et al., 2002, 2000]. Several standards require that FRP products be guaranteed and tested before they are used as external or internal reinforcement in concrete structures.

2.5 Anchorage Devices for CFRP Tendons

One of the major difficulties in the construction of concrete beams prestressed with CFRP tendons is the prestressing of the CFRP tendons. Several anchor systems have been the subject of research and the conclusions and recommendations varies from one study to the other. One problem is that CFRP tendons have excellent mechanical properties in the longitudinal direction of the fibre; on the other hand, in the transverse direction, it is the weaker resin that controls the mechanical properties. Since anchor systems for the prestressing of steel tendons are already optimized, many anchor systems for CFRP tendons have great similarity with their predecessors. An anchor system is recognized effective
only if it provides at least 95% of the specified tensile strength of the tendon provided by the manufacturer [SAYED-ABDEL and SHRIVE, 1998]. The two main categories of anchor system available are the wedges-type and potted-type anchorages.

2.5.1 Wedges-type anchors

For the wedges-type anchor systems, a few systems are available:

- Split wedges anchor system
  This system is the very last one that was developed and tested at the University of Calgary [RETA TAHA and SHRIVE, 2003a]. The components of this system consist of four angle wedges and one sleeve. Both nonmetallic elements are made from an ultra-high-performance concrete of 200 MPa of compressive strength. This concrete was specifically designed to carry the high stresses in the anchor during prestressing. To make the concrete sleeve stronger and more durable, a CFRP sheet was wrapped around the sleeve. This system works very easily: the stronger the prestressing force applied to the tendon, the more the four concrete wedges tighten the tendon as these fit into the concrete sleeve. This type of concrete anchor system was designed for post-tensioning CFRP tendons, but it could be very applicable for pretensioning. This new anchor system avoids punctual crushing of the tendon by the conventional steel wedge edges [CAMPBELL et al., 2000]. The ultra-high-performance concrete sleeve/wedges anchor system provides an effectiveness of 95.9% [RETA TAHA and SHRIVE, 2003b].

Another type of wedges-type anchor was also devised at the University of Calgary by SAYED-ABDEL and SHRIVE [1998] and at the University of Waterloo by AL-MAYAH et al. [2001]. This one appears very similar to the anchor system use for prestressing conventional steel tendons. The anchorage consists of three components: a stainless steel sleeve with a conical socket; a four-piece set of stainless steel conical wedges, and a thin soft metal sleeve that is placed between the wedges and the tendon. The effectiveness of this system has exceeded 100% of the specified tensile strength of the tendon, which means that the values supplied by the manufacturer were underestimated. In their study, they also modelled the anchor system using a finite element program to determine the effects of the prestressing force as well as the coefficient of friction used between the different components. In a similar
study, it is mentioned that steel sleeve/wedges anchor had made good progress, but that certain parameters causing the rupture of the tendon still remain unknown. Supplementary studies are required [CAMPBELL et al., 2000]. One of the problems encountered with this system is the local damage on the tendon caused by the sharp edges of the angle wedges. Another problem is the local shear failure of the tendon caused by excessive shear stresses in the anchor zone [RETA TAHIA and SHRIVE, 2003a]. However, at this moment this anchor system is the most effective, compact, easy to assemble, reusable and reliable on the market [NANNI et al., 1996].

A variant of this system uses plastic wedges [KERSTENS et al., 1998]. The system was studied at the University of Technology in Eindhoven, Holland. This type of anchor encountered slipping problems due to the lack of roughness between the interface of the tendon and the wedges. In the tests, no tendon attained more than 90% efficiency because of insufficient gripping.

New improvements have been brought to the steel sleeve/wedges anchor system. The components are identical, except cold-swaging (mechanical action which squeezes or removes a part of material of a piece) is done in the anchor zone on the tendon as well as on the wedges [PINCHEIRA and WOYAK, 2001]. The cold-swaging on the pieces allows a better fitting of the tendon and the wedges to develop a higher shear resistance during prestressing.

- Conical pressure anchor system (multi-tendons)
  DYWIDAG International company has fabricated a multi-tendon anchor system to simultaneously apply prestressing on seven CFRP tendons from 5 mm to 7 mm in diameter. This system gives good results, but the maximum applied prestressing force is limited to 280 kN and 520 kN for the 5 mm and 7 mm diameters, respectively [NOISTERNIG and JUNGWIRTH, 1996].

### 2.5.2 Potted-type anchors

For the potted-type anchor systems, a few systems are also available:

- Contoured sleeve anchor filled with resin
  The Swiss Federal Laboratories for Materials Testing and Research (EMPA) in association with Swiss company BBR, developed and patented this type of anchorage [FISHER and BASSETT, 1997; KIM et al., 1998]. This system is in use for the
prestressing of one CFRP stay cable on the Stork Bridge in Winterthur, Switzerland [Schurter and Meier, 1996]. The components consist of a tubular metallic housing with an conical inside form and filled with a special epoxy matrix containing a high-modulus ceramic filler. The resin is deposited in layers of different percentages of ceramic filler into the tubular housing, then is left a moment so it hardens, and finally prestressing is applied. This system offers more than 90% efficiency. Because this system is not easy to use and time consuming in the construction field it has not become a commercial product. The prestressing step takes more time, because a period of time is needed for the resin hardening. In addition, this anchorage cannot be reused.

The University of Stuttgart, in Germany, conducted tests on multi-tendon anchorages with sleeves filled with resin [Sippel, 1992]. This anchorage was designed for GFRP tendons, but could also be applicable to CFRP tendons. With this system, eight 7.5 mm diameter tendons can be prestressed at the same time. Its efficiency reaches 98%. To optimize this system, the anchorage was modelled using a finite element program.

- Cylinder sleeve anchor filled with resin
  At the University of Sherbrooke, Zhang and Benmokrane [1997] studied an anchor system made of a cylindrical sleeve shape filled with resin. They also developed theoretical equations with the objective to predict the maximal prestressing force which can be applied on the tendon.

- Sleeve/wedges anchor filled with highly expansive materials
  Developed in Japanese universities, this system resembles the steel sleeve/wedges anchor, except that a space is left between the tendon and the wedges to insert a highly expansive material [Khin et al., 1996]. This material allows a better shear transfer between the tendon and the wedges without damaging the tendon. An analytical approach was also used to determine the stress distributions in the components.

In this chapter, the design recommendations from standards and design manuals of three countries have been presented. Then, the general characteristics and conclusions of previous projects similar to the research of this thesis were given. Some researchers investigated the serviceability behaviour, while others studied the shear and flexural ultimate capacities. The experiments showed the same conclusions for flexural loading. In the last part of this chapter, we reviewed the many efforts that have been devoted to produce an
anchorage system that can develop the full tensile strength of the CFRP tendons. The wedges-type and the plotted-type were the two main categories of anchorage systems that have been investigated. Anchor system that can be reusable, easy and fast to install in the field are also characteristics that also have to be considered. The next chapter presents the theoretical equations to predict the loss of prestress, to calculate the flexural and shear capacities and to evaluate the deflections of concrete elements reinforced and prestressed with FRP materials.
Chapter 3

Design Methods

3.1 Introduction

Various equations will be presented in this chapter to predict the loss of prestress, to calculate the flexural capacity, the shear capacity and to estimate the deflections for elements reinforced and prestressed with composite materials. Many equations were taken from ACI 440.4R-04 [2004], CAN/CSA-S6-00 [2000], CSA A23.3 – 94 [1994], ISIS CANADA M-03 [2001], PICARD [2001] and COLLINS and MITCHELL [1997]. In most of these references, the equations are provided for concrete structures prestressed and reinforced with steel. As a result, some equations were modified to add the contribution of the FRP materials in concrete structures prestressed and reinforced with FRPs.

3.2 Material Properties

To calculate the loss of prestress, the flexural and shear strength and the deflection, we need to know the mechanical properties of each component inside the elements; i.e., the concrete, tendons, reinforcement bars and stirrups. From their stress–strain behaviours, the entire behaviour of the elements can be analyzed. Specific experimental tests exist to characterize the theoretical equations for all these materials. In the laboratory, it is possible to check the equations with tests on material samples. The tests on material samples allow the structural engineers to validate the equation assumptions which are used in their designs.
3.2.1 Concrete

The mechanical properties of concrete depend on the proportion of cement, water and aggregates that constitute the concrete. A small water/cement (w/c) ratio will produce a concrete with a higher compressive strength, but on the other hand, this concrete will have less workability. To obtain a concrete with a high compressive strength and which is easy to pour, a superplasticizer is added in the mixture without increasing the w/c ratio. To increase the concrete durability, an air entraining admixture between 3% to 10% is also added in the concrete mixture. Fly ash and silica fume can also be used to reduce the quantity of cement and also to improve some of the concrete mechanical properties.

Concrete compressive strength

The concrete compressive strength is one of the fundamental properties to characterize concrete and is the most measured. This property is very important because the other mechanical properties of concrete (i.e., the tensile strength and modulus of elasticity) are usually a function of the compressive strength. In design, we generally use the specified concrete compressive strength, $f'_{ct}$, at 28 days, except when the concrete is stressed at an earlier age. In this case, the concrete compressive strength at the day of stressing, $f'_{ct}$, is used. An example of an early age stressing on concrete is the transfer of the prestressing force a few days, even sometimes few hours, after the concrete is poured in the mould. Uniaxial compressive tests on 300 mm in height by 150 mm in diameter cylinder samples are used to determine the value of $f'_{c}$ according to the method described in the CSA A23.2–9C Standard.

The increase of compressive strength after 28 days is generally neglected. However, the compressive strength before 28 days ($f'_{ct}$) can be predicted by the following equation

$$f'_{ct} = \frac{t}{a + bt} f'_{c} \quad (t \leq 28 \text{ days}) \quad (3.1)$$

The constants $a$ and $b$ are given in Table 3.1 for conventional concretes ($f'_{c} \leq 40 \text{ MPa}$) using the two most popular cement powders and for a normal or fast curing.
TABLE 3.1 CONSTANTS $a$ AND $b$ FOR Eq. 3.1

<table>
<thead>
<tr>
<th>Normal curing</th>
<th>Fast curing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>Cement</td>
</tr>
<tr>
<td>type 10</td>
<td>type 30</td>
</tr>
<tr>
<td>$a = 4.00$</td>
<td>$a = 2.30$</td>
</tr>
<tr>
<td>$b = 0.85$</td>
<td>$a = 1.00$</td>
</tr>
<tr>
<td>$b = 0.92$</td>
<td>$a = 0.70$</td>
</tr>
<tr>
<td>$b = 0.95$</td>
<td>$b = 0.98$</td>
</tr>
</tbody>
</table>

Note: For high-performance concrete with a normal curing, we can use $a = 1.40$ and $b = 0.95$.

For high performance concrete (HPC) ($40 < f'_c \leq 80$ MPa), the increase of the compressive strength is greater at an early age and for a normal curing, we can used Eq. 3.1 with the constants $a = 1.40$ and $b = 0.95$. HPC reaches 43% of its 28 day compressive strength after one day and 87% after 7 days.

The complete concrete compressive behaviour can be determined by the model proposed by Collins and Mitchell [1997]. Equation 3.2 describes the relationship between the concrete stress at the level considered, $f_c$, and its corresponding strain, $\varepsilon_c$

$$\frac{f_c}{f'_c} = \frac{n(\varepsilon_c/\varepsilon_o)}{n - 1 + (\varepsilon_c/\varepsilon_o)^{nk}}$$

(3.2)

Here:

- $\varepsilon_o$ : concrete strain corresponding to the maximum stress $f'_c$,  
- $n$ : fitting curve coefficient,  
- $E'_c = f'_c / \varepsilon_o$,  
- $E_c$ : modulus of elasticity of concrete (Eq. 3.6),  
- $k$ : stress reduction coefficient.

These coefficients can be calculated using Eqs. 3.3 to 3.5.

- Fitting curve coefficient, $n$:
  
  For a normal concrete, Eq. 3.3 is used to estimate the value of $n$.

$$n = 0.8 + \frac{f'_c}{17}$$

(3.3)

- Strain, $\varepsilon_o$, at the maximum stress, $f'_c$:

$$\varepsilon_o = \frac{f'_c}{E_c} \frac{n}{n - 1}$$

(3.4)
Figure 3.1  Typical stress–strain compressive curves for concrete

where $E_c$ can be evaluated with Eq. 3.6.

- Stress reduction coefficient, $k$:

$$k = 0.67 + \frac{f'_c}{62} > 1.0$$  \hspace{1cm} (3.5)

equal to 1.0 for $\varepsilon_c / \varepsilon_o < 1.0$, and higher than 1.0 for $\varepsilon_c / \varepsilon_o > 1.0$.

Typical curves for different grades of concrete ($f'_c$) are shown in Fig. 3.1. In many design codes, the maximum compressive strain, $\varepsilon_{cu}$, is taken as 0.0035 for $f'_c \leq 80$ MPa. The value of $\varepsilon_o$ varies between 0.002 to 0.0025 for normal concrete.

**Concrete elastic modulus**

The modulus of elasticity of concrete depends on the moduli of elasticity of the constituents inside the concrete mixture. Also, it depends on the concrete density, $\gamma_c$, and its compressive strength, $f'_c$. $E_c$ corresponds to the slope connecting the points $f_c = 0.0$ and $f_c = 0.4 f'_c$. In North-American codes, the concrete elastic modulus is estimated with Eq. 3.6 at the age of 28 days. This equation is valid for $30 \leq f'_c \leq 80$ MPa.

$$E_c = (3300 \sqrt{f'_c} + 6900) \left( \frac{\gamma_c}{2300} \right)^{1.5}$$  \hspace{1cm} (3.6)
Concrete tensile strength

The concrete tensile strength is generally small and very variable. Before cracking, the concrete in tension behaves like an elastic material. The concrete modulus of cracking, \( f_r \), is determined from Eq. 3.7. It is used to calculate the flexural cracking moment, \( M_{cr} \), and to determine the allowable tensile stresses for flexural elements.

\[
    f_r = 0.6 \lambda \sqrt{f'_c}
\]  

(3.7)

\( \lambda \) : concrete density coefficient (1.0 for normal density concrete),
\( f'_c \) : specified concrete compressive strength.

The Poisson’s ratio and the thermal expansion coefficient are two other concrete properties used in the calculations. For uncracked concrete, 0.2 can be used for the Poisson’s ratio, although this value is equal to zero for cracked concrete. For the thermal expansion coefficient, \( 10 \times 10^{-6}/^\circ\text{C} \) is employed in the designs.

3.2.2 Steel reinforcement bars and prestressing tendons

The steel behaviour is very different compared to that of concrete since it is an elastic and ductile material. Few models are proposed to estimate the theoretical stress–strain behaviour of steel reinforcement. Some models are used for structural design, while others are used to investigate with precision the results from laboratory tests.

Steel reinforcement bars

For normal steel reinforcement bars, the elastic modulus, \( E_s \), is equal to 200,000 MPa. The important points in the behaviour of steel are: the specified yield stress, \( f_y \), the corresponding strain at yielding, \( \varepsilon_y \), and the strain hardening point, \( \varepsilon_{sh} \). Finally, \( f_{su} \) and \( \varepsilon_{su} \) are respectively the ultimate tensile strength and ultimate tensile strain of steel reinforcement bars. PARK and PAULAY in 1975 proposed Eq. 3.8 to obtain the complete stress–strain relationship for steel bars.

\[
    f_s = \begin{cases} 
    \varepsilon_s E_s \\
    f_y \left[ \frac{m (\varepsilon_s - \varepsilon_{sh}) + 2}{60 (\varepsilon_s - \varepsilon_{sh}) + 2} + \frac{(\varepsilon_s - \varepsilon_{sh}) (60 - m)}{2 (30 r + 1)^2} \right] 
    
    & \text{for } 0 \leq \varepsilon_s \leq \varepsilon_y; \\
    f_y & \text{for } \varepsilon_y \leq \varepsilon_s \leq \varepsilon_{sh}; \\
    f_y \left[ \frac{m (\varepsilon_s - \varepsilon_{sh}) + 2}{60 (\varepsilon_s - \varepsilon_{sh}) + 2} + \frac{(\varepsilon_s - \varepsilon_{sh}) (60 - m)}{2 (30 r + 1)^2} \right] 
    
    & \text{for } \varepsilon_{sh} \leq \varepsilon_s \leq \varepsilon_{su}. 
    \end{cases}
\]  

(3.8)

where:
Figure 3.2  Typical tensile stress–strain curves for steel reinforcement bars

\[
\begin{align*}
    r & = \varepsilon_{su} - \varepsilon_{sh} \\
    m & = \frac{(f_{su}/f_y)(30r + 1)^2 - 60r - 1}{15r^2}
\end{align*}
\]

Equation 3.8 is used for research purposes because it considers the strain hardening of the steel. For design applications, the steel behaviour is simplified by neglecting the increase of the stress after the yielding point and is modelled with a bilinear stress–strain curve as in Eq. 3.9.

\[
f_s = \begin{cases} 
    \varepsilon_s E_s & \text{for } 0 \leq \varepsilon_s \leq \varepsilon_y; \\
    f_y & \text{for } \varepsilon_s > \varepsilon_y.
\end{cases}
\]  \hspace{1cm} (3.9)

The two curves from Eqs. 3.8 and 3.9 are presented in Fig. 3.2. The mechanical properties of steel bars are assumed to be the same in tension as in compression.

**Steel prestressing tendons**

To prestress concrete elements, steel tendons with a high strength and high yielding point are used. A steel tendon is characterized by its ultimate tensile stress, \( f_{puu} \), its yielding point, \( f_{psy} \), its elastic modulus, \( E_{ps} \), and its ultimate strain at failure, \( \varepsilon_{psu} \). Two categories are available on the market: normal relaxation and low relaxation. Steel relaxation produces loss of prestress with time and is discussed in Section 3.3.2. For construction
projects, low relaxation steel tendons are generally recommended. Even if the initial cost for low relaxation steel tendons is higher, the additional cost is largely compensated by the reduction of loss of prestress. For low relaxation steel tendons, $E_{ps} = 200,000$ MPa and $\varepsilon_{psu} \geq 0.035$. Since steel tendons do not have as obvious a yielding point as do steel reinforcement bars, $f_{psy}$ is defined as the stress which corresponds to the strain $\varepsilon_{psy} = 0.01$. In CAN/CSA-S6-00 [2000], the maximum prestressing stress recommended at jacking, $f_{pju}$, has to be less than $0.80 f_{psy}$. The general stress–strain curve for low relaxation steel tendons is given by Eq. 3.10 and is illustrated in Fig. 3.3 [COLLINS and MITCHELL, 1997].

$$f_{ps} = E_{ps} \varepsilon_{ps} \left\{ 0.025 + \frac{0.975}{\left[1 + (118 \varepsilon_{ps})^{0.10}\right]^{0.10}} \right\} \leq 1860 \text{ MPa} \quad (3.10)$$

### 3.2.3 FRP

The stress–strain behaviour of FRPs is linear elastic until failure (Eq. 3.11).

$$f_{FRP} = E_{FRP} \varepsilon_{FRP} \quad (3.11)$$
The ultimate tensile strength, $f_{FRP}$, is used in analyses. This value can be provided by the manufacturer or determined by standardized tests. The ultimate tensile strength depends on the type of fibres, the fibre volume ratio and the fabrication process. Generally, FRP with glass fibres has a lower tensile strength than FRP with carbon fibres or aramid fibres. The elastic modulus also depends on the type of fibres. $E_{FRP}$ varies from 30 GPa for GFRP to 300 GPa for CFRP. The elastic modulus can be determined directly from tensile tests.

The compressive strength of FRPs is relatively low compared to its tensile strength. The compressive strength also depends on the type of fibres, the fibre volume ratio and the fabrication process. The compressive strengths of AFRP bars, CFRP bars and GFRP bars are about 10%, 30% to 50%, and 30% to 40% of their tensile strengths, respectively. The compression elastic modulus is a function of the length/diameter ratio, the type of bar, its size, and also other factors such as the compression loading conditions. From compression test results, the compressive elastic modulus for FRPs is between 77% to 97% of its tensile elastic modulus.

Figure 3.4 presents the stress–strain curves for different types of FRP bars used for internal reinforced and prestressed concrete elements. Many other characteristic properties for FRP products are presented in the ACI 440.4R-04 [2004] document and ISIS CANADA M-03 [2001] manual.
To prestress concrete elements with FRP products, only aramid and carbon fibres are recommended. Glass fibres have a poor resistance to creep under sustained loads and are more susceptible to alkaline degradation than carbon and aramid fibers. However, GFRP are good material for passive reinforcement and stirrups.

3.3 Loss of Prestress

In this section, the theoretical equations to evaluate the loss of prestress in prestressed concrete elements will be presented [Picard, 2001]. The total loss of prestress can be divided into five individual losses of prestress. Also, the individual losses of prestress are grouped into two different time scales. First, the instantaneous loss of prestress occurs at jacking and transfer; then, a long-term loss of prestress occurs because of the concrete and steel material properties.

3.3.1 Instantaneous loss

During prestressing, the tendons are first stressed and blocked with the anchorage system. At this stage, a loss of prestress occurs with the deformation of the anchor system, the slippage in the anchorage and the shortening of the steel mould. This loss of prestress can be estimated with Eq. 3.12.

\[
\Delta T = \frac{\sum_{i=1}^{3} g_i}{L} A_p E_p
\]  

(3.12)

\(g_i\) : deformation of the anchor system (\(\approx 1\ mm\)),
\(g_s\) : slippage in the anchorage (between 4 mm to 10 mm),
\(g_h\) : shortening of the steel mould (\(\approx 1\ mm\)).

Then the concrete is poured in the mould and cured until the concrete reaches its design compressive strength at transfer. The prestressing force can be transferred during a period as short as one day after pouring the concrete depending on the type of curing process and also the required compressive strength before transfer. At transfer, the concrete mass is stressed due to the prestressing force and produces an elastic shortening of the concrete.
This value can be evaluated using Eq. 3.13.

\[ \Delta P_{elas} = \Delta \sigma_{elas} A_p = \left( \frac{f_{cs}}{E_{ci}} \right) E_p A_p \]  

(3.13)

where:

- \( E_{ci} \): concrete elastic modulus at the time of transfer,
- \( f'_{ci} \): compressive strength at the time of transfer,
- \( f_{cs} \): concrete stress at the tendons' centre of gravity, at a considered cross-section,
- \( f_{cs} \) can be calculated using Eq. 3.14 at any section along the element.

\[ f_{cs} = \frac{P_i}{A_g} + \frac{M_p e}{I_g} + \frac{M_g e}{I_g} \]  

(3.14)

where:

- \( P_i \): prestressing force at transfer,
- \( A_g \): gross area of section,
- \( M_p \): moment due to prestressing, \( = P_i \cdot e \),
- \( M_g \): moment due to girder weight,
- \( e \): eccentricity of tendon,
- \( I_g \): moment of inertia of gross concrete section about the centroid axis, neglecting the passive reinforcement.

By adding Eq. 3.12 to 3.13, the loss of prestress from the instantaneous loss is determined. Then, in the case of precast elements, they are taken out from the mould and installed at their place on the construction site. The long-term loss of prestress is presented in the next section.

### 3.3.2 Long-term loss

The long-term loss of prestress in prestressed concrete element results from the shrinkage and creep of the concrete and also from the relaxation of the steel tendons. Relaxation for CFRP tendons can be neglected in the calculations.

**Shrinkage of the concrete**

Unless kept under water or in air at 100\% relative humidity, the concrete loses moisture with time and decreases in volume, a process known as shrinkage. The amount of shrinkage depends strongly on the concrete composition and the \( w/c \) ratio in the concrete mixture.
The shrinkage of the concrete begins after the curing is complete. Equation 3.15 is a theoretical approach to estimate the loss of prestress due to the shrinkage of the concrete.

\[ \Delta P_{shc}(t) = \Delta f_{shc}(t) A_p = \varepsilon_{sh}(t) E_p A_p \]  (3.15)

To estimate the strain variation, a reference strain, \( \varepsilon_{sh} \), is first defined by Eq. 3.16.

\[ \varepsilon_{sh} = (850 - 0.015 E_c) \times 10^6 \]  (3.16)

Then, the time effect on the strain variation is calculated from Eq. 3.17.

\[ \varepsilon_{sh}(t) = \varepsilon_{sh} \cdot K_{shh} \cdot K_v \cdot K_r \cdot f_{sh}(t) \]  (3.17)

where:

- \( K_{shh} \): relative humidity coefficient, \( H \),
  \[ = 2 - 0.0143 H \]
- \( K_v \): coefficient that consider the volume/area ratio exposed to air, \( v \),
  \[ = 1.13 - 0.0032 v \quad \text{for} \quad v < 150 \text{ mm}; \]
  \[ = 0.65 \quad \text{for} \quad v \geq 150 \text{ mm}. \]
- \( K_r \): reinforcement ratio coefficient, \( r \),
  \[ = 1 - 17 r \]
- \( f_{sh}(t) \): time function,
  \[ = \frac{t}{(t + 19 v)} \]

**Creep of the concrete**

The stress–strain response of concrete depends upon the rate of loading and the time history of the loading. If the stress is maintained constant for a period of time, the strain will increase. This phenomenon is referred to as creep of the concrete. Creep starts only after the prestressing force is transferred to the concrete because it induces a permanent compression stress in the concrete. The equation proposed for the loss of prestress due to creep is valid for concrete without silica fume and for specified concrete compressive strengths \( f'_c \) lower than 80 MPa (Eq. 3.18).

\[ \Delta P_{cre}(t) = \Delta f_{cr}(t) A_p = \varepsilon_{cr}(t) E_p A_p = \varepsilon_i C_{cr}(t) E_p A_p \]  (3.18)

To start the calculation, the elastic shortening of the concrete, \( \varepsilon_i \), is taken from the calculation of the instantaneous loss of prestress at the transfer of the prestressing force
on the concrete (Eq. 3.13) with \( \varepsilon_i = f_{ci} / E_{ci} \). After, a reference value, called the final creep coefficient, \( C_{cr} \), is first determined with Eq. 3.19.

\[
C_{cr} = \begin{cases} 
4.25 - 0.033 f'_c & \text{for } f'_c \leq 50 \text{ MPa;} \\
3.60 - 0.020 f'_c & \text{for } 50 < f'_c \leq 80 \text{ MPa.}
\end{cases}
\] (3.19)

Finally, the time effect on the creep coefficient, \( C_{cr}(t) \), is calculated from Eq. 3.20.

\[
C_{cr}(t) = C_{cr} K_{crh} K_v K_r K_t f_{cr}(t)
\] (3.20)

where:

- \( K_{crh} \) : relative humidity coefficient, \( H \),
  \[ K_{crh} = 1.70 - 0.01 H \]
- \( K_v \) : coefficient that consider the volume/area ratio exposed to air, \( v \),
  \[ K_v = \begin{cases} 
1.13 - 0.0032 v & \text{for } v < 150 \text{ mm;} \\
0.65 & \text{for } v \geq 150 \text{ mm.}
\end{cases} \]
- \( K_r \) : reinforcement ratio coefficient, \( r \),
  \[ K_r = 1 - 17 r \]
- \( K_t \) : coefficient considering the age of the concrete at transfer, \( t_{co} \),
  \[ K_t = (t_{co})^{0.12} \]
- \( f_{cr}(t) \) : time function,
  \[ f_{cr}(t) = \left( \frac{t}{10 + t^{0.6}} \right) \]

**Relaxation of the steel tendons**

The force required to hold a highly stressed steel tendon at a given elongation will reduce with time. This phenomenon is referred to as relaxation. Relaxation is negligibly small if the initial stress, \( f_{psi} \), applied to the steel is less than 0.55 \( f_{psy} \). Relaxation for steel is analogous in many ways to creep of concrete and, like creep, relaxation can be accurately predicted only if information for the specific material under the specific conditions is available. The relaxation of steel tendons starts at the time of prestressing of the tendons. Equation 3.21 is proposed to estimate the loss of prestress by relaxation for low relaxation steel tendons.

\[
\Delta P_{rela}(t_i \text{ to } t_j) = \frac{P_{is}}{A_{ps}} \left( \log \frac{24 t_j - 24 t_i}{45} \right) \left( \frac{P_{is}}{A_{ps} f_{psy}} - 0.55 \right) A_{ps}
\] (3.21)

where:

- \( t_i \) and \( t_j \) : time interval considered to calculate the relaxation, days.
By adding Eqs. 3.15, 3.18 and 3.21, the total long-term loss is evaluated. Then, by adding the instantaneous loss with the long-term loss, the total loss of prestress is predicted.

### 3.4 Flexural Strength

Since the equations derived to design the flexural strength of prestressed concrete with steel are well established in many references, this section will present only the equations derived to design the flexural strength of prestressed concrete with FRP reinforcement. The basic approaches in the design of flexural strength is the same for both materials. For FRP prestressed elements, they will deform elastically until cracking, then a linear post-cracking behaviour occurs under the increase of load until the tendon ruptures or the ultimate concrete compression strain is exceeded.

The assumptions taken for the sectional analysis of prestressed concrete with FRPs are listed below:

- Ultimate compressive strain for the concrete, $\varepsilon_{cu}$, is 0.0035 (if compression tests show that $\varepsilon_{cu} > 0.0035$, the higher strain should be taken in the calculations),

- Tensile strength of the concrete is neglected in cracked sections,

- A equivalent rectangular stress block is used to model the concrete behaviour,

- Strain distribution in the concrete and FRPs is linear over the depth of the section (i.e., plane sections remain plane); this assumption implies strain compatibility,

- Stress–strain relationship for the FRPs is linear up to failure,

- Full bonding condition is assumed between the concrete and FRPs,

- Passive FRP reinforcement in the compression zone is neglected in the design.

In the next section, the equations for the three possible failure modes in flexure are presented.

#### 3.4.1 Failure modes

Three modes are possible for the flexural failure for a concrete section prestressed with FRPs:
1. **Balanced failure** — simultaneous failure by rupture of the FRP tendons and crushing of the concrete,

2. **Compression-controlled failure** — concrete fails in compression before failure of the FRP tendons,

3. **Tension-controlled failure** — FRP tendon failure in tension before failure of the concrete.

The concept of a balanced ratio is the approach used for the design of concrete prestressed with FRPs. The balanced ratio corresponds to the prestress reinforcement ratio that simultaneously results in rupture of the tendons and crushing of the concrete.

1. **Balanced failure**

Figure 3.5 shows the cross-section and the strain and stress conditions for a bonded section that occur at the balance failure. The equations presented are for a rectangular section or a T-section with a single layer of FRP tendons where the compression block is within the depth of the flange, i.e., $a < h_f$.

Equation 3.22 presents the amount of strain available for flexure, $\varepsilon_f$.

$$\varepsilon_f = \varepsilon_{pu \text{FRP}} - \varepsilon_{pc \text{FRP}} - \varepsilon_d - \varepsilon_{pr}$$  \hspace{1cm} (3.22)

where:

- $\varepsilon_{pu \text{FRP}}$ : ultimate tensile strain of the FRP tendon,
- $\varepsilon_{pc \text{FRP}}$ : strain used for the prestressing,
- $\varepsilon_d$ : strain used to decompress the concrete,
- $\varepsilon_{pr}$ : any loss of strain capacity due to sustained loads.
Then, from the strain compatibility of Fig. 3.5, it is possible to determine the \(c/d\) ratio in terms of \(\varepsilon_f\) by using similar triangles.

\[
\frac{c}{d} = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{pu,FRP} - \varepsilon_{pe,FRP} - \varepsilon_d - \varepsilon_{pr}} \tag{3.23}
\]

Equilibrium of the section equals the tensile force in the tendons to the compressive force on the concrete, \(C = T\). Hence,

\[
C = \alpha_1 f_c' \beta_c b 
\tag{3.24}
\]

\[
T = \rho \, d \, b \, f_{pu,FRP} 
\tag{3.25}
\]

where:

\[
\alpha_1 = 0.85 - 0.015 f_c' \geq 0.67,
\]

\[
\beta_1 = 0.97 - 0.025 f_c' \geq 0.67.
\]

Solving Eqs. 3.24 and 3.25 for the balanced reinforcement ratio, \(\rho = \rho_b\), where \(\rho = A_{pu,FRP}/bd\) is the prestress reinforcement ratio, gives:

\[
\rho_b = \alpha_1 \beta_1 f_c' f_{pu,FRP} \frac{c}{d} \tag{3.26}
\]

Replacing the expression for \(c/d\) from Eq. 3.23 into Eq. 3.26 gives the balanced ratio in terms of the material properties:

\[
\rho_b = \alpha_1 \beta_1 f_c' f_{pu,FRP} \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{pu,FRP} - \varepsilon_{pe,FRP} - \varepsilon_d - \varepsilon_{pr}} \tag{3.27}
\]

It is possible to simplify the last equation. First, the strain loss due to sustained loads, \(\varepsilon_{pr}\), is approximately zero if this strain is less than 50% of the ultimate tensile strain. Second, the decompression strain, \(\varepsilon_d\), is typically an order of magnitude less than the flexural strain, \(\varepsilon_f\). After simplifications, \(\rho_b\) is now given by Eq. 3.28.

\[
\rho_b = \alpha_1 \beta_1 f_c' f_{pu,FRP} \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{pu,FRP} - \varepsilon_{pe,FRP}} \tag{3.28}
\]

Depending on the prestress reinforcement ratio, \(\rho\), if \(\rho > \rho_b\), the flexural behaviour of the element will be a compression-controlled failure, and in the other situation, if \(\rho < \rho_b\), the flexural behaviour will be a tension-controlled failure.
2. Compression-controlled failure

For compression-controlled failure, the concrete will crush before the rupture of the tendons. In this condition, the stress and strain compatibility is the same as in Fig. 3.5; except, the strain value of the tendon is unknown. This type of condition is analyzed by locating the neutral axis, assuming a linear elastic behaviour for the FRP tendons and taking an equivalent rectangular stress block for the concrete. Knowing the ultimate compressive strain of the concrete, the strain in the tendons can be defined, then the horizontal forces on the section are equilibrated, after the neutral axis location is solved, and finally the moments are summed about the point where the tendons are located. The depth of the neutral axis, $c$, is calculated by considering axial force equilibrium of the cross-section. The FRP tendon behaviour is taken as presented in Eq. 3.11 ($f_{\text{FPRP}} = E_{\text{FPRP}} \varepsilon_{\text{FPRP}}$).

$$\rho b d f_{\text{FPRP}} = \alpha_i f_c' \beta_i c b$$  \hspace{1cm} (3.29)

From the strain diagram at failure, the flexural strain in the tendons can be determined using similar triangles:

$$\varepsilon_f = \varepsilon_{cu} \frac{d - c}{c}$$  \hspace{1cm} (3.30)

The total strain in the tendons, $\varepsilon_{\text{FPRP}}$, is the sum of the effective prestressing strain, $\varepsilon_{\text{peFPRP}}$, and the flexural strain, $\varepsilon_f$:

$$\varepsilon_{\text{FPRP}} = \varepsilon_{\text{peFPRP}} + \varepsilon_{cu} \frac{d - c}{c}$$  \hspace{1cm} (3.31)

Replacing Eqs. 3.31 and 3.11 into Eq. 3.29 and defining $k_u = c/d$ results in:

$$\rho \left( \varepsilon_{\text{peFPRP}} + \varepsilon_{cu} \frac{1 - k_u}{k_u} \right) E_{\text{FPRP}} = \alpha_i f_c' \beta_i k_u$$  \hspace{1cm} (3.32)

Defining a material constant $\lambda$ such that:

$$\lambda = \frac{E_{\text{FPRP}} \varepsilon_{cu}}{\alpha_i f_c' \beta_i}$$  \hspace{1cm} (3.33)

34
and substituting Eq. 3.33 into Eq. 3.32 allows the resulting quadratic equation to be solved for $k_u$, giving:

$$k_u = \sqrt{\rho \lambda + \left[ \frac{\rho \lambda}{2} \left( 1 - \frac{\varepsilon_{pe,FRP}}{\varepsilon_{cu}} \right) \right]^2 - \frac{\rho \lambda}{2} \left( 1 - \frac{\varepsilon_{pe,FRP}}{\varepsilon_{cu}} \right)}$$  \hspace{1cm} (3.34)

The nominal moment capacity is then determined by summing the moments about the centroid of the tendons, giving:

$$M_n = \alpha_i \cdot f'_c \beta_i \cdot b \cdot k_u \cdot d^2 \left( 1 - \frac{\beta_i \cdot k_u}{2} \right)$$  \hspace{1cm} (3.35)

3. Tension-controlled failure

For tension-controlled failure, the ultimate strain in the FRP tendons is reached before the crushing of the concrete. In this case, the concrete strain, $\varepsilon_c$, will not reach 0.0035, and the use of the equivalent rectangular stress block assumption would not be valid. However, for sections with $0.5 \rho_b < \rho < \rho_b$, the concrete stress distribution will be substantially nonlinear at failure and thus an equivalent rectangular stress distribution can be assumed. Using this assumption, the nominal moment capacity of a tension-controlled failure with a single layer of FRP tendons was developed using the strength design based on a rectangular stress block.

The nominal moment capacity is defined by the summation of the moments about the compression centroid (Fig. 3.5):

$$M_n = \rho b d f_{pu,FRP} \left( d - \frac{a}{c} \right)$$  \hspace{1cm} (3.36)

where $a$ is computed from the equilibrium of forces on the cross-section, giving

$$a = \frac{\rho d}{\alpha_i} \frac{f_{pu,FRP}}{f'_c}$$  \hspace{1cm} (3.37)

Combining the value for $a$ (Eq. 3.37) with the nominal moment equation (Eq. 3.36) provides a combined form for the prediction of the nominal moment capacity:

$$M_n = \rho b d^2 f_{pu,FRP} \left( 1 - \frac{\rho}{2 \alpha_i} \frac{f_{pu,FRP}}{f'_c} \right)$$  \hspace{1cm} (3.38)

For FRP tendons which are distributed vertically over the depth of the section, the calculation of the nominal moment is slightly modified and it accounts for the exact strain.
in each layer of the tendons. Figure 3.6 shows the cross-section and the strain and stress conditions for vertically distributed FRP tendons.

The equations developed are for a T-section with the assumption that the neutral axis is in the flange, i.e., \( a < h_f \). The strain due to decompression of the concrete, \( \varepsilon_{d_i} \), and strains from elastic and nonelastic shortening, \( \varepsilon_{pr} \), have been neglected.

It is assumed that all tendons are prestressed at the same level, \( f_{p_{eFRP}} \), and thus the stress increase at failure in the bottom tendon may be defined as \( f_m = f_{p_{eFRP}} - f_{p_{eFRP}} \). The stress in each FRP tendon can be determined using a function which expresses the strain at any tendon level to the strain of the bottom tendon:

\[
f_i = f_{p_{eFRP}} + f_m \frac{d_i - c/d}{d - c/d} = f_{p_{eFRP}} + f_m \left( \frac{d_i/d - c/d}{1 - c/d} \right) \tag{3.39}
\]

where \( d_i \) is the depth of each individual tendon, and \( d \) is the depth of the bottom tendon.

Defining the initial prestressing ratio \( \xi = (f_{p_{eFRP}}/f_{p_{eFRP}}) \) and \( c = k d \), and assuming that \( \rho_i = (A_{p_{FRPi}}/d b) \) is the reinforcement ratio at level \( i \), it can be demonstrated that:

\[
k = \sqrt{\left( n \sum_{i=1}^{m} \rho_i \right)^2 + 2(1 - \xi) n \sum_{i=1}^{m} \rho_i \left( \xi + \frac{d_i}{d} (1 - \xi) \right) - n \sum_{i=1}^{m} \rho_i} \frac{1}{1 - \xi} \tag{3.40}
\]

where \( m \) is the number of layers of tendons.

The moment capacity is defined by taking the depth ratio of the tendons as \( \psi_i = (d_i/d) \) and assuming uniformly prestressed tendons, hence:

\[
M_n = b d^2 \sum_{i=1}^{m} \rho_i f_{p_{FRPi}} \left( \psi_i - \frac{\beta_i c}{2 d} \right) \tag{3.41}
\]
Table 3.2 Allowable FRP tendon stresses from ACI code

<table>
<thead>
<tr>
<th>Type of tendon</th>
<th>At jacking</th>
<th>At transfer</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP</td>
<td>0.65 $f_{PCFRP,u}$</td>
<td>0.60 $f_{PCFRP,u}$</td>
</tr>
<tr>
<td>AFRP</td>
<td>0.50 $f_{PAFRP,u}$</td>
<td>0.40 $f_{PAFRP,u}$</td>
</tr>
</tbody>
</table>

Table 3.3 Allowable FRP tendon stresses from CSA–S6-00 Standard

<table>
<thead>
<tr>
<th>Type of tendon</th>
<th>At jacking</th>
<th>At transfer</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP</td>
<td>0.70 $f_{PCFRP,u}$</td>
<td>0.65 $f_{PCFRP,u}$</td>
</tr>
<tr>
<td>AFRP</td>
<td>0.40 $f_{PAFRP,u}$</td>
<td>0.35 $f_{PAFRP,u}$</td>
</tr>
<tr>
<td>GFRP</td>
<td>0.30 $f_{PGFRP,u}$</td>
<td>0.25 $f_{PGFRP,u}$</td>
</tr>
</tbody>
</table>

If the FRP tendons through the depth of the member are prestressed at different levels, the nominal moment capacity is given as:

$$M_n = b d^2 \sum_{i=1}^{m} \left[ \rho_i \left( f_{pe,FRPi} + f_m \left( \frac{\psi_i - c/d}{1 - c/d} \right) \left( \psi_i - \frac{\beta_i c}{2d} \right) \right) \right]$$  \hspace{1cm} (3.42)

where $f_{pe,FRPi}$ is the effective prestress at the level $i$.

3.4.2 Allowable FRP tendon stresses at jacking and at transfer

Steel tendons are typically stressed to 85% of their yield stress or approximately 0.005 strain. Allowable stresses in FRP tendons are typically limited to 40% to 65% of their ultimate tensile strength due to stress-rupture limitations. Recommendations of ACI 440.4R-04 [2004] on the stress limitations for FRP tendons are presented in Table 3.2. In the CAN/CSA-S6-00 [2000], the values are slightly different and are shown in Table 3.3.

Based on creep-rupture characteristics of the FRP tendons, these stress limitations should not exceed 70% of the maximum stress that can be developed when tested using the anchorage specified by the manufacturer.

3.4.3 Correction of stress for harped FRP tendons

FRP tendons are linear elastic material up to failure. Consequently, draping or harping of the tendons results in a reduction of the tendon ultimate tensile strength. At the proximity of the harping or draping points, the increase of strain induced by the curvature of the
tendons reduces the ultimate tensile strength of the tendons. When the FRP tendons are harped or draped, the jacking force should be reduced to account for the localized stress increase. The stress increase due to harping can be evaluated by Eq. 3.43:

\[ f_h = \frac{E_f R_t}{R} \]  

(3.43)

where \( E_f \) is the elastic modulus of the fibres; \( R_t \) is the tendon radius; and \( R \) is the curvature radius of the harping saddle.

The combined stress in a tendon of cross-sectional area, \( A_{PPRP} \), at a harping saddle, due to the jacking load, \( P_o \), is given by Eq. 3.44 and should be less than the allowable stress values provided in Tables 3.2 and 3.3.

\[ f_{PPRP} = \frac{P_o}{A_{PPRP}} + \frac{E_f R_t}{R} \]  

(3.44)

### 3.5 Shear Strength

In this section, only those equations relevant for FRP reinforcement will be presented since the design equations for steel are well documented. Because of the differences between FRPs and steel, several issues need to be considered when using FRPs for shear reinforcement, such as:

- FRPs may have a relatively low elastic modulus,
- FRPs have a high tensile strength, but no yielding plateau,
- Tensile strength of the bent portion of an FRP bar is significantly lower than the straight portion,
- FRPs have lower dowel resistance and lower tensile strength in any direction other than that of the fibres,
- The bond characteristics of FRP stirrups may vary significantly from steel stirrups.

#### 3.5.1 Minimum amount of shear reinforcement

In general, even if the calculations indicate that there is no need for stirrups, a minimum amount of shear reinforcement, \( A_{v,\text{min}} \), has to be installed in prestressed concrete elements. In addition to supporting the longitudinal reinforcement bars, \( A_{v,\text{min}} \) minimizes
the possibility to have a brittle shear failure of the elements. The minimum amount of shear reinforcement is calculated from:

\[ A_{v, \text{min}} = 0.06 \sqrt{f'_c b_w s} \frac{s}{\sigma_v} \]  

(3.45)

where:

- \(f'_c\) : specified compressive strength of the concrete,
- \(b_w\) : web width of the element,
- \(s\) : longitudinal spacing of the stirrups,
- \(\sigma_v\) : effective stress of the stirrups defined by Eq. 3.46 and is the smaller of

\[
\begin{align*}
\sigma_v &= \left( \frac{0.05 r}{d_s} + 0.3 \right) f_{\text{FRP,bend}} \\
\sigma_v &= \frac{1.5}{E_{\text{vFRP}}} \varepsilon_v
\end{align*}
\]

(3.46)

where:

- \(r\) : radius of curvature of the bend of an FRP stirrup,
- \(d_s\) : distance from the top slab to the centroid of the bottom transverse FRP bars,
- \(f_{\text{FRP,bend}}\) : specified tensile strength of the straight portion of the FRP bent stirrup,
- \(E_{\text{vFRP}}\) : modulus of elasticity of FRP stirrups,
- \(\varepsilon_v\) : effective strain of the stirrups defined by Eq. 3.47.

\[
\varepsilon_v = 0.0001 \sqrt{f'_c \frac{\rho_s E_{\text{FRP}}}{\rho_{\text{vFRP}} f'_c} \left[ 1 + 2 \left( \frac{\sigma_N}{f'_c} \right) \right]} \leq 0.0025
\]

(3.47)

where:

- \(\rho_s\) : ratio of the cross-sectional area of the longitudinal FRP reinforcement to the effective cross-sectional area of the beam,
- \(\rho_{\text{vFRP}}\) : ratio of the total cross-sectional area of the legs of an FRP stirrup to the product of the width of the beam and the spacing of stirrups,
- \(\sigma_N\) : stress in the concrete due to axial loads.
3.5.2 Spacing limits of shear reinforcement

The maximum stirrup spacing depends on the shear stress intensity, \( \nu_f \), due to the factored loads. The shear stress is given by:

\[
\nu_f = \frac{V_f - \phi_{PPRP} V_{PPRP}}{b_w d_v} \leq 0.25 \phi_c f'_c
\]  

(3.48)

where:

- \( V_f \): factored shear force at a section,
- \( V_{PPRP} \): component in the direction of \( V_f \) of all the effective prestressing forces crossing the critical section factored by \( \phi_{PPRP} \), to be taken as positive if resisting \( V_f \),
- \( d_v \): effective shear depth calculated with Eq. 3.49.

\[
d_v = d - \frac{a}{2} \geq 0.72 h
\]  

(3.49)

If \( \nu_f < 0.10 \phi_c f'_c \), the maximum stirrup spacing is defined by Eq. 3.50:

\[
s_{\text{max}} = 0.75 d_v \leq 600 \text{ mm}
\]  

(3.50)

If \( \nu_f \geq 0.10 \phi_c f'_c \), the maximum stirrup spacing is defined with Eq. 3.51:

\[
s_{\text{max}} = 0.33 d_v \leq 300 \text{ mm}
\]  

(3.51)

From field installation considerations, the minimum stirrup spacing should be 75 mm.

3.5.3 Shear strength with FRP stirrups

The shear strength is needed to support the loads after the concrete web has cracked. Also, shear reinforcement is required to avoid the failure of the web in the shear zone, and leads instead, to a flexural failure which is more ductile. The modern theory on shear strength is based on the modified compression field theory [COLLINS and MITCHELL, 1997]. In addition to using the equilibrium equations, as in the truss model, this theory incorporates the strain compatibility equations and the material stress-strain behaviour of the stirrups and the concrete.

The factored shear strength, \( V_r \), is considered as the sum of the shear resistance provided by the concrete, \( V_c \), the shear resistance provided by the stirrups, \( V_{PPRP} \), and the shear
resistance provided by the vertical component of the prestressing force, \( V_{p_{FRP}} \) (Eq. 3.52).

\[
V_r = V_c + V_{p_{FRP}} + V_{p_{FRP}}
\]  

(3.52)

The contribution from the concrete is the most difficult value to evaluate. From CAN/CSA-S6-00 [2000], \( V_c \) is given by:

\[
V_c = \beta \phi_c \sqrt{f'_c} b_w d_v \sqrt{\frac{E_{long}}{E_s}}
\]  

(3.53)

where:

\( \beta \) : angle of inclination of the transverse reinforcement to the longitudinal axis of an element,

\( E_{long} \) : modulus of elasticity of longitudinal reinforcement,

\( E_s \) : modulus of elasticity of steel.

The parameter \( \beta \) depends on the angle of inclination, \( \theta \), and the principal tensile strain, \( \varepsilon_1 \). A relation exists between the concrete strains \( \varepsilon_1 \) and \( \varepsilon_x \). The strain \( \varepsilon_x \) is calculated at the level of the longitudinal reinforcement; i.e., at a distance \( d \) from the extreme compression fibre. The strain \( \varepsilon_x \) is given by Eq. 3.54 and is positive if in tension.

\[
\varepsilon_x = \frac{M_f}{d_w} + V_f - V_{p_{FRP}} + 0.5 N_f - A_{p_{FRP}} f_{p_{FRP}} \frac{f_{p_{FRP}}}{E_{FRP} A_{p_{FRP}} + E_{p_{FRP}} A_{p_{FRP}}} \leq 0.003
\]  

(3.54)

The values of \( \beta \) and \( \theta \) in function of \( \varepsilon_x \) and \( \nu_f/\phi_c f'_c \) are presented in CAN/CSA-S6-00 [2000].

For components with transverse reinforcement perpendicular to the longitudinal axis, \( V_{FRP} \) is calculated from:

\[
V_{FRP} = \frac{\phi_{FRP} 2 A_{FRP} \sigma_v d_v \cot \theta}{s}
\]  

(3.55)

For components with transverse reinforcement inclined at an angle, \( \alpha \), to the longitudinal axis, \( V_{FRP} \) is calculated from:

\[
V_{FRP} = \frac{\phi_{FRP} 2 A_{FRP} \sigma_v d_v (\cot \theta + \cot \alpha) \sin \alpha}{s}
\]  

(3.56)
where:

\( \sigma_e \): defined by Eq. 3.46.

The tensile strain in the FRP shear reinforcement is limited to 0.002.

### 3.6 Deflections and Crack-width

#### 3.6.1 Deflections

Deflections are checked for structural elements to ensure that the deflections under service loads will not affect the serviceability behaviour of the structure. The calculation of the deflections under loads is possible using the traditional mechanics of materials methods. These equations are valid considering that the entire concrete section is not cracked and reacts as a whole to the deformations. After the concrete has cracked, the classical equations are modified to predict the deflections.

The total short-term deflections, \( \Delta_{ti} \), that occur just after the prestressing force is released to the concrete is given by:

\[
\Delta_{ti} = \Delta_{po} + \Delta_g + \Delta_{ws} + \Delta_L
\]  
(3.57)

where:

\( \Delta_{po} \): instantaneous elastic deflection due to the eccentricity of the prestressing force,
\( \Delta_g \): deflection due to the girder weight,
\( \Delta_{ws} \): deflection from the permanent dead loads,
\( \Delta_L \): deflection from live loads.

Delayed deflection occurs with the ageing of the concrete. The concrete, under sustained compressive stress, will creep. So, the long-term deflection is composed of elastic deflections and a delayed deflection, \( \Delta_d \). The long-term deflection can be estimated from Eq. 3.58.

\[
\Delta_t = (\Delta_p + \Delta_g + \Delta_{ws} + \Delta_L) + \Delta_d
\]  
(3.58)

where:

\( \Delta_p \): deflection due to the eccentricity of the effective prestressing force after losses,
Δₜ : delayed deflection from the creep of the concrete.

The delayed deflection can be calculated from the creep coefficient method with Eq. 3.59.

\[
\Delta_d = [0.5(\Delta_{pe} + \Delta_p) + \Delta_g] C_{cr}(t_i) + \sum \Delta_{wst} C_{cr}(t_j)
\]  

(3.59)

where \( C_{cr}(t) \) is defined by Eq. 3.20.

Depending if the element is cracked or not, the moment of inertia is different. If the element has not cracked, the gross moment of inertia, \( I_g \), is used in the calculation. However, if the deflection calculation following crack formation is required, methods that take into account the softening effect that cracking has on concrete members need to be used. This can be accomplished using a modified effective moment of inertia, \( I_e \), and given by Eq. 3.60.

\[
I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \beta_d I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq I_g
\]  

(3.60)

where:

\( M_{cr} \): cracking moment,
\( M_a \): maximum moment in a element at which the deflection is being computed,
\( \beta_d \): factor to soften \( I_e \) provided by Eq. 3.61,
\( I_g \): gross moment of inertia,
\( I_{cr} \): cracked moment of inertia, which for a rectangular section or a flanged section with \( k d < h_f \), can be calculated from Eq. 3.62.

\[
\beta_d = 0.5 \left[\frac{E_{pfrb}}{E_s} + 1\right]
\]  

(3.61)

\[
I_{cr} = \frac{b(kd)^3}{3} + n A_{pfrb}(d - k d)^2
\]  

(3.62)

where \( k \) is defined in Eq. 3.40.

The deflection is not only due to the change of the effective moment of inertia \( I_e \), but also due to the change of the eccentricity of the prestressing force after cracking. It has been suggested to consider the change in the effective eccentricity by finding the effective neutral axis location using Eq. 3.63.

\[
y_{eff} = \left(\frac{M_{cr}}{M_a}\right)^2 y_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^2\right] y_{cr}
\]  

(3.63)
where:

- $y_g$: distance from the extreme compression fibre to the centroid of the gross section,
- $y_{cr}$: distance from the extreme compression fibre to the neutral axis of the cracked section (that is, $kd$).

### 3.6.2 Crack-width

From CAN/CSA-S6-00 [2000], when the maximum tensile strain in the FRP reinforcement under full service loads exceeds 0.0015, cross-sections of the element in maximum positive and negative moment regions shall be proportioned so that the crack-width does not exceed 0.5 mm for an element subject to aggressive environments otherwise 0.7 mm, where the crack width is given by:

$$w_{cr} = \frac{f_{frp}}{E_{frp}} \frac{h_2}{h_1} k_b \sqrt{d_c^2 + \left(\frac{s}{2}\right)^2} \quad (3.64)$$

where the value of $k_b$ shall be determined experimentally, but in the absence of test data, if may be taken as 0.8 for sand-coated and 1.0 for deformed bars. In the calculation of $d_c$, the clear cover shall not be taken greater than 50 mm. $h_1$ is the distance from the centroid of the tension reinforcement to the neutral axis and $h_2$ is the distance from the extreme flexural tension surface to the neutral axis.

The material properties for the concrete, the steel reinforcement bars and prestressing tendons and the FRP bars were presented in this chapter and were characterized using the equations provided by standards, codes and design manuals. Also in this chapter, the methodology to evaluate the loss of prestress in the tendons was described. Then, from the theoretical equations, the development to calculate the flexural and shear strength were given. Finally, the deflections and crack-width theoretical developments were presented in the last part of this chapter. The next chapter gives details about the characteristics of the panels under investigation.
Chapter 4

Experimental Program

4.1 Introduction

This chapter presents the general characteristics of the panels under investigation. These experimental panels had the exact same dimensions as the panels that were replaced on the Jacques-Cartier Bridge in Montreal in the summer 2001–02. Then, the material properties used in the experimental panels are presented. Tests on each component of the panels were performed and used in the numerical models and the analytical calculations. Furthermore, few anchorage systems for the prestressing of the CFRP tendons to be able to achieve the design tensile strength of the tendons during prestressing were investigated. In the last part of this chapter, the test set-up for the loading of the experimental panels and all the instrumentation installed to record the strains and deflections under the loadings are described.

4.2 Panel Description

In this investigation, four full-scale bridge deck panels were fabricated, instrumented and tested. Two panels were of a double-tee (DT) cross-section, and the other two were a single-tee (ST) cross-section. The elements were 7.418 m long, by 2.99 m and 1.5 m wide for the two DT and two ST sections, respectively. The depth of the stems varied from 0.5 m at the supports to 0.8 m at mid-span (Fig. 4.1).

The reinforcements, particularly the prestressing, of the four experimental panels were designed to resist the axle load of 163 kN prescribed in CAN/CSA-S6-00 [2000], at service and ultimate load. The theoretical development to design for the flexural capacity
of the concrete elements prestressed with CFRP tendons was based on the methodology given by BURKE and DOLAN [2001]. For each configuration (DT or ST), one experimental panel was built with steel tendons and reinforcement bars, while the other was reinforced and prestressed entirely with FRPs.

### 4.2.1 Steel panels description

For the two elements with steel, four 15 mm diameter draped steel prestressing tendons were used in each stem to resist the flexural moments. Three hold-down devices were used to make the draped shape (Fig. 4.2). In addition, the reinforcement includes 10M stirrups in the stems for shear, and two beds of 15M bars in each direction in the slab. The spacing of the stirrups varies along the element. In the DT steel panel, two 15M passive reinforcement bars were placed in each stem to maintain the stirrups in place (Figs. 4.3, 4.4, 4.5, 4.6 & 4.7).
Figure 4.2  Hold-down device to make the draped shape of the tendons

Figure 4.3  Layout of reinforcement and tendons for DT steel panel

Figure 4.4  Layout of reinforcement and tendons for ST steel panel
Figure 4.5  **ST** steel panel reinforcement position
Figure 4.6  Cross-section details and prestressing forces for DT steel panel

Figure 4.7  Cross-section details and prestressing forces for ST steel panel

In Figures 4.6 and 4.7, the prestressing force applied, after losses, on each tendon is given for the steel panels.

4.2.2 FRP panels description

For the two panels with FRPs, each 9.5 mm diameter GFRP stirrup consisted of two "C" shaped parts and provided in this shape by the manufacturer. Two beds of GFRP straight bars of 16 mm diameter were used for the slab reinforcement. All GFRP reinforcements were supplied by Hughes Brothers Inc. and are distributed under the trade name of Aslan 100 GFRP. The GFRP bars and stirrups were assembled with tie-wraps to provide completely steel-free panels. In the DT FRP panel, two 16 mm diameter GFRP passive
reinforcement bars were placed at the neutral axis in each stem to maintain the stirrups in place. Five and six prestressed 10 mm diameter draped indented Leadline CFRP tendons, produced by the Mitsubishi Chemical Corporation, were used in each stem to resist the flexural moments in the DT and ST panels, respectively. These panels also had the same three hold-down devices as the steel panels (Figs. 4.8, 4.9, 4.10, 4.11 & 4.12).

In Figs. 4.11 and 4.12, the prestressing force applied, after losses, on each tendon is given for the FRP panels. When a value of 0 kN is indicated, this means that a slippage of the anchorage system had occurred. However, the tendon in question did not rupture, so it was left there as a passive reinforcement. The symbol “s.o.” means that the tendon had failed during prestressing, ruptured in many pieces and was not replaced. The pieces were removed from the mould before pouring the concrete.
Figure 4.10  **ST** FRP panel reinforcement position

![Image of FRP panel reinforcement]

Figure 4.11  Cross-section details and prestressing forces for **DT** FRP panel

<table>
<thead>
<tr>
<th></th>
<th>STEM 1</th>
<th>STEM 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>66</td>
<td>89</td>
</tr>
<tr>
<td>s.o.</td>
<td>70</td>
<td>89</td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>82</td>
</tr>
<tr>
<td>56</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>Total</td>
</tr>
<tr>
<td>262</td>
<td>331</td>
<td></td>
</tr>
</tbody>
</table>
Figure 4.12 Cross-section details and prestressing forces for ST FRP panel

The chronological order of the panel construction helps to explain the variation in the prestressing force in each panel. The first specimen fabricated was the DT steel panel and the target prestressing force was reached without problems (Fig. 4.6). Then, the DT FRP panel was built and, as shown in Fig. 4.11, 62% and 52% less prestressing force in stem 1 and stem 2, respectively, was obtained compared to the DT steel panel. It happened that the second tendon to be prestressed slipped out of the anchorage system when it reached the target load of 95 kN and, ruptured into multiple pieces. It was decided to proceed with the other tendons with caution; that is, to stop prestressing at lower levels than designed when suspicious noises were detected.

To be able to compare an FRP panel to a steel panel, the ST FRP panel was the third one fabricated and the maximum prestressing force was attempted. But still, the prestressing force was 71% less than that for the the DT steel panel. For the last specimen, the ST steel panel, the prestressing force was adjusted to the same level as in the ST FRP panel. For the ST panels, a difference of only 7% was present, although comparisons between these two panels will be more accurate (Figs. 4.7 & 4.12).

4.3 Material Properties

To make comparisons between the experimental results and the analytical calculations and the predictions from the numerical models, the material properties have to be well characterized. The important properties of the concrete, steel reinforcement bars, steel
prestressing tendons, CFRP prestressing tendons and GFRP reinforcement bars and stirrups were find using standardized tests for each material.

### 4.3.1 Concrete

The compressive strength design for the HPC was established at 60 MPa. The concrete for each panel was provided by a local supplier. Table 4.1 shows the compressive strength at the transfer of the prestressing, at 28 days and at the time of testing and also, the percentage of air and the slump at casting. The evolution of the concrete compressive strength is shown in Appendix A. The prestressing force was transferred once the concrete compressive strength had reached 25 MPa. The variability in the supplied concrete mostly affected the serviceability behaviour of the panels. Table 4.2 presents the mix design for 1 cubic meter of concrete for the DT steel panel while Table 4.3 presents the concrete used for the ST steel, DT FRP and ST FRP panels. Both mix designs are similar; except that 620 ml more air entraining admixture was added for the ST steel and FRP panels.

<table>
<thead>
<tr>
<th>Panels</th>
<th>Compressive strength [MPa]</th>
<th>Tensile strength [MPa]</th>
<th>Air [%]</th>
<th>Slump [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>At transfer 28 days At testing</td>
<td>At testing</td>
<td>At casting</td>
<td>At casting</td>
</tr>
<tr>
<td>DT Steel</td>
<td>-32  —</td>
<td>-74</td>
<td>—</td>
<td>7</td>
</tr>
<tr>
<td>DT FRP</td>
<td>-28  —</td>
<td>-31</td>
<td>—</td>
<td>7</td>
</tr>
<tr>
<td>ST Steel</td>
<td>-36  -51</td>
<td>-58</td>
<td>4.8</td>
<td>8</td>
</tr>
<tr>
<td>ST FRP</td>
<td>-30  -42</td>
<td>-44</td>
<td>3.8</td>
<td>7</td>
</tr>
</tbody>
</table>

¹ 1500 ml of superplasticizer was added to increase the slump at 145 mm
² Ice was added during transportation to keep the concrete under 20°C

**Table 4.2 Concrete mix design for the DT steel panel**

<table>
<thead>
<tr>
<th>Properties</th>
<th>Quantities</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement type 10 SF</td>
<td>450 kg/m³</td>
</tr>
<tr>
<td>Water/cement ratio</td>
<td>0.30</td>
</tr>
<tr>
<td>Aggregates (5–14 mm)</td>
<td>990 kg/m³</td>
</tr>
<tr>
<td>Sand (0–5 mm)</td>
<td>760 kg/m³</td>
</tr>
<tr>
<td>Superplasticizer (PS1248)¹</td>
<td>4500 ml</td>
</tr>
<tr>
<td>Air entraining admixture (Micro-air)¹</td>
<td>380 ml</td>
</tr>
<tr>
<td>Set retarding admixture (100XR)¹</td>
<td>1000 ml</td>
</tr>
</tbody>
</table>

¹ Master Builders Technologies
### Table 4.3  Concrete mix design for the ST steel, DT FRP and ST FRP panels

<table>
<thead>
<tr>
<th>Properties</th>
<th>Quantities</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement type 10 SF</td>
<td>455 kg/m³</td>
</tr>
<tr>
<td>Water/cement ratio</td>
<td>0.32</td>
</tr>
<tr>
<td>Aggregates (5–14 mm)</td>
<td>1000 kg/m³</td>
</tr>
<tr>
<td>Sand (0–5 mm)</td>
<td>780 kg/m³</td>
</tr>
<tr>
<td>Superplasticizer (Eucon37)</td>
<td>4583 ml</td>
</tr>
<tr>
<td>Air entraining admixture (Micro-air)</td>
<td>1000 ml</td>
</tr>
<tr>
<td>Set retarding admixture (EuconDX)</td>
<td>1000 ml</td>
</tr>
</tbody>
</table>

1. Master Builders Technologies  
2. Euclid

### 4.3.2  Steel reinforcement bars

No tensile tests were done on the steel stirrups and reinforcing bars. Their properties were assumed to be as specified by the manufacturers.

### 4.3.3  Steel prestressing tendons

Tables 4.4 and 4.5 present the results from six tensile tests performed on 15 mm diameter steel prestressing tendons. As can be seen in Table 4.4, the first three tendons were used in the DT steel panel and their average modulus of elasticity, $E_{ps}$, was 213,300 MPa. Moreover, in Table 4.5, the three other tendons were used in the ST steel panel and, in this case, the average modulus of elasticity was 203,900 MPa.

### Table 4.4  Tensile test results on three steel prestressing tendons used in the DT steel panel

<table>
<thead>
<tr>
<th>Test</th>
<th>$f_{psu}$</th>
<th>$\varepsilon_{psu}$</th>
<th>$E_{ps}$</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[MPa]</td>
<td>[%]</td>
<td>[MPa]</td>
<td></td>
</tr>
<tr>
<td>1–02</td>
<td>1856</td>
<td>6.56</td>
<td>*209 500</td>
<td>no failure</td>
</tr>
<tr>
<td>2–02</td>
<td>1838</td>
<td>3.21</td>
<td>*217 100</td>
<td>strand broke in anchorage</td>
</tr>
<tr>
<td>3–02</td>
<td>1814</td>
<td>2.06</td>
<td>*213 400</td>
<td>strand broke in anchorage</td>
</tr>
<tr>
<td></td>
<td>1836</td>
<td>3.94</td>
<td>213 300</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1860</td>
<td>4.15</td>
<td>200 000</td>
<td></td>
</tr>
</tbody>
</table>

$E_{ps}$ was find by making a linear trendline with the points
Table 4.5 Tensile test results on three steel prestressing tendons used in the ST steel panel

<table>
<thead>
<tr>
<th>Test</th>
<th>$f_{psu}$ [MPa]</th>
<th>$\varepsilon_{psu}$ [%]</th>
<th>$E_{ps}$ [MPa]</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>4-03</td>
<td>1886</td>
<td>5.49</td>
<td>*205 100</td>
<td>slippage in the anchorage</td>
</tr>
<tr>
<td>5-03</td>
<td>1812</td>
<td>4.32</td>
<td>*202 600</td>
<td>strand broke in anchorage</td>
</tr>
<tr>
<td>6-03</td>
<td>1747</td>
<td>1.65</td>
<td>*204 000</td>
<td>anchorage failure</td>
</tr>
<tr>
<td>Average</td>
<td>1815</td>
<td>3.82</td>
<td>203 900</td>
<td></td>
</tr>
<tr>
<td>Manufacturer</td>
<td>1860</td>
<td>4.15</td>
<td>200 000</td>
<td></td>
</tr>
</tbody>
</table>

* $E_{ps}$ was find by making a linear trendline with the points

Table 4.6 Tensile test results on three CFRP prestressing tendons

<table>
<thead>
<tr>
<th>Test</th>
<th>$f_{PCFPR}u$ [MPa]</th>
<th>$\varepsilon_{PCFPR}u$ [%]</th>
<th>$E_{PCFPR}$ [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2762</td>
<td>1.58</td>
<td>*164 400</td>
</tr>
<tr>
<td>2</td>
<td>2583</td>
<td>1.59</td>
<td>*162 600</td>
</tr>
<tr>
<td>3</td>
<td>2376</td>
<td>1.54</td>
<td>*155 800</td>
</tr>
<tr>
<td>Average</td>
<td>2574</td>
<td>1.57</td>
<td>160 900</td>
</tr>
<tr>
<td>Manufacturer</td>
<td>2255</td>
<td>1.6</td>
<td>147 000</td>
</tr>
</tbody>
</table>

* $E_{PCFPR}$ was find by making a linear trendline with the points

It is obvious that there were two different batches of steel prestressing tendon based on their different modulus of elasticity. The first batch was provided by the precast plant in Longueuil in 2002 and was used in the DT steel panel. The second batch was purchased by the University of Sherbrooke in 2003 and was used in the ST steel panel.

4.3.4 CFRP prestressing tendons

Table 4.6 shows the results of three tensile tests on 10 mm diameter CFRP Leadline tendons. The average ultimate tensile strength, $f_{PCFPRu,ave}$, was 2574 MPa, the average ultimate strain, $\varepsilon_{PCFPRu,ave}$, was 1.57% and average modulus of elasticity, $E_{PCFPR,ave}$, was 160,900 MPa from the three tests. The manufacturer specification sheet gives 2255 MPa and 1.6% for the maximum tensile strength and ultimate strain, respectively. The maximum tensile strength provided by the manufacturer is 14% lower than what was recorded
in our tensile tests. Since the ultimate strain is almost the same as the manufacturer, this means that the stiffness of the CFRP tendons are higher by 16% compared with the manufacturer. For the analytical calculations and the numerical models, the average ultimate stress and strain from the tensile tests were employed.

4.3.5 GFRP reinforcement bars

Table 4.7 presents the results of three tensile tests on 16 mm diameter GFRP bars. The results for the average ultimate tensile strength, \( f_{\text{GFRP,ave}} \), was 676 MPa, the average ultimate strain, \( \varepsilon_{\text{GFRP,ave}} \), was 1.57% and average modulus of elasticity, \( E_{\text{GFRP,ave}} \), was 43,000 MPa for the three tests. These values are in good agreement with values provided by the manufacturer. The specification sheet gives a guaranteed ultimate tensile strength of 655 MPa and an ultimate strain of 1.61%.

<table>
<thead>
<tr>
<th>Test</th>
<th>( f_{\text{GFRP}} ) [MPa]</th>
<th>( \varepsilon_{\text{GFRP}} ) [%]</th>
<th>( E_{\text{GFRP}} ) [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>734</td>
<td>1.75</td>
<td>*41700</td>
</tr>
<tr>
<td>2</td>
<td>660</td>
<td>1.50</td>
<td>*44100</td>
</tr>
<tr>
<td>3</td>
<td>634</td>
<td>1.46</td>
<td>*43300</td>
</tr>
<tr>
<td>Average</td>
<td>676</td>
<td>1.57</td>
<td>43000</td>
</tr>
<tr>
<td>Manufacturer</td>
<td>655</td>
<td>1.61</td>
<td>40800</td>
</tr>
</tbody>
</table>

*\( E_{\text{GFRP}} \) was find by making a linear trendline with the points

Tensile tests were not performed on the GFRP stirrups. Their mechanical properties were taken as in the specification sheet provided by the manufacturer. The modulus of elasticity was taken as 40,800 MPa and the tensile strength as 760 MPa for the 9.5 mm diameter GFRP stirrups.

4.4 Prestressing Anchorage Devices

Although reliable systems are readily available for prestressing steel tendons, the situation is different for CFRP tendons [RETA TAH And SHRIVE, 2003a]. In this research project, three anchorage systems were tested. The main objective of each system was to develop
Figure 4.13  Glue anchor system

the full tensile strength of the CFRP tendon without having slippage or failure of the tendon in the anchorage device.

4.4.1  Glued anchorage

The first system tested consisted of a thick-walled steel tube in which the CFRP tendon was bonded using an epoxy-gel adhesive (Fig. 4.13). The bond strength of the epoxy on the CFRP was increased by the indentation of the tendon, which provides some mechanical load transfer. The inner diameter of the tube was reduced at the extremity to provide a more effective mechanical load transfer. This system was not sufficiently reliable because of the difficulty to completely fill the tube without leaving voids.

4.4.2  Wedges-type anchorages

The second system tested was derived from the tapered sleeve/wedges system used for steel cables. It consisted of an interior tapered steel sleeve with four aluminum-tapered wedges (Fig. 4.14). For about one third of the length of the sleeve, the interior tapering angle was less than the angle of the wedges in order to reduce the transverse stresses on the tendon at the extremity of the anchorage (Fig. 4.15). Also, the wedges were fabricated in aluminum, a relatively soft metal, to minimize damage on the CFRP tendon. In the laboratory tests, this system was able to transfer about 80% of the specified tension load capacity prior
Figure 4.14 Steel sleeve/aluminium wedges anchor system

to the failure of the CFRP tendon near the anchorage. During the fabrication of the experimental panels, this anchorage system was easy to install, but failed by slippage at 55% or less of the specified tension load capacity.

The third CFRP anchorage system tested, and used for the prestressing, was developed by Sayed-Abdel and Shrive [1998]. It consisted of an interior low-angle-tapered steel sleeve, four or three low-angle-tapered steel wedges for the DT and ST FRP panel, respectively, and a thin inner copper sleeve (Fig. 4.16). The taper angle of the wedges was greater than the taper angle of the steel sleeve. Thus, upon insertion of the wedges into the sleeve, the wider end of the wedges form a contact before the narrower end in order to reduce the transverse stress on the CFRP tendon at this critical section. Figure 4.17 shows the details of the sleeve and wedges of this system. The inner copper sleeve was comprised of two parts placed in the interior channel of the wedges. The outer diameter of the inner sleeve matched the diameter of the wedge channel, while its interior diameter matched the CFRP tendon. The tendon was held by the sandblasted surface of the inner copper sleeve, which was placed to diffuse the stresses.

The laboratory tests on the low-angle steel sleeve/wedges anchorage were successful as the full specified tension load capacity was obtained prior to the failure of the CFRP tendon. Table 4.8 presents the results from anchorage tests on four piece wedges with a 2.09° outside angle. Furthermore, to try to achieve equal stresses around the tendon, the results from a modified anchor system with three wedges, also with a 2.09° outside angle,
is presented in Table 4.9. This modification did not increase the maximum prestressing force since the four piece wedges already developed the full tensile strength of the CFRP tendon. However, the installation of the three piece wedges on the tendon was easier than the four piece wedges. The test set-up to verify the capacity of the anchor system for the prestressing of the CFRP tendons is illustrated in Fig. 4.18. Also, as shown in Fig. 4.18, when the anchor system develops the full tensile strength of the CFRP tendon, the tendon fails in multiple pieces and releases a huge amount of energy. The anchorage system was also tested with draped CFRP tendons using hold-down devices to simulate the on-field prestressing of the experimental FRP panels (Fig. 4.19). Despite this success, this anchorage device occasionally failed by slippage at less than 55% of the specified tension load capacity during the prestressing of the DT FRP full-scale panel. The target prestressing load was 60% of the ultimate tensile strength of the CFRP tendon. The failure was partly attributed to the damage on the indentation of the CFRP tendon due to the numerous trials in prestressing them, partly due to the difficulties to achieve equal stresses on the four wedges of the anchorage and also by the increase of stress at the draping points due to the hold-down devices. To overcome the failure problem cause by the draping points, high density plastic hold-down devices should be used instead of
conventional steel roller (Fig. 4.20). Plastic hold-down devices have been used with success for the prestressing of CFRP tendons on Taylor Bridge in Winnipeg in 1997 [RIZKALLA et al., 1998].
Figure 4.17  Details for steel sleeve/wedges anchor system
Figure 4.18 Set-up for anchorage tests on steel sleeve/wedges anchor system

Figure 4.19 Set-up for draped CFRP tendon tensile tests

Figure 4.20 Plastic hold-down devices
### Table 4.8 Anchorage Test Results on 4 Piece Wedges

<table>
<thead>
<tr>
<th>Test</th>
<th>Maximum load [kN]</th>
<th>Tendon used</th>
<th>Copper sleeve preparation</th>
<th>Type of failure</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>1–4w</td>
<td>131.7</td>
<td>new</td>
<td>no copper sleeve anchor extremity</td>
<td>failure at the anchor extremity</td>
<td></td>
</tr>
<tr>
<td>2–4w</td>
<td>108.5</td>
<td>new</td>
<td>no copper sleeve, clogging up the sleeve</td>
<td>slippage</td>
<td></td>
</tr>
<tr>
<td>3–4w</td>
<td>139.0</td>
<td>new</td>
<td>no copper sleeve anchor extremity</td>
<td>failure at the anchor extremity</td>
<td></td>
</tr>
<tr>
<td>4–4w</td>
<td>161.9</td>
<td>use</td>
<td>end A: no copper sleeve end B: glued and sandblast sandblast only</td>
<td>failure at the anchor extremity</td>
<td>slippage</td>
</tr>
<tr>
<td>5–4w</td>
<td>175.7</td>
<td>use</td>
<td>glued and sandblast</td>
<td>failure at the anchor extremity</td>
<td>load increased to 111.2 kN, then maintained 10 min at 97.9 kN</td>
</tr>
<tr>
<td>6–4w</td>
<td>162.4</td>
<td>new</td>
<td>glued and sandblast</td>
<td>failure at the anchor extremity</td>
<td>load increased to 111.2 kN, then maintained 10 min at 97.9 kN</td>
</tr>
<tr>
<td>7–4w</td>
<td>155.7</td>
<td>use: short</td>
<td>glued and sandblast</td>
<td>failure of tendon at mid-length</td>
<td>load increased to 111.2 kN, then maintained 5 min at 97.9 kN</td>
</tr>
<tr>
<td>8–4w</td>
<td>166.6</td>
<td>use: short</td>
<td>glued and sandblast</td>
<td>failure of tendon at mid-length</td>
<td>load increased to 111.2 kN, then maintained 5 min at 97.9 kN</td>
</tr>
<tr>
<td>9–4w</td>
<td>170.8</td>
<td>use</td>
<td>glued and sandblast</td>
<td>failure of tendon at mid-length</td>
<td>load increased to 111.2 kN, then maintained 5 min at 97.9 kN</td>
</tr>
<tr>
<td>10–4w</td>
<td>163.9</td>
<td>new</td>
<td>glued and sandblast</td>
<td>failure of tendon at mid-length</td>
<td>load increased to 111.2 kN, then maintained 5 min at 97.9 kN</td>
</tr>
<tr>
<td>11–4w</td>
<td>131.0</td>
<td>new: long</td>
<td>glued only</td>
<td>failure of tendon at mid-length</td>
<td>load increased to 111.2 kN, then maintained 5 min at 97.9 kN</td>
</tr>
<tr>
<td>12–4w</td>
<td>154.6</td>
<td>use: 600 mm</td>
<td>glued and sandblast</td>
<td>failure of tendon at mid-length</td>
<td>tendon slipped at one end by 15 mm</td>
</tr>
<tr>
<td>13–4w</td>
<td>153.9</td>
<td>use: 600 mm</td>
<td>glued and sandblast</td>
<td>failure of the anchor extremity longitudinal splitting</td>
<td>tendon slipped at one end by 20 mm</td>
</tr>
<tr>
<td>14–4w</td>
<td>139.6</td>
<td>use: 600 mm</td>
<td>glued and sandblast</td>
<td>failure of tendon at mid-length</td>
<td>tendon slipped at one end by 15 mm; beginning of fibres rupture at 66.7 kN</td>
</tr>
<tr>
<td>15–4w</td>
<td>142.9</td>
<td>use: 600 mm</td>
<td>glued and sandblast</td>
<td>failure of tendon at mid-length</td>
<td>tendon slipped at one end by 15 mm; beginning of fibres rupture at 66.7 kN</td>
</tr>
<tr>
<td>16–4w</td>
<td>140.8</td>
<td>use: 600 mm</td>
<td>glued and sandblast</td>
<td>failure of tendon at mid-length</td>
<td></td>
</tr>
<tr>
<td>17–4w</td>
<td>&lt; 8.9</td>
<td>use: 600 mm alu 13.3 kN</td>
<td>glued and sandblast</td>
<td>slippage</td>
<td></td>
</tr>
<tr>
<td>18–4w</td>
<td>151.7</td>
<td>use: 600 mm alu 13.3 kN</td>
<td>glued and sandblast</td>
<td>failure of tendon at mid-length</td>
<td></td>
</tr>
<tr>
<td>19–4w</td>
<td>151.2</td>
<td>use: 600 mm alu 13.3 kN</td>
<td>glued and sandblast</td>
<td>failure of tendon at mid-length</td>
<td></td>
</tr>
<tr>
<td>20–4w</td>
<td>134.6</td>
<td>use: 600 mm alu 13.3 kN</td>
<td>glued and sandblast</td>
<td>failure of tendon at mid-length</td>
<td></td>
</tr>
<tr>
<td>21–4w</td>
<td>161.0</td>
<td>use: 600 mm alu 13.3 kN</td>
<td>glued and sandblast</td>
<td>failure of tendon at mid-length</td>
<td></td>
</tr>
</tbody>
</table>

Average 155.12

Target 94.93

1 test with the aluminium wedges up to 13.3 kN, then changed with the steel wedges up to failure.
2 average is for the sandblast and glued inner copper sleeve only.
3 is the target load for the prestressing in field.
Table 4.9 Anchorage Test Results on 3 Piece Wedges

<table>
<thead>
<tr>
<th>Test</th>
<th>Maximum load [kN]</th>
<th>Tendon used</th>
<th>Copper sleeve preparation</th>
<th>Type of failure</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>1–3w</td>
<td>147.9</td>
<td>new: long</td>
<td>sandblast only</td>
<td>failure of tendon</td>
<td>unlubricated anchor</td>
</tr>
<tr>
<td>2–3w</td>
<td>1.1</td>
<td>new: long</td>
<td>none</td>
<td>slippage</td>
<td>wedges lubricated</td>
</tr>
<tr>
<td>3–3w</td>
<td>32.9</td>
<td>new: long</td>
<td>glued only</td>
<td>slippage</td>
<td></td>
</tr>
<tr>
<td>4–3w</td>
<td>148.3</td>
<td>use: 600 mm</td>
<td>glued and sandblast</td>
<td>failure of tendon</td>
<td>tendon slipped at one end by 10 mm</td>
</tr>
<tr>
<td>5–3w</td>
<td>158.4</td>
<td>use: 600 mm</td>
<td>glued and sandblast</td>
<td>failure at the anchor extremity</td>
<td>tendon slipped at one end by 10 mm</td>
</tr>
<tr>
<td>6–3w</td>
<td>151.7</td>
<td>use: 600 mm</td>
<td>glued and sandblast</td>
<td>failure of tendon</td>
<td>tendon slipped at one end by 10 mm</td>
</tr>
<tr>
<td>7–3w</td>
<td>138.6</td>
<td>use: 600 mm</td>
<td>glued and sandblast</td>
<td>failure of tendon</td>
<td>beginning of fibres rupture at 66.7 kN</td>
</tr>
<tr>
<td>8–3w</td>
<td>148.8</td>
<td>use: 600 mm</td>
<td>glued and sandblast</td>
<td>longitudinal splitting</td>
<td>tendon slipped at one end by 10 mm</td>
</tr>
<tr>
<td>9–3w</td>
<td>140.3</td>
<td>use: hold-down device 5°</td>
<td>glued and sandblast</td>
<td>failure of tendon</td>
<td></td>
</tr>
<tr>
<td>10–3w</td>
<td>165.9</td>
<td>use: hold-down device 5°</td>
<td>glued and sandblast</td>
<td>failure of tendon</td>
<td>in 3 pieces</td>
</tr>
</tbody>
</table>

Average 149.1

Target 94.9

1 average is for the sandblast and glued inner copper sleeve only.
2 is the target load for the prestressing in field.

The development of a reliable and easy to use prestressing anchorage system for CFRP tendon is still ongoing. Current research is examining the slippage problem observed during prestressing of the CFRP tendons for large-scale panels.

4.5 Test Set-up and Instrumentation

The full-scale DT panels were too large to enter our laboratory for testing. As a result, a specific outdoor loading set-up was designed and constructed (Fig. 4.21). For the ST panels, a more conventional laboratory loading set-up was built (Fig. 4.22). In both test set-ups, one end was simply supported and, at the other end, longitudinal displacements were allowed to avoid creation of undesirable axial stress in the panels. All supports allowed rotation of the panels at both extremities. Hydraulic jacks were used to apply the loads on the panels.
Figure 4.21  Test set-up for DT panels

Figure 4.22  Test set-up for ST panels
Electrical resistive strain gauges were placed on the prestressing tendons, stirrups, slab reinforcing bars and on the concrete (Figs. 4.23, 4.24 & 4.25). They were installed to investigate flexural behaviour at the mid-span, as well as that at the critical shear sections and an intermediate zone where shear, flexure and prestressing interact. The strain gauge positions for the ST FRP panel are illustrated in Fig. 4.26. For the other panels, the strain gauge positions are presented in Appendix C. Three linear variable differential transducers (LVDTs) were installed on the stem to measure vertical deflections at mid-span and at 1.7 m from the supports (Fig. 4.27). The displacement of the supports was monitored with dial indicators and the values are used for the calculation of the effective vertical deflection (Fig. 4.28). Figure 4.29 shows the horizontal LVDT installed at mid-span between the two layers of tendons to record the strain at this level. A computer driven data acquisition system simultaneously recorded all channels at each loading step.
Figure 4.26  Stem strain gauges positions for the ST FRP Panel
The applied load patterns were representative of a double axle of the design load truck from CAN/CSA-S6-00 [2000]. The loading set-up applied an equal force on two points, spaced at 1.2 m, located over the stem. The loading frame was moved from position to position along the elements to apply loads in regions of high shear or flexure. Figure 4.30 illustrates the three loading positions during testing. In the first test, the load was centred and gave the most critical flexural condition. Then, the load was moved to an intermediate zone at a third point to apply an intermediate loading. Afterwards, the load was moved to the other side near the roller support to simulate a critical shear loading. For the first three tests, the load was increased until the first cracks appeared, then unloaded to study the
serviceability behaviour. This order of testing is valid for the DT steel, ST steel and ST FRP panels. For the DT FRP panel, the order of testing was different starting with the intermediate loading followed by the central loading and continued with the shear loading positions. The order of loading influences the results and can be seen in Section 6.3 where the test results are presented. The panels are permanently damaged from the previous loading positions and it is for this reason that the curve generally never starts at a zero deflection value. Finally, to investigate the ultimate capacity of the elements, the last test performed on each experimental panel was a central loading until the panels failed or exhibited excessive deflections.

In this chapter, the dimensions and the reinforcement layout for the steel and FRP panels were described. The total prestressing force after all losses was also presented for each panel. The prestressing force is a major factor which influences the behaviour of the panels. Moreover, the material properties of the concrete, steel reinforcement bars, steel prestressing tendons, CFRP prestressing tendons and GFRP reinforcement bars were given. Also, the results were presented for the three anchorage systems for the CFRP tendon tested in this research project. The pull-out tests demonstrated that the steel sleeve/wedges anchor system with a thin inner copper sleeve performed the best. Even if this anchor system developed more than 95% of the ultimate tensile strength of the CFRP tendon in laboratory, in the field less than 55% was reached. Finally, the test set-up for the different loading positions and the instrumentation on the different components were described. The next chapter presents the procedure to model the experimental panels in two finite element softwares.
Figure 4.30  Three loading configurations on panels
Chapter 5

Numerical Models

5.1 Introduction

The use of CFRP as internal prestressing for concrete structures is still considered as a new application. To establish accurate design recommendations for codes, laboratory tests are required on specimens to validate the results obtained from the proposed equations. Laboratory tests are quite expensive to realize. However, a good alternative to reduce the number of specimens in an experimental program is to use a finite element program. Once the model is calibrated in the finite element program, it is possible to change many parameters and, as a result, the full behaviour of numerous specimens are predicted with good accuracy.

Due to the complexity of the stress–strain relationship for the different materials, the use of a finite element program is essential to predict the nonlinear behaviour of a specimen. Many commercial softwares for structural analysis are available on the market, but these softwares are generally used for design purposes; i.e., their response stays in the linear elastic part of the behaviour. Consequently, to compare the experimental results at the failure of the panels, we needed a software able to predict the nonlinear behaviour of these panels. In addition, since the panels had a variable inertia cross-section along the length, the software had to be flexible enough to input custom sections.

Two softwares were use to simulate the tests carried out on the four panels. The commercial software ADINA and the free software Response 2000® were calibrated and then used to evaluate the influence on the level of prestressing, the compressive strength of the concrete and the reinforcement ratio.
5.2 ADINA

ADINA is a program that allows the user to model reinforced concrete elements and to predict the nonlinear behaviour of a structure [ADINA R&D, Inc., 2003]. To model a concrete element prestressed and reinforced with steel or FRPs, twelve general steps have to be done before obtaining results:

1. Model generalities (Heading, Degrees of freedom, Coordinate system),
2. Definition of the geometric points for the concrete and rebars,
3. Assigning surfaces and surface thicknesses for the concrete elements,
4. Assigning lines for the tendons and reinforcement; input their respective area cross-sections,
5. Material definitions (Concrete, Tendons, Reinforcement bars, Stirrups),
6. Definition and grouping the elements together (Plane stress elements, Rebar elements),
7. Definition of the boundary conditions (Fix or Free degree of freedom at the supports),
8. Definition of the load cases (weight of the panels, three loading positions),
9. Choosing the appropriate mesh-density for the concrete surfaces and the rebar lines; then, meshing the elements,
10. Adjusting the analysis parameters,
11. Run the pre-processing file,
12. Read and output the different results from the post-processing file.

A two-dimensional finite element model was employed with material nonlinearities (concrete cracking/crushing and rebar plasticity). The concrete was modelled using nine-node isoparametric plane stress elements, the thickness of which was the same as the panel width normal to the two-dimensional plane. The concrete elements were meshed to have elements of dimension 200 by 200 mm (Fig. 5.1). This meshing size provided the most accurate results. The prestressing tendons, slab reinforcement and stirrups were modelled as truss rebar elements. They were represented using three-node truss elements, each section area of which equalled the total section area of the rebars placed in parallel in the direction of the specimen width. The truss elements shared nodes with the plane
stress elements, which implies that bond-slipping was not considered. To allow slipping, the tension strain constant, $\chi$, was adjusted to consider the reinforcement ratio.

The ADINA program has a predefined package to enter the material properties of concrete. These properties are input in ADINA using the test results on the concrete samples or the equations described in Section 3.2.1. Table 5.1 presents the values input in ADINA for each panel to characterize the concrete behaviour. With the uniaxial maximum compressive stress, $f_c$, the uniaxial ultimate compressive stress, $f_{cu}$, the uniaxial cut-off tensile stress, $f_t$, the uniaxial maximum compressive strain at above stress, $\epsilon_o$, the uniaxial ultimate compressive strain, $\epsilon_{cu}$, the tangent modulus at zero strain, $E_t$, the Poisson ratio, $\nu$, and constant for tensile strain definition, $\chi$, the concrete can be defined in ADINA. The concrete properties are the ones measured at the day of testing of the panels. Figures 5.2 to 5.5 show the behaviour of the concrete in ADINA for each panel since we had four different concretes.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Unit</th>
<th>DT Panels</th>
<th>ST Panels</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_c$</td>
<td>[MPa]</td>
<td>74</td>
<td>58</td>
</tr>
<tr>
<td>$f_{cu}$</td>
<td>[MPa]</td>
<td>52</td>
<td>30</td>
</tr>
<tr>
<td>$f_t$</td>
<td>[MPa]</td>
<td>5.2</td>
<td>4.3</td>
</tr>
<tr>
<td>$\epsilon_o$</td>
<td>[με]</td>
<td>2590</td>
<td>2360</td>
</tr>
<tr>
<td>$\epsilon_{cu}$</td>
<td>[με]</td>
<td>4000</td>
<td>4000</td>
</tr>
<tr>
<td>$E_t$</td>
<td>[MPa]</td>
<td>35,460</td>
<td>32,097</td>
</tr>
<tr>
<td>$\nu$</td>
<td>-</td>
<td>0.167</td>
<td>0.167</td>
</tr>
<tr>
<td>$\chi$</td>
<td>-</td>
<td>7</td>
<td>6</td>
</tr>
</tbody>
</table>
Figure 5.2  ADINA concrete stress–strain for the DT steel panel

Figure 5.3  ADINA concrete stress–strain for the DT FRP panel
Figure 5.4  ADINA concrete stress–strain for the ST steel panel

Figure 5.5  ADINA concrete stress–strain for the ST FRP panel
Table 5.2  Prestressing strain input in ADINA

<table>
<thead>
<tr>
<th>Tendon position</th>
<th>DT Panels Steel Force [kN]</th>
<th>Strain $\mu \varepsilon$</th>
<th>FRP Force [kN]</th>
<th>Strain $\mu \varepsilon$</th>
<th>ST Panels Steel Force [kN]</th>
<th>Strain $\mu \varepsilon$</th>
<th>FRP Force [kN]</th>
<th>Strain $\mu \varepsilon$</th>
</tr>
</thead>
<tbody>
<tr>
<td>top left</td>
<td>171</td>
<td>5725</td>
<td>66</td>
<td>5596</td>
<td>0</td>
<td>0</td>
<td>77</td>
<td>6530</td>
</tr>
<tr>
<td>top centre</td>
<td>171</td>
<td>5725</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>61</td>
<td>5180</td>
</tr>
<tr>
<td>top right</td>
<td>171</td>
<td>5725</td>
<td>70</td>
<td>5896</td>
<td>115</td>
<td>4033</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>bottom left</td>
<td>171</td>
<td>5725</td>
<td>70</td>
<td>5896</td>
<td>70</td>
<td>2440</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>bottom centre</td>
<td>171</td>
<td>5725</td>
<td>56</td>
<td>4695</td>
<td>-</td>
<td>-</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>bottom right</td>
<td>171</td>
<td>5725</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>60</td>
<td>5029</td>
</tr>
</tbody>
</table>

Reinforcement and stirrup materials were input as from the material test or using the manufacturer specifications. Figures 5.6 and 5.7 present the stress–strain relationship input in ADINA for the steel reinforcement bars and the GFRP reinforcement bars, respectively. Prestressing is modelled by inputting an initial strain in the tendons using Eq. 5.1, where $P_{eff}$ is the prestressing force after all losses, $E_p$ is the modulus of elasticity of the CFRP or steel tendons and $A_p$ is the cross-section area of the tendon. Table 5.2 shows the prestressing strain input in ADINA corresponding to the prestressing force in each tendon. The stress–strain relationship for the steel tendons used in the DT steel panel, for the steel tendons used in the ST steel panel and the linear elastic behaviour of the CFRP tendons used in the FRP panels were also input in ADINA (Figs. 5.8, 5.9 & 5.10).

$$\varepsilon_i = \frac{P_{eff}}{E_p A_p}$$  \hspace{1cm} (5.1)

According to the manufacturer, 41.5 mm/m is the ultimate strain for the prestressing steel tendons. With this value of ultimate strain, the ADINA program stopped prematurely because of a localized excessive strain in one part of the element tendon. This occurred when the panels were beginning to be severely damaged by the cracking of the concrete. To model the complete behaviour of the steel panels, the ultimate strain of the prestressing steel tendons was thus increased to 900 mm/m. The localized excessive strain may be caused by the way the contact between the reinforcement and the concrete
Figure 5.6  ADINA steel reinforcement bars stress–strain for the steel panels

Figure 5.7  ADINA GFRP stress–strain for the FRP panels
Figure 5.8  ADINA steel tendon stress–strain for the DT steel panel

Figure 5.9  ADINA steel tendon stress–strain for the ST steel panel
Figure 5.10  ADINA CFRP stress–strain for the FRP panels

was modelled; it was assumed to be a full contact between the two elements. The same occurred for the FRP panels. The ultimate strain of the CFRP tendons was 16 mm/m. CFRP is an elastic material, so if the ultimate strain was increased, the ultimate strength also had to be proportionally increased to maintain the same behaviour of the material. Thus, to model the complete response of the FRP panels, the ultimate strain was increased to 40 mm/m, with a proportional increase of the ultimate strength. By allowing more strain in the prestressing tendons, the program was able to attain a higher load capacity, and thus simulate more deflection and more damage in the concrete.

For the boundary conditions, one end of the panels was modelled as a simple support allowing only rotation and the other end was modelled as a roller support allowing horizontal displacements and rotations.

A static analysis with a nonlinear formulation was performed. The simulations were carried out using an incremental approach with a constant load increment of 4 kN per load step. A total of 100 steps were used to allow the program to converge to a solution when the panels were severely damaged. The loading positions were also performed in the same order as in the experimental program (Figs. 5.11, 5.12 & 5.13). This can be realized in ADINA by defining the time step for the different load cases.

The results obtained from ADINA are presented in Section 6.4.2 and Table 6.2 compares these results with the values from the experimental program.
Figure 5.11  ADINA central loading position

Figure 5.12  ADINA intermediate loading position

Figure 5.13  ADINA shear loading position
5.3 Response 2000®

Response 2000® is a free downloadable software developed by DENTZ, E. C. [2001] of the University of Toronto. The software has been developed for reinforced concrete sectional analysis using the modified compression field theory. This program is simple to use. Step by step, the user defines the material properties, concrete cross-section and then positions each reinforcement bar, stirrup and prestressing tendon in the element to be analyzed. Prestressing is applied by inputting an initial strain in the tendons (Eq. 5.1). By default, a 43 mm/m ultimate strain is specified by the program for the prestressing steel tendons. As in the ADINA program, to achieve the complete behaviour of the panels, the ultimate strain of the prestressing steel tendons was increased to 70 mm/m and 86.3 mm/m for the DT steel and ST steel panels, respectively. In the same way, to model the complete response of the FRP panels, the ultimate strain was increased to 26 mm/m and 23 mm/m for the DT FRP and ST FRP panels, respectively, with a proportional increase of the ultimate strength. By allowing more strain in the prestressing tendons, the simulation is able to reach a higher load capacity and higher deflection. The model characteristics of the ST steel and ST FRP panels are presented in Figs. 5.14 and 5.15, respectively.

Response 2000® calculates the sectional response, but it is also possible to obtain the full member response of the element by inputting the length of the element. Many graphical results are possible such as load–deflection curves, predicted crack patterns at failure, as well as stresses and strains in the concrete and reinforcement.

The results obtained from the Response 2000® analyses are compared to the experimental values in Table 6.2 and presented in detail in Section 6.4.2. The results with Response 2000® are in good agreement with the experimental values and very similar to the results obtained with ADINA. However, to obtain similar results, Response 2000® is an easier program to work with and less time-consuming.

This chapter showed how to model the panels with steel and FRPs using the two finite element softwares, ADINA and Response 2000®. From the results, the Response 2000® program performed as well as ADINA, but with much less computational time. Its interface makes the program very easy to use and modifications are directly input. Meanwhile, the Response 2000® had program limitations; namely, non-symmetrical loading of the panels and the input of different cross-sections along the length. The next chapter presents the results from the experimental and numerical programs. The results are compared and analyzed to investigate the service and ultimate behaviours of the panels.
Figure 5.14 Response 2000© ST steel panel model

### Geometric Properties

<table>
<thead>
<tr>
<th>Area (mm²) x 10³</th>
<th>409.5</th>
<th>425.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inertia (mm⁴) x 10⁶</td>
<td>19063.0</td>
<td>20022.9</td>
</tr>
<tr>
<td>y₁ (mm)</td>
<td>222</td>
<td>222</td>
</tr>
<tr>
<td>y₂ (mm)</td>
<td>578</td>
<td>578</td>
</tr>
<tr>
<td>S₁ (mm³) x 10³</td>
<td>85733.4</td>
<td>90271.1</td>
</tr>
<tr>
<td>S₂ (mm³) x 10³</td>
<td>33001.2</td>
<td>34630.3</td>
</tr>
</tbody>
</table>

### Crack Spacing

\[ 2 \times \text{dist} + 0.1 \frac{d_p}{p} \]

### Loading (N, M, V + dN, dM, dV)

\[ 0.0, 0.0, 0.0 + 0.0, 1.0, 0.0 \]

### Concrete

- \( f'_c = 57.8 \text{ MPa} \)
- \( a = 19 \text{ mm} \)
- \( f_t = 2.28 \text{ MPa (auto)} \)
- \( c'_c = 2.36 \text{ mm/m} \)

### Rebar

- \( f_y = 400 \text{ MPa} \)
- \( e_y = 100.0 \text{ mm/m} \)

### P-Steel

- \( f_{pu} = 1860 \text{ MPa} \)
- \( e_p = 86.3 \text{ mm/m} \)

All dimensions in millimetres

Clear cover to transverse reinforcement = 30 mm

ST Steel Panel

Derek Tardif    2004/12/9
Geometric Properties

<table>
<thead>
<tr>
<th>Area (mm²) x 10^3</th>
<th>409.5</th>
<th>412.8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inertia (mm⁴) x 10⁶</td>
<td>19063.0</td>
<td>19554.3</td>
</tr>
<tr>
<td>ye (mm)</td>
<td>222</td>
<td>224</td>
</tr>
<tr>
<td>ye (mm)</td>
<td>578</td>
<td>576</td>
</tr>
<tr>
<td>S_e (mm³) x 10^3</td>
<td>85733.4</td>
<td>87180.4</td>
</tr>
<tr>
<td>S_e (mm³) x 10^3</td>
<td>33001.2</td>
<td>33965.9</td>
</tr>
</tbody>
</table>

Crack Spacing

2 x dist + 0.1 dy / dp

Loading (N.M.V + dN.dM.dV)

0.0, 0.0, 0.0 + 0.0, 1.0, 0.0

Concrete

f'_c = 43.6 MPa

f'_c = 2.15 mm/m

a = 19 mm

f_t = 2.04 MPa (auto)

Rebar

f'_b,r = 676 MPa

f'_b,r = 6.0 mm/m

P-CFRP

f'_{P+C} = 3799 MPa

f'_{P+C,r} = 23.0 mm/m

2 layers of

A_0 = 1308 mm²

A_v = 84 mm² per leg

@ 300 mm

1 x 72 mm²

Δε_{P+C} = 5.18 mm/m

slope = 0.14%

1 x 72 mm²

Δε_{P+C} = 6.53 mm/m

slope = 0.14%

3 x 72 mm²

slope = 0.14%

1 x 72 mm²

Δε_{P+C} = 5.03 mm/m

slope = 0.14%

All dimensions in millimetres

Clear cover to transverse reinforcement = 31 mm

ST FRP Panel

Derek Tardif 2004/12/9
Chapter 6

Test Results and Analysis

6.1 Introduction

This last chapter presents the results obtained from the experimental program and compares them to the values calculated with the theoretical equations and also with the predictions obtained from the two finite element programs. The behaviour of the panels were characterized by the mid-span and third point deflections and also by the strains obtained from the gauges fixed on the tendons, concrete, reinforcement bars and stirrups. The first part of the chapter presents the main experimental values compared to the hand-calculation predictions. After, the experimental results, for the serviceability behaviour under the three loading positions, are explained. Then, in the final section, the detailed experimental and numerical results are presented and compared for the ultimate behaviour of the panels under the central loading position.

6.2 Main Experimental Results

The main experimental results of the four loading tests are summarized in Table 6.1. The total area section of prestressing, $A_p$, the total effective prestressing force after losses, $P_{eff}$, and the compressive strength of the concrete at the time of testing, $f_c$, are presented. Then, the cracking moment, $M_{cr}$, is compared to the experimental moment, $M_{exp,cr}$, obtained from the first three loading tests on each specimen until cracks first appeared. The cracking moment, $M_{cr}$, was evaluated using Eq. 6.1. For $M_{exp,cr}$, values are given for the maximum moment applied under the central loading position, $M_{centre}$, the maximum moment applied under the intermediate loading position, $M_{intermediate}$, and the maximum
moment applied under the shear loading position, \( M_{\text{shear}} \), when cracks first appeared for each loading configuration. In Eq. 6.1, \( A_g \) is the gross area of the section, \( f_r \) is the tensile strength of the concrete at the time of testing, \( I_g \) is the moment of inertia of the gross concrete section about the centroid axis, neglecting reinforcement, and \( y_{\text{bottom}} \) is the distance to the extreme tension fibre from the centroid of the uncracked section. The cracking moment was determined by considering the moment due to the eccentricity of the prestressing force, \( P_{\text{eff}} \cdot e \), as an exterior force where \( e \) is the eccentricity of a tendon relative to the centroid axes. The experimental moment, \( M_{\text{exp}} \), is the sum of the moment due to the weight of the panel, \( M_{DL} \), with the moment from the applied load on the panel during the testing, \( M_{LL} \), and the moment due to the eccentricity of the prestressing force, \( M_P \) (Eq. 6.2). With this rearrangement, the cracking moment becomes constant when the cross-section is constant. Since the maximum moments, for the three load positions, were always in the central part of the panel with a constant cross-section, it was easier to compare the experimental moments with a constant cracking moment.

\[
M_{cr} = \left( \frac{P_{\text{eff}}}{A_g} - f_r \right) \frac{I_g}{y_{\text{bottom}}} \quad (6.1)
\]

\[
M_{\text{exp}} = M_{DL} + M_{LL} + P_{\text{eff}} \cdot e \quad (6.2)
\]

The experimental moments for the two DT panels show a very good agreement with respect to the cracking moment. On the other hand, a difference of 9% and 40% for the ST steel panel and ST FRP panels, respectively, is observed for the central loading position. For the ST steel panel, mishandling of the panel at an early age had initiated micro-cracks near the right hold-down device, and these cracks can be seen in Fig. 6.1. For the ST FRP panel, many possibilities could result in a lower cracking moment: air in the concrete, bad placing of the concrete during casting, variability of the tensile strength of the concrete, the actual prestressing force, or mishandling at a young age. However, the experimental moments for the intermediate and shear loading were more accurate for the cracking moment.

The nominal moment, \( M_{\text{nominal, strain}} \), is compared with the experimental moment, \( M_{\text{exp, failure}} \), under the central loading until the panels reached failure. All material resistance factors are taken as unity. The nominal moments were calculated assuming strain
compatibility and considering failure of the critical section when the ultimate strain of one of the tendons was reached. In addition, because the ultimate strain in the top layer of the concrete was never reached, the values of $\alpha$ and $\beta$ for the equivalent compression block of the concrete was taken as proposed in the ISIS Canada manual [ISIS CANADA M-03, 2001]. The strain compatibility method is an excellent, reliable and easy way to predict the ultimate capacity of steel and FRP panels. The software Response 2000® was also used to check the nominal moments, $M_{\text{nominal, Response 2000}}$, and the results are in good agreement with those calculated by strain compatibility.

### Table 6.1 Main results of the experimental program

<table>
<thead>
<tr>
<th></th>
<th>Unit</th>
<th>DT Panels</th>
<th>ST Panels</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>STEEL</td>
<td>FRP</td>
</tr>
<tr>
<td>$A_p$, by stem</td>
<td>mm$^2$</td>
<td>560</td>
<td>359</td>
</tr>
<tr>
<td>$P_{\text{eff}}^*$, stem 1; stem 2</td>
<td>kN</td>
<td>684; 684</td>
<td>262; 331</td>
</tr>
<tr>
<td>$f_c$</td>
<td>MPa</td>
<td>74</td>
<td>31</td>
</tr>
<tr>
<td>$M_{cr}$</td>
<td>kN.m</td>
<td>225</td>
<td>130</td>
</tr>
<tr>
<td>$M_{\text{centre}}$ ; $M_{\text{centre}} / M_{cr}$</td>
<td>kN.m ; -</td>
<td>220 ; 0.98</td>
<td>130 ; 1.00</td>
</tr>
<tr>
<td>$M_{\text{exp,cr}}$ ; $M_{\text{intermediate}} / M_{cr}$</td>
<td>kN.m ; -</td>
<td>260 ; 1.15</td>
<td>140 ; 1.08</td>
</tr>
<tr>
<td>$M_{\text{shear}}$ ; $M_{\text{shear}} / M_{cr}$</td>
<td>kN.m ; -</td>
<td>260 ; 1.15</td>
<td>140 ; 1.08</td>
</tr>
<tr>
<td>$M_{\text{nominal, Response 2000}}$</td>
<td>kN.m</td>
<td>949</td>
<td>575</td>
</tr>
<tr>
<td>$M_{\text{nominal, strain}}$</td>
<td>kN.m</td>
<td>900</td>
<td>580</td>
</tr>
<tr>
<td>$M_{\text{exp, failure}}$ ; $M_{\text{exp, failure}} / M_{\text{nominal, strain}}$</td>
<td>kN.m ; -</td>
<td>$&gt; 660 ; 0.73$</td>
<td>$560 ; 0.97$</td>
</tr>
</tbody>
</table>

* The detail calculations of the prestress loss for each panel are shown in Appendix B.

### 6.3 Serviceability Behaviour

To study the serviceability behaviour, three different loading positions were performed on the panels until the first cracks appeared. As presented in Section 4.5, the order of loading position for the DT steel, ST FRP and ST steel panels is the central loading, then the intermediate loading and finally, the shear loading positions. For the DT FRP panel, the order was slightly changed; starting with the intermediate loading, then the central loading and finally, the shear loading positions.

#### 6.3.1 Panel flexural behaviour under central loading until cracking

For three of the four panels, the first loading step consisted of increasing the load up to the cracking of the stems under the central loading configuration. This allowed us to
study the behaviour of the panels within the service load range. Figures 6.1 and 6.2 show the results for the mid-span and third point deflections, respectively. They also give the reference design load truck CL-625 of 163 kN. In the first graph, it also shows the cracking pattern at the cracking. The cracks all occurred in the constant moment zone as it should be. The horizontal axis is the mid-span or third point displacements of the stems and the vertical axis is the total load applied on only one stem. For the DT specimens, since the loading frame weighed 20 kN, this has been added to the applied load and it is the reason why the graph begins and finished at this loading value. In the same figures, it can be seen that the DT FRP panel curve is not starting with zero deflection since it was not tested in the same order of loading positions as in the other three panels. Consequently, the DT FRP panel was permanently damaged from a previous loading configuration.

In Figs. 6.1 and 6.2, cracking of the panels is observed when a relatively small increase in the load produces a relatively large displacement. The cracking load for the DT steel panel of 310 kN is well above the design load, as it should be. On the other hand, the cracking load of the DT FRP, ST FRP and ST steel panels was less than the design load with values of 130 kN, 80 kN and 115 kN, respectively. The lack of sufficient prestressing force combined with a lower compressive strength of the concrete are largely responsible for these low results. For the FRP panels, because of the problem with the anchorage system, it was impossible to attain the target prestressing force. So to be able to make meaningful comparisons, the level of prestressing for the ST steel panel was made the same as that for the ST FRP panel. From the cracking load, it can be seen that the level of prestressing directly influences the cracking load. At the time of the tests, the concrete strength was only 31 MPa and 44 MPa for the DT FRP and ST FRP panels, respectively, and 58 MPa and 74 MPa for the DT steel and ST steel panels, respectively. Had the prestressing force and the concrete strength been in accordance with the design specifications for the FRP panels, the cracking load would have been in the range of 280 kN. When the load was removed, the displacements did not return exactly to zero, thus confirming the presence of damage and cracking. In addition, strain gauges installed on stirrups and bars near the constant moment zone indicated significant changes in the measurements when the elements first cracked. All the recorded strain variations during the testings are presented in Appendix C. The displacements at the third point showed, as well, good symmetry. The maximum deflection under service load prescribed in the CAN/CSA-S6-00 [2000] is 6 mm; thus, all the panels satisfy this requirement.
Figure 6.1 Load to centre displacement behaviour until cracking for the central loading test
6.3.2 Panel flexural behaviour under intermediate loading until cracking

The second loading step for two of the four panels consisted of increasing the load up to the cracking of the stems under the intermediate loading configuration. This loading test was not performed on the ST steel panel due to time restriction in the laboratory schedule; so no results are presented for this panel in this section. All specimens were already cracked from the first test configuration, except for the DT FRP panel, since this was its first loading test. As a result, it can be seen on the graphs that the permanent deflections are considerable for the two other panels. Figures 6.3 and 6.4 present the deflections at mid-span and at third point, respectively, for the second test. A good relationship exists between the deflection at third point on the passive side with the deflection at third point on the active side of the panel (Fig. 6.4). The cracking pattern for this test is illustrated with dark black lines while, the permanent cracks from the first test configuration are shown with the light gray lines (Fig. 6.3). The cracks, from this loading test, also first appeared in the maximum constant moment zone. Figure 6.5 shows a picture of the cracks from the intermediate loading position for the ST FRP panel. As in the first test, for the same reasons, the DT steel panel had a much higher cracking load than the other two panels for the intermediate loading configuration.
Figure 6.3 Load to centre displacement behaviour until cracking for the intermediate loading test
Figure 6.4  Load to intermediate displacement behaviour until cracking for the intermediate loading test

Figure 6.5  Cracks for the ST FRP panel from the intermediate loading position
Figure 6.6  Load to centre displacement behaviour until cracking for the shear loading test

6.3.3  Panel shear behaviour under loading near the support until cracking

The third loading step was performed on all the panels by increasing the load up to the cracking of the stems under the shear loading configuration. At this stage, all specimens were already cracked from the two previous test configurations. The deflection results of the third test are shown in Figs. 6.6 and 6.7. As in the two other loading tests, for the same reasons, the DT steel panel had a much higher cracking load than the other three panels for the shear loading configuration. Figure 6.8 shows a picture of the cracks from the shear loading position for the ST FRP panel.

At service loads, the only specimen that was able to support the truck load was the DT steel panel. The FRP panels and the ST steel panel did not meet this requirement because they did not have enough prestressing force and/or compressive strength for the concrete.
Figure 6.7  Load to intermediate displacement behaviour until cracking for the shear loading test

Figure 6.8  Cracks for the ST FRP panel from the shear loading position
6.4 Ultimate Behaviour: Panel flexural behaviour under central loading until failure

6.4.1 Experimental results

For the final loading test, the panels were centrally loaded to failure to observe the flexural behaviour at the ultimate state. The results for the mid-span and third point deflections of the four experimental panels are presented in Figs. 6.9 and 6.10, respectively. As previously, the load on the graph is for each stem. The crack distributions are also illustrated (Fig. 6.9). The measurements at third point positions showed good symmetry up to failure for all panels. (Fig. 6.10). At the ultimate state, the lower than expected concrete compressive strength for the FRP panels shows little influence on the capacity, mainly because of the small depth of the neutral axis.
The steel elements exhibit typical steel prestressed concrete behaviour; an elastic behaviour until cracking, then the rate of member deflection progressively increase as the tendons yield. No rupture occurred in the steel panels. They exhibited significant ductility and the loading process was stopped before rupture of the prestressing steel tendons. For the DT steel panel, the ultimate strength, corresponding to the yielding of the steel, is a load of 500 kN and a mid-span vertical displacement of 23 mm. After this point, there was no significant increase in the load, but large deflections at mid-span and at the third point were occurring. It is worth recalling that the DT steel panel had passive flexural reinforcement, which provided 15% of the strength; it also had its full prestressing force. From the graphs, the ultimate strength for the ST steel panel is 420 kN with a central displacement of 100 mm. Figures 6.11 and 6.12 present the cracks pattern for the DT and ST steel panels at failure, respectively.

The FRP deck panels exhibited significant post cracking stiffness up to failure, instead of the usual plateau observed for steel reinforced concrete (Figs. 6.9 & 6.10). Nevertheless, signs of distress, such as excessive deflections and large cracks, appeared well before actual failure and provided clear warnings of imminent failure, rather than conventional ductility. Failure occurred by the rupture of the tendons. A single very wide crack, in the range of 25 mm, at the mid-span section for the DT FRP panel and at the left hold-down
Figure 6.11  Cracks at failure for the DT steel panel

Figure 6.12  Cracks at failure for the ST steel panel
device for the **ST** FRP panel, suggest that the hold-down devices were the initiator of the tendon rupture (Fig. 6.9). The reduction of the CFRP tendon tensile strength caused by the hold-down device has been reported by other researchers [Grace and Abdel-Sayed, 2000]. As discussed in Section 4.4.2, high density plastic hold-down devices should be used instead of conventional steel roller to avoid the premature failure of the CFRP tendons. For both the **DT** FRP and **ST** FRP panels, failure was observed under a total load of 410 kN for a mid-span vertical displacement of 100 mm and 110 mm, respectively. At the ultimate state, the **DT** FRP, **ST** FRP and **ST** steel panels showed the same behaviour because they all had virtually the same level of prestressing. Figures 6.13 and 6.14 present the cracks for the **DT** and **ST** FRP panels at failure, respectively.

All strain gauges installed on the prestressing tendons were lost at an early stage in the loading. Consequently, the maximum stresses in the tendons at the ultimate load were not recorded. Many signals for the strain gauges on the stirrups and bars were also lost during the final test. Therefore, it was not possible to determine all the stresses in the stirrups or reinforcement bars. However, a maximum stress of 121 MPa and 70 MPa were recorded on the stirrups for the **ST** FRP and **DT** FRP panels, respectively. These levels of stresses are 18% and 10% of the ultimate stress for the GFRP bars. This indicates that the shear reinforcement is able to resist the maximum load without rupture. From the test results, a rupture by flexure was observed and this mode of rupture is clearly preferable to a brittle shear rupture. For the steel panels, yielding in the slab reinforcement and in the stirrups was recorded; again, the measured strains were much lower than the ultimate strain. All the load to strain graphs for each panel and also for each loading configuration are presented in Appendix C. Meanwhile, the following presents the strain results during
the ultimate loading test for a tendon, two slab reinforcement bars and two stirrups. The position of the strain gauges can be found in Appendix C. As shown in Fig. 6.15, the strain measurements on the tendon could not be used to determine the maximum stress at failure. Almost every strain gauge on the tendons failed during the final loading and even the recorded strains were far away from the ultimate strain of the materials. Then, the strain graph for the top slab reinforcement bar at mid-span is presented (Fig. 6.16). From each curve, it can be seen that the strains started in compression and shifted to tension as the neutral axis moved upward. For the steel reinforcement bars, the yielding points have been reached before the failure of the steel panels. For the FRP panels, the strains of the GFRP bars were very low compared to its ultimate strain. The next figure illustrates the strain graph for the top slab reinforcement bar at third point (Fig. 6.17). In this case, it seems that the strains for the DT FRP and DT steel panels remained in compression which means that the neutral axis never reached the layer of reinforcement at this section, although for the ST FRP and ST steel panels, the neutral axis had moved over the reinforcements. However, the final tensile strains were very low for both materials compared to their ultimate strains. At this section, the steel reinforcement did not yield. The next two graphs, Figs. 6.18 and 6.19, show the behaviour of the strain for the stirrup n°14 from the active side and the behaviour of the strain for the stirrup n°7 from the
Figure 6.15  Load–strain behaviour for the tendons for the central loading test until failure.

passive side, respectively. Since the panels were controlled by flexure, the strain activity in the stirrups began only once the panels were very damaged. The recorded strains were very low even at failure providing clear conclusion that the shear reinforcement was absolutely sufficient for these type of panels.

In the steel panels, the crack distribution shows cracks that are much closely spaced than those for the FRP panels (Fig. 6.9). This phenomenon is related to the bonding between the tendons and concrete. The prestressing steel tendons offered a rougher and greater contact surface than the CFRP indented tendons. For that reason, the steel tendons provided a better bonding to the concrete and led to a very dense cracking distribution compared to the CFRP tendons. The crack distribution is not an indicator to conclude that one type of panel is better than the other one; it only reflects the bonding behaviour.
Figure 6.16  Load–strain behaviour for the top central longitudinal bar for the central loading test until failure

Figure 6.17  Load–strain behaviour for the top longitudinal bar at third point for the central loading test until failure
Figure 6.18  Load – strain behaviour for stirrup n°14 for the central loading test until failure

Figure 6.19  Load – strain behaviour for stirrup n°7 for the central loading test until failure
6.4.2 Numerical results

The last part of this work provides the values predicted by the two finite element programs, ADINA and Response 2000®, for the ultimate loading test. Comparisons between the numerical predictions and the experimental values are presented in Table 6.2. For the steel panels, the ultimate load is predicted with an accuracy of ±5% with both the ADINA and Response 2000® programs. Meanwhile, up to 26% and 18% more mid-span vertical deflection for the DT steel and ST steel panels, respectively, were obtained from both programs. For the third point deflection, up to 56% and 12% more deflection were predicted for the DT steel and ST steel panels, respectively. These results confirmed that the panels still had strength reserves before the actual failure. In the tests, the steel panels never reached failure, thus confirming the presence of a strength reserve. The mid-span and third point deflection predictions for the DT steel panel are shown in Figs. 6.20 and 6.21, respectively. For the ST steel panel, the same deflection predictions are presented in Figs. 6.22 and 6.23. For the steel panels, both softwares give a good prediction of the behaviour up to failure. In Figs 6.20 and 6.22, the predictions of the as-designed FRP panel are also presented to illustrate the different behaviours for the two materials.

In Figs. 6.24 and 6.25, the mid-span and third point vertical displacement predictions for the DT FRP panel are given. For the ST FRP panel, these results are shown in Figs. 6.26 and 6.27. It can be seen that the general behaviour predicted by both programs is very similar to that observed in the experiments. For the FRP panels, ADINA and Response 2000® predicted values 10% and 16% less load capacity than the ultimate experimental load. In addition, the vertical deflection at mid-span only reached 66% to 90% of the experimental values. For the third point vertical deflection, the values were within the range of 60% to 95%. Also, in Figs. 6.24 and 6.26, the predictions of the as-designed FRP panel is presented to show what it would have been if the design prestressing level were applied and if the compressive strength of the concrete of 60 MPa had been used. ADINA and Response 2000® were somewhat less accurate for predicting the behaviour of the FRP panels than the steel panels.

Predictions obtained with ADINA and Response 2000® are quite sensitive to the parameters employed in the analysis. The concrete mechanical properties are very complex and their interaction with other materials is also complex. The general behaviour of the panels will be slightly changed by increasing the number of time steps or changing the meshing size for the concrete elements or changing the tension stiffness factor in the con-
## Table 6.2 Main results of the numerical program

<table>
<thead>
<tr>
<th>Panel</th>
<th>Experimental at failure</th>
<th>Ratio</th>
<th>Response 2000°</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ADINA</td>
<td>ADINA/Exp.</td>
<td>2000°/Exp.</td>
<td></td>
</tr>
<tr>
<td>DT Steel</td>
<td>Load [kN]</td>
<td>604</td>
<td>590 0.98</td>
<td>635</td>
</tr>
<tr>
<td></td>
<td>Δcentre [mm]</td>
<td>135</td>
<td>168 1.24</td>
<td>170</td>
</tr>
<tr>
<td></td>
<td>Δ1700 [mm]</td>
<td>82</td>
<td>128 1.56</td>
<td>92</td>
</tr>
<tr>
<td>DT FRP</td>
<td>Load [kN]</td>
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<td>370 0.90</td>
<td>345</td>
</tr>
<tr>
<td></td>
<td>Δcentre [mm]</td>
<td>100</td>
<td>90 0.90</td>
<td>71</td>
</tr>
<tr>
<td></td>
<td>Δ1700 [mm]</td>
<td>62</td>
<td>59 0.95</td>
<td>42</td>
</tr>
<tr>
<td>ST FRP</td>
<td>Load [kN]</td>
<td>410</td>
<td>375 0.91</td>
<td>385</td>
</tr>
<tr>
<td></td>
<td>Δcentre [mm]</td>
<td>110</td>
<td>81 0.74</td>
<td>73</td>
</tr>
<tr>
<td></td>
<td>Δ1700 [mm]</td>
<td>73</td>
<td>55 0.75</td>
<td>44</td>
</tr>
<tr>
<td>ST Steel</td>
<td>Load [kN]</td>
<td>500</td>
<td>480 0.96</td>
<td>530</td>
</tr>
<tr>
<td></td>
<td>Δcentre [mm]</td>
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<td>267 1.18</td>
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</tr>
<tr>
<td></td>
<td>Δ1700 [mm]</td>
<td>130</td>
<td>143 1.10</td>
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<tr>
<td>FRP, as design</td>
<td>Load [kN]</td>
<td>515</td>
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<td></td>
</tr>
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<td>Δcentre [mm]</td>
<td>60</td>
<td>53</td>
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</tr>
<tr>
<td></td>
<td>Δ1700 [mm]</td>
<td>37</td>
<td>30</td>
<td></td>
</tr>
</tbody>
</table>

Concrete material properties. To verify if the programs exploited the full capacity of each material, the stress–strain relationship for each material was checked at strategic points on the panels. As presented in Table 6.2, the programs were more accurate to predict the load capacity than the deflections. Since the number of time steps or meshing size or tension stiffness factor are unknown values at the beginning, we had to estimate them to the best of our knowledge. Then, after having the first prediction, these values were changed to reflect the actual behaviour of the experimental panels. This allowed us to calibrate the four experimental panels and then, it was easy to modify few parameters to obtain the behaviour of similar panels with good accuracy. Since the programs cannot predict both load and deflections in a direct way, these programs should be used with great judgment for design purpose if no experimental results are available.
Figure 6.20 Numerical and experimental load to mid-span deflection behaviour until failure of the DT steel panel

Figure 6.21 Numerical and experimental load to intermediate deflection behaviour until failure of the DT steel panel
Figure 6.22  Numerical and experimental load to mid-span deflection behaviour until failure of the ST steel panel

Figure 6.23  Numerical and experimental load to intermediate deflection behaviour until failure of the ST steel panel
Figure 6.24  Numerical and experimental load to mid-span deflection behaviour until failure of the DT FRP panel

Figure 6.25  Numerical and experimental load to intermediate deflection behaviour until failure of the DT FRP panel
Figure 6.26 Numerical and experimental load to mid-span deflection behaviour until failure of the ST FRP panel

Figure 6.27 Numerical and experimental load to intermediate deflection behaviour until failure of the ST FRP panel
The ADINA and Response 2000® softwares were also able to predict the cracking patterns from the different loading configurations. With ADINA, it was possible to simulate all loading cases, while with Response 2000® it was only possible to simulate the symmetrical central loading until cracking and up to failure due to program limitation. Figures 6.28, 6.30 and 6.31 illustrate the crack predictions from ADINA for the three loading configurations which studied the serviceability behaviour. For Response 2000®, the crack predictions for the central loading position until cracks first appeared are shown in Fig. 6.29. As in the experimental panels, the cracks occurred in the maximum constant moment zone. At failure, Fig. 6.32 presents the cracks simulated by ADINA while Fig. 6.33 illustrates the cracks predicted by Response 2000®. Results from the ADINA and Response 2000® simulations give crack distributions that are very similar to those observed for the experimental panels under service and ultimate loads (Figs. 6.1, 6.3, 6.6 & 6.9).

Figure 6.28  ADINA cracking for the central loading position

Figure 6.29  Response 2000® cracking for the central loading position
Figure 6.30  ADINA cracking for the shear loading position

Figure 6.31  ADINA cracking for the intermediate loading position

Figure 6.32  ADINA cracking at failure

Figure 6.33  Response 2000® cracking at failure
With the ADINA and Response 2000® softwares, it is also possible to output the values of stress and strain at any point on the panels. Figures 6.34 and 6.35 present the stress and strain predictions, respectively, just after the prestressing force was transferred to the concrete. At the failure of the panels, Figs. 6.36 and 6.37 show the stress and strain predictions. Many discontinuities can be seen in the stresses and strains between the concrete elements at failure. Again, this demonstrates the difficulty to model with high precision concrete specimens. However, before the concrete started to be severely damaged, the stresses and strains more accurately represented the experimental measurements.

Figure 6.34  ADINA stress after the transfer of prestressing on the concrete

Figure 6.35  ADINA strain after the transfer of prestressing on the concrete

Figure 6.36  ADINA stress at failure for the central loading position
Figure 6.37  ADINA strain at failure for the central loading position

In this chapter, it was demonstrated that the experimental results from the testing program behave adequately as calculated from the equations of traditional mechanics of materials or from standards or design manuals. At service loads, only the DT steel panel was able to provide the required strength. It was also shown that for prestressed concrete elements, the level of prestressing force and the strength of the concrete directly influenced the serviceability behaviour. At ultimate load, all panels satisfied the selected truck load. Even if the FRP panels did not provide a conventional ductility as in the steel panels, they exhibited clear and sufficient warnings before their actual failure. Once the panels were calibrated, the two finite element programs, ADINA and Response 2000®, were able to predict the load capacity, the deflections, the stresses and the strains in a relatively good way. In the following chapter, the conclusions of this research project will be summarized and added-up with recommendations for further work.
Conclusion

7.1 Closing Remarks

This investigation aimed to demonstrate, through comparisons between steel prestressing reinforcement and FRP materials, the influence of the type of reinforcement on the serviceability, ultimate strength and mode of failure for bridge deck systems. The main goal of this research project was to verify from tests and comparisons if the material substitution using FRPs is a potential solution to overcome the steel corrosion problem. Four full-scale bridge deck panels were fabricated and tested: two with steel reinforcement serving as references, and two others reinforced with CFRP prestressing tendons and GFRP bars and stirrups. In addition, the experimental results were calibrated in two finite element programs to examine the influence of different parameters such as the grade of the concrete, the level of prestressing and the reinforcement ratio.

Based on the results of this investigation, the following conclusions can be drawn:

- The results from the flexural and shear loading tests on the steel-free panels demonstrated that FRP reinforcements can provide the required load capacity at the ultimate state. At service loads, only the DT steel panel was able to resist the required load. The DT FRP, ST FRP and ST steel panels did not have enough prestressing force and/or concrete compressive strength to carry the service load truck. At ultimate capacity, all the panels were able to resist the selected truck load. At failure, we saw that both materials behaved differently. For the FRP panels, the behaviour was characterized by a linear elastic part until the cracking load is reach, followed by a linear post-cracking part until the rupture of the CFRP tendons. For the steel panels, the behaviour was characterized by the same linear elastic part until the cracking load was reached, then once the yielding point was attained, large deformations occurred until the steel tendons reached their ultimate tensile strain. With
a similar level of prestressing force, the DT FRP, ST FRP and ST steel panels gave approximately the same ultimate load. However, the deflections in the steel panels were greater than the FRP panels.

- The strains recorded from the gauges installed on the stirrups demonstrated that the shear reinforcement for both materials was sufficient. Rupture in flexure occurred for all panels providing clear conclusions that the shear reinforcement was able to support the design truck load.

- Major difficulties were encountered in the development of a reliable anchorage system for the CFRP prestressing tendons. Thus far, three systems have been tested: a glue anchorage, a steel sleeve/aluminum wedges anchorage and a steel sleeve/wedges anchorage with a thin inner cooper sleeve. The last one performed the best in the laboratory. However, in field, rupture and slippage of the CFRP tendons occurred during prestressing. Further research and development is required to address this aspect.

- From sample material tests, the mechanical properties of each material were the same as those provided by the manufacturers. The good correlation between the sample tests and the manufacturer’s values allows engineers to design their structures with confidence.

- The theoretical equations were in good agreement with the results from the experimental tests. The strain compatibility method was very accurate to estimate the nominal moment at the critical section for the FRP panels.

- The failure mode for the FRP panels was caused by the rupture of the tendons at the hold-down devices. However, as demonstrated, the specimens showed signs of distress before actual failure; i.e., large cracks and strong vertical deflections, thus providing clear warnings of imminent failure. The failure mode for the steel panels is unknown because the experimental steel panels were not loaded up to failure because of excessive deflections. However, from the strain compatibility method we had expected that failure occurs once the ultimate tensile strain of the steel tendons is reached. It is important to consider that if, the level of prestressing is too great, the failure mode will change with the brittle crushing of the top concrete zone.

- Two nonlinear finite element programs, ADINA and Response 2000®, were used in
this investigation. The four models were calibrated with experimental values obtained from the four-panel testing program. Both softwares demonstrated excellent results for the steel panels and very good load predictions for the FRP panels. However, to obtain similar results, Response 2000® is an easier program to work with and less time-consuming. Response 2000® is a specific program to investigate prestressed and reinforced concrete elements, while ADINA is a general purpose finite element program.

7.2 Recommendations

Based on this investigation, the following recommendations can be made:

- In this research, the maximum design tensile stress for the CFRP tendons was 60% of the ultimate tensile strength at jacking. However, during the prestressing stage, almost every CFRP tendons failed or slipped under 55% of the ultimate strength. The value of 70% of the ultimate tensile strength provided by CAN/CSA-S6-00 [2000] should perhaps be revised to consider the type of anchorage system. Corrections have already been made for the ultimate effective stress for harped or draped tendons.

- More investigations are needed to develop an anchorage system for the CFRP tendons. The weaker mechanical properties in the transverse direction lead to many problems. The new anchor system should progressively transfer the stress to the tendon to avoid shear or local ruptures. Also, for practical considerations in precast plants, this system should be fast and easy to install and removed.

- For draped or harped CFRP tendons, high density plastic hold-down devices should be used in future projects instead of conventional steel rollers to reduce the stresses on the tendons at the draping points. High density plastic hold-down devices have already been used for the draping of the CFRP tendons on the Taylor Bridge in Winnipeg.

- Fatigue tests should be performed to investigate the long-term response of similar panels.
• To have a better prediction for the loss of prestress in the CFRP tendons, permanent static loading tests should be carried out to investigate whether relaxation occurs with time in the CFRP tendons.

• Investigations should be conducted on the development length of the CFRP tendons to predict the position along the tendon where the effective stress is developed from the end.

• Although we had ordered concretes with a compressive strength of 60 MPa from a local supplier, we did not have the same compressive strength for each panel. The worst concrete had a compressive strength of 31 MPa (half of what was requested). This error would have cost considerable sums of money to the supplier and a loss of time for the project manager had this occurred in a real construction project.

• The use of finite element programs for design purposes should be used with great judgement. Many parameters influence the predictions, such as the meshing size, material parameters and number of time steps.

• In the ADINA program, the two-dimensional specimens were modelled assuming a full bonding condition between the concrete and the reinforcements. This model does not allow any slippage between the two elements. To allow slippage, new models could be developed by creating interface elements between the concrete and the reinforcement. The stiffness of the interface is unknown, however, it could be calibrated to obtain accurate results. Again, because the finite element programs are sensitive to many parameters, there is no direct way to obtain accurate load, deflection, stress and strain predictions. The calibrations of the models are necessary before carrying out a parametric study.

For prestressed concrete bridge elements, the design is generally carried out to avoid cracking under the service loads. So, if either steel or FRP tendons are used for the prestressing, the elements will never reach the cracking load unless the live loads are increased during the lifetime of the bridge. By remaining in the linear elastic part of the behaviour, no cracks should occur. Hence, the elements should not have any corrosion problems with the steel tendons. However, humidity and chlorides from de-icing salts infiltrate the porous concrete and initiate the corrosion process on the steel tendons. The corrosion reduces the strength of the elements as well as the lifetime of the structure. By replacing the steel by a non-corrosive material such as FRPs, the structures will be able
to last longer and maintain their strength over the years. In a few years, the use of FRP materials in concrete structure will advance the limits of civil engineering designs. Once the anchorage system and hold-down devices problems are resolved, this new technology will increase the service lives of structures and will also reduce the maintenance and repair costs for future generations.
References


ACI 440.4R-04 (2004): *Prestressing Concrete Structures with FRP Tendons* (ACI 440.4R-04). American Concrete Institute, Farmington Hills.


ISIS CANADA M-05 (2005): Prestressing Concrete Structures with Fibre Reinforced Polymers (ISIS-M05-05). The Canadian Network of Centres of Excellence on Intelligent Sensing and Innovative Structures (ISIS Canada), University of Manitoba, Winnipeg.


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Appendix A

Concrete compressive strength evolution

For each concrete mixture, six cylinder samples were cast. Compression tests were performed before transfer to see if the concrete had reached its 25 MPa compressive strength, then other tests were done at 28 days to have the specified compressive strength of the concrete for the analytical calculations. Finally, the last samples were tested at the time of testing of the experimental panels to obtain the values to input in the finite element programs.

![Concrete compressive strength evolution graph](image)

Figure A.1  Concrete compressive strength evolution
Appendix B

Prestress loss calculation

B.1 Prestress loss for the DT steel panel

\[ A_{ps} = 140 \text{ mm}^2 \]
\[ L = 9144 \text{ mm} \]
\[ E_{ps} = 213333 \text{ MPa} \] From tensile tests
\[ f_{py} = 1860 \text{ MPa} \]
\[ d_n = 12 \text{ mm} \]

<table>
<thead>
<tr>
<th>Tendon position</th>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 2</td>
<td>5 6</td>
<td></td>
</tr>
<tr>
<td>4 3</td>
<td>8 7</td>
<td></td>
</tr>
</tbody>
</table>

1. Prestressing of the tendons
Hydraulic pressure applied by the jacking system (Area = 8 in²), [PSI]

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>5800 5800</td>
<td>5800 5800</td>
</tr>
<tr>
<td>5800 5800</td>
<td>5800 5800</td>
</tr>
</tbody>
</table>

Measured elongation, [mm]
Mark written after an initial prestressing of about 17 kN

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>67</td>
<td>64</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

125
Initial prestressing force, $P_e$, [kN]

<table>
<thead>
<tr>
<th></th>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>206.4</td>
<td>206.4</td>
</tr>
<tr>
<td></td>
<td>206.4</td>
<td>206.4</td>
</tr>
</tbody>
</table>

2. Prestress loss due to instantaneous loss

$g_1$: deformation of the anchor system ($\approx 1$ mm),
$g_2$: slippage in the anchorage (between 4 mm to 10 mm),
$g_3$: shortening of the steel mould ($\approx 1$ mm).

\[
g_1 = 1 \text{ mm} \\
g_2 = 4 \text{ mm} \\
g_3 = 1 \text{ mm}
\]

\[
\Delta T = \frac{\sum_{i=1}^{3} g_i}{L} A_p E_p
\]

The prestress loss of $\Delta T$, [kN]:

<table>
<thead>
<tr>
<th></th>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>19.6</td>
<td>19.6</td>
</tr>
<tr>
<td></td>
<td>19.6</td>
<td>19.6</td>
</tr>
</tbody>
</table>

3. Pouring the concrete in the mould

4. Concrete curing until the concrete compressive strength reaches at least 25 MPa

\[
f'_{co} = 32.4 \text{ MPa}
\]

5. Transfer of the prestressing force to the concrete
Prestress loss from the elastic shortening of the concrete

\[
\Delta P_{elas} = \Delta \sigma_{elas} A_p = \left(\frac{f_{cs}}{E_{ci}}\right) E_p A_p
\]

Where $E_{ci} = (3300\sqrt{f_{co}} + 6900) \left(\frac{\gamma_c}{2300}\right)^{1.5}$
\[ E_{ci} = 24762 \text{ MPa} \]

and \[ f_{cs} = \frac{P_i}{A_g} + \frac{M_p e}{I_g} + \frac{M_g e}{I_g} \]

To start the calculation, we took the assumption that: \( P_i = P_o - \Delta T \)

The instantaneous prestressing force, \( P_i \), is, [kN]:

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>186.8</td>
<td>186.8</td>
</tr>
<tr>
<td>186.8</td>
<td>186.8</td>
</tr>
</tbody>
</table>

\( f_{cs} \) (At the critical section), [MPa]:

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.93</td>
<td>3.93</td>
</tr>
</tbody>
</table>

Thus, \( \Delta P_{elas} \) equals to [kN]:

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.73</td>
<td>4.73</td>
</tr>
<tr>
<td>4.73</td>
<td>4.73</td>
</tr>
</tbody>
</table>

6. \( P_i = P_o - \Delta T - \Delta P_{elas} \), [kN]:

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>182.1</td>
<td>182.1</td>
</tr>
<tr>
<td>182.1</td>
<td>182.1</td>
</tr>
</tbody>
</table>

7. The concrete panel is then put out from the mould

8. Curing of the concrete until the panel is tested

9. Long-term loss from the shrinkage of the concrete, \( \Delta P_{shc}(t) \), [kN]

\[ \Delta P_{shc}(t) = \Delta f_{shc}(t) A_p = \varepsilon_{sh}(t) E_p A_p \]
\[ f'_c = 72 \text{ MPa} \]
\[ E'_c = 33417 \text{ MPa} \]
\[ \varepsilon_{sh} = 0.0003488 \text{ mm/mm} \]

Where \( E_c = (3300\sqrt{f'_c} + 6900) \left( \frac{\gamma_c}{2300} \right)^{1.5} \)
and \( \varepsilon_{sh} = (850 - 0.015 E_c) \times 10^6 \)

The time effect on the strain variation is calculated with:
\[ \varepsilon_{sh}(t) = \varepsilon_{sh} K_{shh} K_v K_r f_{sh}(t) \]

\( K_{shh} \) relative humidity coefficient, \( H \),
\[ K_{shh} = 1 \text{ if } H = 70\% \]
\[ H = 70 \text{ (city of Sherbrooke)} \]
\[ K_{shh} = 1 = 2 - 0.0143 H \]

\( K_v \) coefficient that consider the volume/area ratio exposed to air, \( v \),
\[ K_v = 1 \text{ if } v = 40 \text{ mm} \]
\[ K_v = 0.65 \text{ if } v > 150 \text{ mm} \]
\[ v = 93 \text{ mm} \]
\[ K_v = 0.83 = 1.13 - 0.0032 v \]

\( K_r \) reinforcement ratio coefficient, \( r \),
\[ K_r = 0.97 \text{ if } r = 0.0015 \]
\[ K_r = 0.83 \text{ if } r > 0.01 \]
\[ r = 0.0097 \]
\[ K_r = 0.84 = 1 - 17 r \]

\( f_{sh}(t) \) time function, [day],
\[ t = 147 \text{ days} \]
\[ f_{sh}(t) = 0.64 = t/(t + 0.9 v) \]

\[ \varepsilon_{sh}(t) = 0.000155 \]

The prestress loss due to the shrinkage of the concrete, \( \Delta P_{shc}(t) \), [kN]

\begin{tabular}{llll}
Stem 1 & & Stem 2 & \\
4.63 & 4.63 & 4.63 & 4.63 \\
4.63 & 4.63 & 4.63 & 4.63 \\
\end{tabular}

10. Long-term loss from the creep of the concrete, \( \Delta P_{crc}(t) \), [kN]

\[ \Delta P_{crc}(t) = \Delta f_{cr}(t) A_p = \varepsilon_{cr}(t) E_p A_p = \varepsilon_i C_{cr}(t) E_p A_p \]
$C_{cr}$ reference final creep coefficient

For $H = 70\%$, $v = 40$ mm, $r = 0$ mm and $t = 1$ day

\[ f'_c = 72 \text{ MPa} \]

$C_{cr} = 4.25 - 0.033 f'_c$ for $f'_c \leq 50$ MPa

$C_{cr} = 2.16$ for $50$ MPa $\leq f'_c \leq 80$ MPa

The time effect on the creep coefficient is calculated with:

$C_{cr}(t) = C_{cr} K_{crh} K_v K_r K_t f_{cr}(t)$

$K_{crh}$ relative humidity coefficient, $H$,

$K_{crh} = 1 \text{ if } H = 70\%$

$H = 70$ (city of Sherbrooke)

$K_{crh} = 1 = 1.70 - 0.01 H$

$K_v$ coefficient that consider the volume/area ratio exposed to air, $v$,

$K_v = 1 \text{ if } v = 40$ mm

$K_v = 0.65 \text{ if } v > 150$ mm

$K_v = 0.83 = 1.13 - 0.0032 v$

$K_r$ reinforcement ratio coefficient, $r$,

$K_r = 0.97 \text{ if } r = 0.0015$

$K_r = 0.83 \text{ if } r > 0.01$

$r = 0.0097$

$K_r = 0.84 = 1 - 17 r$

$K_t$ coefficient considering the age of the concrete at transfer, $t_{co}$,

$t_{co} = 0.69$ day

$K_t = 1.00 \text{ if } t_{co} = 1$ day

$K_t = 0.67 \text{ if } t_{co} = 28$ days

$K_t = 1.05 = (t_{co})^{-0.12}$

$f_{cr}(t)$ time function, [day],

$t = 147$ days

$f_{cr}(t) = 0.67 = t^{0.6}/(10 + t^{0.6})$

$C_{cr}(t) = 1.05$

Concrete stress at the tendons centre of gravity, $f_{cs}$, with $P_t$ total (At the critical section), [MPa]:

<table>
<thead>
<tr>
<th>Stem 1</th>
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<tbody>
<tr>
<td>3.79</td>
<td>3.79</td>
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</table>

129
Modulus of elasticity of the concrete at transfer
\[ E_{ci} = 24762 \text{ MPa} \]

Instantaneous shortening in each stem, \( \varepsilon_i \):

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
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</thead>
<tbody>
<tr>
<td>0.00015289</td>
<td>0.00015289</td>
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</table>

The prestress loss due to the creep of the concrete, \( \Delta P_{cre}(t) \), [kN]

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<td>4.78 4.78</td>
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</table>

11. Relaxation of the steel tendons, \( \Delta P_{rela} \), [kN]

\[ \Delta P_{rela}(t_i \text{ to } t_j) = \frac{P_{is}}{A_{ps}} \left( \frac{\log 24 t_j - \log 24 t_i}{45} \right) \left( \frac{P_{is}}{A_{ps} f_{psy}} - 0.55 \right) A_{ps} \]

\( t_0 = -1 \text{ day tendons initial prestressing} \)
\( t_1 = 147 \text{ days day of testing} \)

The prestress loss due to the relaxation of the steel tendons, \( \Delta P_{rela} \), [kN]

<table>
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<tr>
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<tr>
<td>2.14 2.14</td>
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<td>2.14 2.14</td>
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12. Summation of the long-term loss, \( \Delta P_{long-term} \), [kN]

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<th>Stem 1</th>
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<tbody>
<tr>
<td>11.56 11.56</td>
<td>11.56 11.56</td>
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<td>11.56 11.56</td>
<td>11.56 11.56</td>
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</tbody>
</table>

13. Effective prestressing force after all losses at the day of testing the panel, \( \Delta P_{eff} \), [kN]

\[ \Delta P_{eff} = P_i - \Delta P_{long-term} \]
14. Effectiveness of the prestressing, $m$
   Ratio $P_{\text{eff}}/P_o$

<table>
<thead>
<tr>
<th>Stem 1</th>
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<td>170.54</td>
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</tr>
</thead>
<tbody>
<tr>
<td>0.83</td>
<td>0.83</td>
</tr>
<tr>
<td>0.83</td>
<td>0.83</td>
</tr>
</tbody>
</table>

B.2 Prestress loss for the DT FRP panel

$$A_{PCFRP} = 71.8 \text{ mm}^2$$
$$L = 9144 \text{ mm}$$
$$E_{PCFRP} = 165157 \text{ MPa} \quad \text{From tensile tests}$$
$$f_{pumFRP} = 2573.7 \text{ MPa} \quad \text{From tensile tests}$$
$$d_n = 9.5 \text{ mm}$$

**Tendon position**

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>3</td>
</tr>
<tr>
<td>7</td>
<td>1</td>
</tr>
<tr>
<td>10</td>
<td>2</td>
</tr>
<tr>
<td>12</td>
<td>5</td>
</tr>
<tr>
<td>8</td>
<td>6</td>
</tr>
<tr>
<td>11</td>
<td>4</td>
</tr>
</tbody>
</table>

1. Prestressing of the tendons
   Hydraulic pressure applied by the jacking system (Area = 8 in$^2$), [PSI]

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>2200</td>
<td>2850</td>
</tr>
<tr>
<td>s.o.</td>
<td>2850</td>
</tr>
<tr>
<td>2300</td>
<td>s.o.</td>
</tr>
<tr>
<td>2300</td>
<td>2650</td>
</tr>
<tr>
<td>1900</td>
<td>0</td>
</tr>
<tr>
<td>0</td>
<td>2350</td>
</tr>
</tbody>
</table>

Measured elongation, [mm]
Mark written after an initial prestressing of about 17 kN

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>66</td>
<td>84</td>
</tr>
<tr>
<td>70</td>
<td>88</td>
</tr>
<tr>
<td>75</td>
<td>74</td>
</tr>
<tr>
<td>54</td>
<td>80</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>74</td>
</tr>
</tbody>
</table>
Initial prestressing force, \( P_0 \), [kN]

\[
\begin{array}{ccc}
    \text{Stem 1} & \text{Stem 2} \\
    75.7 \text{ s.o.} & 79.3 & 98.8 & 98.8 \text{ s.o.} \\
    79.3 & 65.0 & 0 & 91.7 & 0 & 81.0
\end{array}
\]

2. Prestress loss due to instantaneous loss

\( g_1 \): deformation of the anchor system (\( \approx 1 \) mm),
\( g_2 \): slippage in the anchorage (between 4 mm to 10 mm),
\( g_3 \): shortening of the steel mould (\( \approx 1 \) mm).

\[
\begin{align*}
    g_1 &= 1 \text{ mm} \\
    g_2 &= 4 \text{ mm} \\
    g_3 &= 0.5 \text{ mm} \\
    \Delta T &= \sum_{i=1}^{3} g_i \\
    &= \frac{1}{L} A_p E_p
\end{align*}
\]

The prestress loss of \( \Delta T \), [kN]:

\[
\begin{array}{ccc}
    \text{Stem 1} & \text{Stem 2} \\
    7.13 \text{ s.o.} & 7.13 & 7.13 & 7.13 \text{ s.o.} \\
    7.13 & 7.13 & 0 & 7.13 & 0 & 7.13
\end{array}
\]

3. Pouring the concrete in the mould

4. Concrete curing until the concrete compressive strength reaches at least 25 MPa

\[
f'_{co} = 27.9 \text{ MPa}
\]

5. Transfer of the prestressing force to the concrete

Prestress loss from the elastic shortening of the concrete

\[
\Delta P_{elas} = \Delta \sigma_{elas} A_p = \left( \frac{f_{cs}}{E_{ci}} \right) E_p A_p
\]

Where \( E_{ci} = (3300\sqrt{f_{co}} + 6900) \left( \frac{\gamma_c}{2300} \right)^{1.5} \)
\[ E_{ci} = 23492 \text{ MPa} \]

and \[ f_{cs} = \frac{P_i}{A_g} + \frac{M_p e}{I_g} + \frac{M_p}{I_g} \]

To start the calculation, we took the assumption that: \[ P_i = P_o - \Delta T \]

The instantaneous prestressing force, \( P_i \), is, \([\text{kN}]:\)

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>68.56 s.o.</td>
<td>91.69 91.69 s.o.</td>
</tr>
<tr>
<td>72.12 57.88 0</td>
<td>84.57 0 73.90</td>
</tr>
</tbody>
</table>

\( f_{cs} \) (At the critical section), \([\text{MPa}]:\)

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.421</td>
<td>0.938</td>
</tr>
</tbody>
</table>

Thus, \( \Delta P_{elas} \) equals to \([\text{kN}]:\)

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.21 s.o. 0.21</td>
<td>0.47 0.47 s.o.</td>
</tr>
<tr>
<td>0.21 0.21 0</td>
<td>0.47 0 0.47</td>
</tr>
</tbody>
</table>

6. \[ P_i = P_o - \Delta T - \Delta P_{elas}, [\text{kN}]: \]

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>68.35 s.o. 71.90</td>
<td>91.22 91.22 s.o.</td>
</tr>
<tr>
<td>71.90 57.67 0</td>
<td>84.10 0 73.42</td>
</tr>
</tbody>
</table>

7. The concrete panel is then put out from the mould

8. Curing of the concrete until the panel is tested

9. Long-term loss from the shrinkage of the concrete, \( \Delta P_{shc}(t), [\text{kN}] \)

\[ \Delta P_{shc}(t) = \Delta f_{shc}(t) A_{PCFRP} = \varepsilon_{sh}(t) E_{PCFRP} A_{PCFRP} \]
\[ f'_c = 31 \text{ MPa} \]
\[ E_c = 24377 \text{ MPa} \]
\[ \varepsilon_{sh} = 0.0004843 \text{ mm/mm} \]

Where \( E_c = (3300\sqrt{f'_c} + 6900) \left( \frac{\gamma_c}{2300} \right)^{1.5} \)
and \( \varepsilon_{sh} = (850 - 0.015 E_c) \times 10^6 \)

The time effect on the strain variation is calculated with:
\[ \varepsilon_{sh}(t) = \varepsilon_{sh} K_{shh} K_v K_r f_{sh}(t) \]

- \( K_{shh} \) relative humidity coefficient, \( H \),
  - \( K_{shh} = 1 \) if \( H = 70\% \)
  - \( H = 70 \) (city of Sherbrooke)

- \( K_v \) coefficient that consider the volume/area ratio exposed to air, \( v \),
  - \( K_v = 1 \) if \( v = 40 \text{ mm} \)
  - \( K_v = 0.65 \) if \( v > 150 \text{ mm} \)
  - \( v = 93 \text{ mm} \)
  - \( K_v = 0.83 = 1.13 - 0.0032v \)

- \( K_r \) reinforcement ratio coefficient, \( r \),
  - \( K_r = 0.97 \) if \( r = 0.0015 \)
  - \( K_r = 0.83 \) if \( r > 0.01 \)
  - \( r = 0.0094 \)
  - \( K_r = 0.84 = 1 - 17r \)

- \( f_{sh}(t) \) time function, [day],
  - \( t = 66.6 \text{ days} \)
  - \( f_{sh}(t) = 0.44 = t/(t + 0.9v) \)

- \( \varepsilon_{sh}(t) = 0.0001507 \)

The prestress loss due to the shrinkage of the concrete, \( \Delta P_{shc}(t) \), [kN]

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.79 s.o. 1.79</td>
<td>1.79 1.79 s.o.</td>
</tr>
<tr>
<td>1.79 1.79 0</td>
<td>1.79 0 1.79</td>
</tr>
</tbody>
</table>

10. Long-term loss from the creep of the concrete, \( \Delta P_{crc}(t) \), [kN]

\[ \Delta P_{crc}(t) = \Delta f_{cr}(t) A_p = \varepsilon_{cr}(t) E_p A_p = \varepsilon_i C_{cr}(t) E_p A_p \]
$C_{cr}$ reference final creep coefficient

For $H = 70\%$, $v = 40$ mm, $r = 0$ mm and $t = 1$ day

$\frac{f'_c}{C_{cr}} = 31$ MPa

$C_{cr} = 3.227$ for $f'_c \leq 50$ MPa

$C_{cr} = 3.60 - 0.020 f'_c$ for $50$ MPa $\leq f'_c \leq 80$ MPa

The time effect on the creep coefficient is calculated with:

$C_{cr}(t) = C_{cr} K_{crh} K_v K_r K_t f_{cr}(t)$

$K_{crh}$ relative humidity coefficient, $H$,

$K_{crh} = 1$ if $H = 70\%$

$H = 70$ (city of Sherbrooke)

$K_{crh} = 1 = 1.70 - 0.01H$

$K_v$ coefficient that consider the volume/area ratio exposed to air, $v$,

$K_v = 1$ if $v = 40$ mm

$K_v = 0.65$ if $v > 150$ mm

$v = 93$ mm

$K_v = 0.83 = 1.13 - 0.0032v$

$K_r$ reinforcement ratio coefficient, $r$,

$K_r = 0.97$ if $r = 0.0015$

$K_r = 0.83$ if $r > 0.01$

$r = 0.0094$

$K_r = 0.84 = 1 - 17r$

$K_t$ coefficient considering the age of the concrete at transfer, $t_{co}$,

$t_{co} = 8.63$ day

$K_t = 1.00$ if $t_{co} = 1$ day

$K_t = 0.67$ if $t_{co} = 28$ days

$K_t = 0.77 = (t_{co})^{-0.12}$

$f_{cr}(t)$ time function, [day],

$t = 66.6$ days

$f_{cr}(t) = 0.55 = t^{0.6}/(10 + t^{0.6})$

$C_{cr}(t) = 0.97$

Concrete stress at the tendons centre of gravity, $f_{cs}$, with $P_i$ total (At the critical section), [MPa]:

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.415</td>
<td>0.925</td>
</tr>
</tbody>
</table>
Modulus of elasticity of the concrete at transfer
\[ E_{ci} = 23492 \text{ MPa} \]

Instantaneous shortening in each stem, \( \varepsilon_i \):

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0000177</td>
<td>0.0000394</td>
</tr>
</tbody>
</table>

The prestress loss due to the creep of the concrete, \( \Delta P_{cre}(t) \), [kN]

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20 s.o. 0.20</td>
<td>0.45 0.45 s.o.</td>
</tr>
<tr>
<td>0.20 0.20 0</td>
<td>0.45 0 0.45</td>
</tr>
</tbody>
</table>

11. Relaxation of the CFRP tendons, \( \Delta P_{rela} \), is negligible

12. Summation of the long-term loss, \( \Delta P_{long-term} \), [kN]

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.99 s.o. 1.99</td>
<td>2.24 2.24 s.o.</td>
</tr>
<tr>
<td>1.99 1.99 0</td>
<td>2.24 0 2.24</td>
</tr>
</tbody>
</table>

13. Effective prestressing force after all losses at the day of testing the panel, \( \Delta P_{eff} \), [kN]

\[ \Delta P_{eff} = P_1 - \Delta P_{long-term} \]

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>66.36 s.o. 69.92</td>
<td>88.98 88.98 s.o.</td>
</tr>
<tr>
<td>69.92 55.68 0</td>
<td>81.86 0 71.19</td>
</tr>
</tbody>
</table>

14. Effectiveness of the prestressing, \( m \)

Ratio \( P_{eff}/P_o \)

<table>
<thead>
<tr>
<th>Stem 1</th>
<th>Stem 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.88 s.o. 0.88</td>
<td>0.90 0.90 s.o.</td>
</tr>
<tr>
<td>0.88 0.86 0</td>
<td>0.89 0 0.88</td>
</tr>
</tbody>
</table>
B.3 Prestress loss for the ST FRP panel

\[ A_{PC_{FRP}} = 71.8 \text{ mm}^2 \]
\[ L = 9144 \text{ mm} \]
\[ E_{PC_{FRP}} = 165157 \text{ MPa} \quad \text{From tensile tests} \]
\[ f_{Pu_{PC_{FRP}}} = 2573.7 \text{ MPa} \quad \text{From tensile tests} \]
\[ d_n = 9.5 \text{ mm} \]

Tendon position

<table>
<thead>
<tr>
<th>Stem</th>
<th>2</th>
<th>4</th>
<th>3</th>
<th>6</th>
<th>1</th>
<th>5</th>
</tr>
</thead>
</table>

1. Prestressing of the tendons
   Hydraulic pressure applied by the jacking system (Area = 8 in\(^2\)), [PSI]

<table>
<thead>
<tr>
<th>Stem</th>
<th>2500</th>
<th>2050</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>2000</td>
<td></td>
</tr>
</tbody>
</table>

Measured elongation, [mm]
Mark written after an initial prestressing of about 17 kN

<table>
<thead>
<tr>
<th>Stem</th>
<th>67</th>
<th>57</th>
<th>-</th>
</tr>
</thead>
<tbody>
<tr>
<td>-</td>
<td>-</td>
<td>51</td>
<td></td>
</tr>
</tbody>
</table>

Initial prestressing force, \(P_o\), [kN]

<table>
<thead>
<tr>
<th>Stem</th>
<th>88.98</th>
<th>72.96</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>71.18</td>
<td></td>
</tr>
</tbody>
</table>

2. Prestress loss due to instantaneous loss

\[ g_1 : \text{deformation of the anchor system (\(\approx\) 1 mm)}, \]
\[ g_2 : \text{slippage in the anchorage (between 4 mm to 10 mm)}, \]
\[ g_3 : \text{shortening of the steel mould (\(\approx\) 1 mm)}. \]
\[ g_1 = 1 \text{ mm} \]
\[ g_2 = 4 \text{ mm} \]
\[ g_3 = 0.5 \text{ mm} \]

\[ \Delta T = \frac{\sum_{i=1}^{3} g_i}{L} A_p E_p \]

The prestress loss of \( \Delta T \), [kN]:

\[ \begin{array}{ccc}
7.13 & 7.13 & 0 \\
0 & 0 & 7.13 \\
\end{array} \]

3. Pouring the concrete in the mould

4. Concrete curing until the concrete compressive strength reaches at least 25 MPa

\[ f'_{co} = 30.0 \text{ MPa} \]

5. Transfer of the prestressing force to the concrete
   Prestress loss from the elastic shortening of the concrete

\[ \Delta P_{elas} = \Delta \sigma_{elas} A_p = \left( \frac{f_{cs}}{E_{ci}} \right) E_p A_p \]

Where \( E_{ci} = (3300 \sqrt{f_{co}} + 6900) \left( \frac{\gamma_c}{2300} \right)^{1.5} \)

\[ E_{ci} = 24097 \text{ MPa} \]

and \[ f_{cs} = \frac{P_i}{A_g} + \frac{M_p e}{I_g} + \frac{M_g e}{I_g} \]

To start the calculation, we took the assumption that: \( P_i = P_o - \Delta T \)

The instantaneous prestressing force, \( P_i \), is, [kN]:

\[ \begin{array}{ccc}
81.84 & 65.83 & 0 \\
0 & 0 & 64.05 \\
\end{array} \]

\( f_{cs} \) (At the critical section), [MPa]:

138
Thus, $\Delta P_{elas}$ equals to [kN]:

<table>
<thead>
<tr>
<th>Stem</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.501</td>
</tr>
</tbody>
</table>

6. $P_i = P_o - \Delta T - \Delta P_{elas}$, [kN]:

<table>
<thead>
<tr>
<th>Stem</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.74 0.74 0</td>
</tr>
<tr>
<td>0 0 0.74</td>
</tr>
</tbody>
</table>

7. The concrete panel is then put out from the mould

8. Curing of the concrete until the panel is tested

9. Long-term loss from the shrinkage of the concrete, $\Delta P_{shc}(t)$, [kN]

$$\Delta P_{shc}(t) = \Delta f_{shc}(t) A_{PCFPR} = \varepsilon_{sh}(t) E_{PCFPR} A_{PCFPR}$$

$f'_c = 43.6$ MPa  
$E_c = 27585$ MPa  
$\varepsilon_{sh} = 0.0004362$ mm/mm

Where $E_c = (3300\sqrt{f'_c} + 6900) \left( \frac{\gamma_c}{2300} \right)^{1.5}$

and $\varepsilon_{sh} = (850 - 0.015 E_c) \times 10^6$

The time effect on the strain variation is calculated with:

$\varepsilon_{sh}(t) = \varepsilon_{sh} K_{shh} K_v K_r f_{sh}(t)$

$K_{shh}$ relative humidity coefficient, $H$,

$K_{shh} = 1$ if $H = 70\%$

$H = 70$ (city of Sherbrooke)

$K_{shh} = 1 = 2 - 0.0143 H$

139
$K_v$ coefficient that consider the volume/area ratio exposed to air, $v$,  
$K_v = \begin{cases} 
1 & \text{if } v = 40 \text{ mm} \\
0.65 & \text{if } v > 150 \text{ mm} \\
\end{cases} \\
v = 89 \text{ mm} \\
K_v = 0.85 = 1.13 - 0.0032v$

$K_r$ reinforcement ratio coefficient, $r$,  
$K_r = \begin{cases} 
0.97 & \text{if } r = 0.0015 \\
0.83 & \text{if } r > 0.01 \\
\end{cases} \\
r = 0.0078 \\
K_r = 0.87 = 1 - 17r$

$f_{sh}(t)$ time function, [day],  
$t = 209 \text{ days} \\
f_{sh}(t) = 0.72 = t/(t + 0.9v)$

$\varepsilon_{sh}(t) = 0.0002313$

The prestress loss due to the shrinkage of the concrete, $\Delta P_{shc}(t)$, [kN]

<table>
<thead>
<tr>
<th>Stem</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.74</td>
</tr>
<tr>
<td>2.74</td>
</tr>
<tr>
<td>0</td>
</tr>
<tr>
<td>0</td>
</tr>
<tr>
<td>0</td>
</tr>
<tr>
<td>2.74</td>
</tr>
</tbody>
</table>

10. Long-term loss from the creep of the concrete, $\Delta P_{src}(t)$, [kN]

$$\Delta P_{src}(t) = \Delta f_{cr}(t) A_p = \varepsilon_{cr}(t) E_p A_p = \varepsilon_i C_{cr}(t) E_p A_p$$

$C_{cr}$ reference final creep coefficient  
For $H = 70\%$, $v = 40 \text{ mm}$, $r = 0 \text{ mm}$ and $t = 1 \text{ day}$  
$f'_c = 43.6 \text{ MPa}$  
$C_{cr} = 2.811$  
$C_{cr} = 4.25 - 0.033 f'_c$ for $f'_c \leq 50 \text{ MPa}$  
$C_{cr} = 3.60 - 0.020 f'_c$ for $50 \text{ MPa} \leq f'_c \leq 80 \text{ MPa}$

The time effect on the creep coefficient is calculated with:

$C_{cr}(t) = C_{cr} K_{crh} K_v K_r K_t f_{cr}(t)$

$K_{crh}$ relative humidity coefficient, $H$,  
$K_{crh} = \begin{cases} 
1 & \text{if } H = 70\% \\
0.7 & \text{if } H = 70 \text{ (city of Sherbrooke)} \\
1 & = 1.70 - 0.01 H \\
\end{cases}$

140
$K_v$ coefficient that consider the volume/area ratio exposed to air, $v$,

$K_v = \begin{cases} 
1 & \text{if } v = 40 \text{ mm} \\
0.65 & \text{if } v > 150 \text{ mm} \\
0.85 & \text{if } v = 89 \text{ mm} 
\end{cases}$

$K_r$ reinforcement ratio coefficient, $r$,

$K_r = \begin{cases} 
0.97 & \text{if } r = 0.0015 \\
0.83 & \text{if } r > 0.01 \\
0.87 & \text{if } r = 0.0078 
\end{cases}
= 1 - 17r$

$K_t$ coefficient considering the age of the concrete at transfer, $t_{co}$,

$t_{co} = 3.21 \text{ day}$

$K_t = \begin{cases} 
1.00 & \text{if } t_{co} = 1 \text{ day} \\
0.67 & \text{if } t_{co} = 28 \text{ days} \\
0.87 & \text{if } (t_{co})^{-0.12} 
\end{cases}$

$f_{cr}(t)$ time function, [day],

$\begin{align*}
t &= 209 \text{ days} \\
f_{cr}(t) &= 0.71 = t^{0.6}/(10 + t^{0.6})
\end{align*}$

$C_{cr}(t) = 1.28$

Concrete stress at the tendons centre of gravity, $f_{cs}$, with $P_t$ total (At the critical section), [MPa]:

<table>
<thead>
<tr>
<th>Stem</th>
<th>1.469</th>
</tr>
</thead>
</table>

Modulus of elasticity of the concrete at transfer

$E_{ci} = 24097 \text{ MPa}$

Instantaneous shortening in each stem, $\varepsilon_i$:

<table>
<thead>
<tr>
<th>Stem</th>
<th>0.0000610</th>
</tr>
</thead>
</table>

The prestress loss due to the creep of the concrete, $\Delta P_{cr}(t)$, [kN]
11. Relaxation of the CFRP tendons, $\Delta P_{rela}$, is negligible

12. Summation of the long-term loss, $\Delta P_{long-term}$, [kN]

\[
\begin{array}{c}
\text{Stem} \\
0.92 & 0.92 & 0 \\
0 & 0 & 0.92 \\
\end{array}
\]

13. Effective prestressing force after all losses at the day of testing the panel, $\Delta P_{eff}$, [kN]

\[
\Delta P_{eff} = P_i - \Delta P_{long-term}
\]

\[
\begin{array}{c}
\text{Stem} \\
3.67 & 3.67 & 0 \\
0 & 0 & 3.67 \\
\end{array}
\]

14. Effectiveness of the prestressing, $m$

Ratio $P_{eff}/P_o$

\[
\begin{array}{c}
\text{Stem} \\
77.44 & 61.42 & 0 \\
0 & 0 & 59.64 \\
\end{array}
\]

\textbf{B.4 Prestress loss for the ST steel panel}

\begin{align*}
A_{ps} & = 140 \text{ mm}^2 \\
L & = 9144 \text{ mm} \\
E_{ps} & = 203900 \text{ MPa} \text{ From tensile tests} \\
f_{py} & = 1860 \text{ MPa} \\
d_n & = 12 \text{ mm} \\
\end{align*}

Tendon position
1. Prestressing of the tendons
Hydraulic pressure applied by the jacking system (Area = 8 in\(^2\)), [PSI]

\[
\begin{array}{c c c c c c c c c c}
\hline
Stem & 200 & 4000 & 2700 & 2000 \\
\hline
\end{array}
\]

Measured elongation, [mm]
Mark written after an initial prestressing of about 10–15 kN

\[
\begin{array}{c c c c c c c c c c}
\hline
Stem & 0 & 50 & 37 & 0 \\
\hline
\end{array}
\]

Initial prestressing force, \( P_o \), [kN]

\[
\begin{array}{c c c c c c c c c c}
\hline
Stem & 7.12 & 142.36 & 96.09 & 7.12 \\
\hline
\end{array}
\]

2. Prestress loss due to instantaneous loss

\( g_1 \): deformation of the anchor system (\( \approx 1 \) mm),
\( g_2 \): slippage in the anchorage (between 4 mm to 10 mm),
\( g_3 \): shortening of the steel mould (\( \approx 1 \) mm).

\[
\begin{align*}
g_1 & = 1 \text{ mm} \\
g_2 & = 4 \text{ mm} \\
g_3 & = 1 \text{ mm} \\
\Delta T & = \sum_{i=1}^{3} g_i \int \frac{A_p E_p}{L} \\
& \text{The prestress loss of} \Delta T, [\text{kN}]:
\end{align*}
\]
3. Pouring the concrete in the mould

4. Concrete curing until the concrete compressive strength reaches at least 25 MPa

\[ f'_{co} = 35.5 \text{ MPa} \]

5. Transfer of the prestressing force to the concrete
   Prestress loss from the elastic shortening of the concrete

\[ \Delta P_{elas} = \Delta \sigma_{elas} A_p = \left( \frac{f_{cs}}{E_{ci}} \right) E_p A_p \]

Where
\[ E_{cs} = (3300 \sqrt{f_{co}} + 6900) \left( \frac{\gamma_c}{2300} \right)^{1.5} \]

\[ E_{ci} = 25587 \text{ MPa} \]

and
\[ f_{cs} = \frac{P_t}{A_g} + \frac{M_p e}{I_g} + \frac{M_g e}{I_g} \]

To start the calculation, we took the assumption that: \( P_t = P_o - \Delta T \)

The instantaneous prestressing force, \( P_t \), is, [kN]:

<table>
<thead>
<tr>
<th>Stem</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
</tr>
<tr>
<td>123.63</td>
</tr>
<tr>
<td>77.36</td>
</tr>
<tr>
<td>0</td>
</tr>
</tbody>
</table>

\( f_{cs} \) (At the critical section), [MPa]:

<table>
<thead>
<tr>
<th>Stem</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.42</td>
</tr>
</tbody>
</table>

Thus, \( \Delta P_{elas} \) equals to [kN]:

<table>
<thead>
<tr>
<th>Stem</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
</tr>
<tr>
<td>1.58</td>
</tr>
<tr>
<td>1.58</td>
</tr>
<tr>
<td>0</td>
</tr>
</tbody>
</table>
6. \( P_t = P_o - \Delta T - \Delta P_{\text{elas}}, \text{[kN]} \):

<table>
<thead>
<tr>
<th>Stem</th>
<th>0</th>
<th>122.05</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>75.78</td>
<td>0</td>
</tr>
</tbody>
</table>

7. The concrete panel is then put out from the mould

8. Curing of the concrete until the panel is tested

9. Long-term loss from the shrinkage of the concrete, \( \Delta P_{shc}(t), \text{[kN]} \)

\[
\Delta P_{shc}(t) = \Delta f_{shc}(t) A_p = \varepsilon_{sh}(t) E_p A_p
\]

\[
f_c' = 57.8 \text{ MPa} \]
\[
E_c = 30682 \text{ MPa} \]
\[
\varepsilon_{sh} = 0.000390 \text{ mm/mm} \]

Where \( E_c = (3300\sqrt{f_c} + 6900) \left( \frac{\gamma_c}{2300} \right)^{1.5} \)

and \( \varepsilon_{sh} = (850 - 0.015 E_c) \times 10^6 \)

The time effect on the strain variation is calculated with:

\[
\varepsilon_{sh}(t) = \varepsilon_{sh} K_{shh} K_v K_r f_{sh}(t)
\]

\[
K_{shh} \quad \text{relative humidity coefficient, } H,
\]
\[
K_{shh} = 1 \quad \text{if } H = 70\%
\]
\[
K_{shh} = H = 70 \quad \text{(city of Sherbrooke)}
\]

\[
K_v \quad \text{coefficient that consider the volume/area ratio exposed to air, } v,
\]
\[
K_v = 1 \quad \text{if } v = 40 \text{ mm}
\]
\[
K_v = 0.65 \quad \text{if } v > 150 \text{ mm}
\]
\[
v = 89 \text{ mm}
\]
\[
K_v = 0.85 \quad = 1.13 - 0.0032 v
\]

\[
K_r \quad \text{reinforcement ratio coefficient, } r,
\]
\[
K_r = 0.97 \quad \text{if } r = 0.0015
\]
\[
K_r = 0.83 \quad \text{if } r > 0.01
\]
\[
r = 0.0072
\]
\[
K_r = 0.88 \quad = 1 - 17 r
\]
\( f_{sh}(t) \) time function, [day],
\[
t = 188 \text{ days}
\]
\( f_{sh}(t) = 0.70 = t/(t + 0.9 v) \)

\( \varepsilon_{sh}(t) = 0.000203 \)

The prestress loss due to the shrinkage of the concrete, \( \Delta P_{shc}(t), [kN] \)

<table>
<thead>
<tr>
<th>Stem</th>
<th>5.79</th>
<th>5.79</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5.79</td>
<td>5.79</td>
</tr>
</tbody>
</table>

10. Long-term loss from the creep of the concrete, \( \Delta P_{cre}(t), [kN] \)

\[
\Delta P_{cre}(t) = \Delta f_{cr}(t) A_p = \varepsilon_{cr}(t) E_p A_p = \varepsilon_i C_{cr}(t) E_p A_p
\]

\( C_{cr} \) reference final creep coefficient

For \( H = 70\% \), \( v = 40 \text{ mm} \), \( r = 0 \text{ mm} \) and \( t = 1 \text{ day} \)

\[
f'_c = 57.8 \text{ MPa}
\]

\( C_{cr} = 4.25 - 0.033 f'_c \) for \( f'_c \leq 50 \text{ MPa} \)

\( C_{cr} = 2.444 \) for \( 50 \text{ MPa} \leq f'_c \leq 80 \text{ MPa} \)

The time effect on the creep coefficient is calculated with:

\( C_{cr}(t) = C_{cr} K_{crh} K_v K_r K_i f_{cr}(t) \)

\( K_{crh} \) relative humidity coefficient, \( H \),

\( K_{crh} = 1 \) if \( H = 70\% \)

\( H = 70 \) (city of Sherbrooke)

\( K_{crh} = 1 = 1.70 - 0.01 H \)

\( K_v \) coefficient that consider the volume/area ratio exposed to air, \( v \),

\( K_v = 1 \) if \( v = 40 \text{ mm} \)

\( K_v = 0.65 \) if \( v > 150 \text{ mm} \)

\( v = 89 \text{ mm} \)

\( K_v = 0.85 = 1.13 - 0.0032 v \)

\( K_r \) reinforcement ratio coefficient, \( r \),

\( K_r = 0.97 \) if \( r = 0.0015 \)

\( K_r = 0.83 \) if \( r > 0.01 \)

\( r = 0.0072 \)

\( K_r = 0.88 = 1 - 17 r \)
\( K_t \) coefficient considering the age of the concrete at transfer, \( t_{co} \),
\[
K_t = \begin{cases} 
1.00 & \text{if } t_{co} = 1 \text{ day} \\
0.67 & \text{if } t_{co} = 28 \text{ days} \\
1.00 & = (t_{co})^{-0.12} 
\end{cases}
\]

\( f_{cr}(t) \) time function, [day],
\[
t = 188 \text{ days} \\
f_{cr}(t) = 0.70 = t^{0.6}/(10 + t^{0.6})
\]

\( C_{cr} (t) = 1.27 \)

Concrete stress at the tendons centre of gravity, \( f_{cs} \), with \( P_i \) total (At the critical section), [MPa]:

\[
\begin{array}{c}
\text{Stem} \\
1.37
\end{array}
\]

Modulus of elasticity of the concrete at transfer
\( E_{ci} = 25587 \text{ MPa} \)

Instantaneous shortening in each stem, \( \varepsilon_i \):

\[
\begin{array}{c}
\text{Stem} \\
0.000535
\end{array}
\]

The prestress loss due to the creep of the concrete, \( \Delta P_{crc}(t) \), [kN]

\[
\begin{array}{c}
\text{Stem} \\
0 \quad 1.93 \\
1.93 \quad 0
\end{array}
\]

11. Relaxation of the steel tendons, \( \Delta P_{rela} \), [kN]

\[
\Delta P_{rela}(t_i \text{ to } t_j) = \frac{P_{ts}}{A_{ps}} \left( \frac{\log 24 \ t_j - \log 24 \ t_i}{45} \right) \left( \frac{P_{ts}}{A_{ps}f_{pys}} - 0.55 \right) A_{ps}
\]

\( t_0 = -1 \text{ day} \) tendons initial prestressing
\( t_1 = 188 \text{ days} \) day of testing

The prestress loss due to the relaxation of the steel tendons, \( \Delta P_{rela} \), [kN]
12. Summation of the long-term loss, $\Delta P_{\text{long-term}}$, [kN]

<table>
<thead>
<tr>
<th>Stem</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.81</td>
</tr>
<tr>
<td>1.59</td>
<td></td>
</tr>
</tbody>
</table>

13. Effective prestressing force after all losses at the day of testing the panel, $\Delta P_{\text{eff}}$, [kN]

$$\Delta P_{\text{eff}} = P_i - \Delta P_{\text{long-term}}$$

<table>
<thead>
<tr>
<th>Stem</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>7.72</td>
</tr>
<tr>
<td>7.71</td>
<td></td>
</tr>
</tbody>
</table>

14. Effectiveness of the prestressing, $m$

Ratio $P_{\text{eff}} / P_o$

<table>
<thead>
<tr>
<th>Stem</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.80</td>
</tr>
<tr>
<td>0.71</td>
<td></td>
</tr>
</tbody>
</table>
Appendix C

Load to strain graphs from the strain gauges fixed on tendons, stirrups and slab reinforcement

In this appendix, we present the load to strain behaviour for all strain gauges installed on stirrups, tendons and slab reinforcement. The graphs are presented in the order that they have been fabricated and tested. This order is: DT steel panel, DT FRP panel, ST FRP panel and ST steel panel. Also, for each panel, the graphs are ordered by their loading positions. We had three loading positions until cracks first appeared (central loading position, intermediate loading position and shear loading position) and the final central loading position until the failure of the panels. The order of loading positions until cracks first appeared varies between the panels.

C.1 DT steel panel

C.1.1 Strain gauge positions in the panel
Figure C.1  Stem 1 strain gauge positions for the DT steel panel
Figure C.2  Stem 2 strain gauge positions for the DT steel panel
Figure C.3  Slab reinforcement strain gauge positions for the DT steel panel
C.1.2 Central loading until cracking

Figure C.4 Horizontal elongation at mid-span until cracking for the central loading test

Figure C.5 Strain in the slab at mid-span until cracking for the central loading test
Figure C.6  Strain in a tendon at 600 mm from mid-span until cracking for the central loading test

Figure C.7  Strain in a tendon at 1500 mm active side from mid-span until cracking for the central loading test
Figure C.8  Strain in nº6 stirrup on active side until cracking for the central loading test

Figure C.9  Strain in nº6 stirrup on passive side until cracking for the central loading test
Figure C.10  Strain in nº4 stirrup on passive side until cracking for the central loading test

Figure C.11  Strain in nº7 stirrup on passive side until cracking for the central loading test
Figure C.12  Strain in nº14 stirrup on passive side until cracking for the central loading test

Figure C.13  Strain in slab bar at 1875 mm from passive side until cracking for the central loading test
Figure C.14  Strain in slab transversal bar at 1555 mm from passive side until cracking for the central loading test

Figure C.15  Strain in slab longitudinal bar at 1555 mm from passive side until cracking for the central loading test
Figure C.16  Strain in slab bar n°8 at mid-span until cracking for the central loading test
C.1.3 Intermediate loading until cracking

![Graph of Load vs. Strain](image)

**Figure C.17** Horizontal elongation at mid-span until cracking for the intermediate loading test

![Graph of Load vs. Strain](image)

**Figure C.18** Strain in the slab at mid-span until cracking for the intermediate loading test

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Figure C.19  Strain in a tendon at 600 mm from mid-span until cracking for the intermediate loading test

Figure C.20  Strain in a tendon at 1500 mm active side from mid-span until cracking for the intermediate loading test
Figure C.21  Strain in nº6 stirrup on active side until cracking for the intermediate loading test

Figure C.22  Strain in nº6 stirrup on passive side until cracking for the intermediate loading test
Figure C.23  Strain in nº4 stirrup on passive side until cracking for the intermediate loading test

Figure C.24  Strain in nº7 stirrup on passive side until cracking for the intermediate loading test
Figure C.25 Strain in nº14 stirrup on passive side until cracking for the intermediate loading test

Figure C.26 Strain in slab bar at 1875 mm from passive side until cracking for the intermediate loading test
Figure C.27  Strain in slab transversal bar at 1555 mm from passive side until cracking for the intermediate loading test

Figure C.28  Strain in slab longitudinal bar at 1555 mm from passive side until cracking for the intermediate loading test
Figure C.29  Strain in slab bar n°8 at mid-span until cracking for the intermediate loading test
C.1.4 Shear loading until cracking

Figure C.30  Horizontal elongation at mid-span until cracking for the shear loading test

Figure C.31  Strain in the slab at mid-span until cracking for the shear loading test
Figure C.32 Strain in a tendon at 600 mm from mid-span until cracking for the shear loading test

Figure C.33 Strain in a tendon at 1500 mm active side from mid-span until cracking for the shear loading test
Figure C.34  Strain in n°6 stirrup on active side until cracking for the shear loading test

Figure C.35  Strain in n°6 stirrup on passive side until cracking for the shear loading test
Figure C.36  Strain in n°4 stirrup on passive side until cracking for the shear loading test.

Figure C.37  Strain in n°7 stirrup on passive side until cracking for the shear loading test.
Figure C.38  Strain in n°14 stirrup on passive side until cracking for the shear loading test

Figure C.39  Strain in slab bar at 1875 mm from passive side until cracking for the shear loading test
Figure C.40  Strain in slab transversal bar at 1555 mm from passive side until cracking for the shear loading test

Figure C.41  Strain in slab longitudinal bar at 1555 mm from passive side until cracking for the shear loading test
Figure C.42  Strain in slab bar n°8 at mid-span until cracking for the shear loading test
C.1.5 Central loading until failure

![Graph showing load vs. strain for LVDT 2521 and LVDT 2522.]

Figure C.43 Horizontal elongation at mid-span at failure for the central loading test

![Graph showing load vs. strain with markers J-69, J-85, and J-83.]

Figure C.44 Strain in the slab at mid-span at failure for the central loading test
Figure C.45  Strain on tendon at 600 mm from mid-span at failure for the central loading test

Figure C.46  Strain on tendon at 1500 mm active side from mid-span at failure for the central loading test
Figure C.47  Strain in nº6 stirrup on active side at failure for the central loading test

Figure C.48  Strain in nº6 stirrup on passive side at failure for the central loading test
Figure C.49  Strain in nº4 stirrup on passive side at failure for the central loading test

Figure C.50  Strain in nº7 stirrup on passive side at failure for the central loading test
Figure C.51  Strain in n°14 stirrup on passive side at failure for the central loading test

Figure C.52  Strain in slab bar at 1875 mm from passive side at failure for the central loading test
Figure C.53  Strain in slab transversal bar at 1555 mm from passive side at failure for the central loading test

Figure C.54  Strain in slab longitudinal bar at 1555 mm from passive side at failure for the central loading test
Figure C.55  Strain in slab bar n°8 at mid-span at failure for the central loading test
C.2 DT FRP panel

C.2.1 Strain gauge positions in the panel

Figure C.56 Stem 1 strain gauge positions for the DT FRP panel
Figure C.57   Stem 2 strain gauge positions for the DT FRP panel
Figure C.58  Slab reinforcement strain gauge positions for the DT FRP panel
C.2.2 Intermediate loading until cracking

Figure C.59 Horizontal elongation at mid-span until cracking for the intermediate loading test

Figure C.60 Strain in the slab at mid-span until cracking for the intermediate loading test
Figure C.61  Strain in bottom tendon at mid-span until cracking for the intermediate loading test

Figure C.62  Strain in bottom tendon at passive support until cracking for the intermediate loading test
Figure C.63  Strain in n°6 stirrup on active side until cracking for the intermediate loading test

Figure C.64  Strain in n°14 stirrup on active side stem 1 until cracking for the intermediate loading test
Figure C.65 Strain in n°14 stirrup on active side stem 2 until cracking for the intermediate loading test

Figure C.66 Strain in slab bar at 1875 mm from active side until cracking for the intermediate loading test
Figure C.67  Strain in slab transversal bar at 1555 mm from active side until cracking for the intermediate loading test

Figure C.68  Strain in slab longitudinal bar at 1555 mm from active side until cracking for the intermediate loading test
Figure C.69  Strain in slab bar no. 8 at mid-span until cracking for the intermediate loading test
C.2.3 Central loading until cracking

Figure C.70  Horizontal elongation at mid-span until cracking for the central loading test

Figure C.71  Strain in the slab at mid-span until cracking for the central loading test
Figure C.72  Strain in bottom tendon at mid-span until cracking for the central loading test.

Figure C.73  Strain in bottom tendon at passive support until cracking for the central loading test.
Figure C.74  Strain in n°6 stirrup on active side until cracking for the central loading test

Figure C.75  Strain in n°14 stirrup on active side stem 1 until cracking for the central loading test
Figure C.76  Strain in n°14 stirrup on active side stem 2 until cracking for the central loading test

Figure C.77  Strain in slab bar at 1875 mm from active side until cracking for the central loading test
Figure C.78  Strain in slab transversal bar at 1555 mm from active side until cracking for the central loading test

Figure C.79  Strain in slab longitudinal bar at 1555 mm from active side until cracking for the central loading test
Figure C.80  Strain in slab bar no8 at mid-span until cracking for the central loading test
C.2.4 Shear loading until cracking

Figure C.81  Horizontal elongation at mid-span until cracking for the shear loading test

Figure C.82  Strain in the slab at mid-span until cracking for the shear loading test
Figure C.83  Strain in bottom tendon at mid-span until cracking for the shear loading test

Figure C.84  Strain in nº6 stirrup on active side until cracking for the shear loading test
Figure C.85  Strain in nº6 stirrup on passive side until cracking for the shear loading test

Figure C.86  Strain in nº4 stirrup on passive side until cracking for the shear loading test
Figure C.87  Strain in n°7 stirrup on passive side until cracking for the shear loading test

Figure C.88  Strain in n°14 stirrup on active side stem 1 until cracking for the shear loading test
Figure C.89  Strain in №14 stirrup on active side stem 2 until cracking for the shear loading test

Figure C.90  Strain in slab bar at 1875 mm from active side until cracking for the shear loading test
Figure C.91  Strain in slab longitudinal bar at 1555 mm from active side until cracking for the shear loading test

Figure C.92  Strain in slab bar n°8 at mid-span until cracking for the shear loading test
C.2.5 Central loading until failure

Figure C.93  Horizontal elongation at mid-span at failure for the central loading test

Figure C.94  Strain in the slab at mid-span at failure for the central loading test
Figure C.95  Strain in bottom tendon at mid-span at failure for the central loading test

Figure C.96  Strain in nº6 stirrup on active side at failure for the central loading test
Figure C.97  Strain in n°6 stirrup on passive side at failure for the central loading test

Figure C.98  Strain in n°4 stirrup on passive side at failure for the central loading test
Figure C.99  Strain in n°7 stirrup on passive side at failure for the central loading test

Figure C.100  Strain in n°14 stirrup on active side stem 1 at failure for the central loading test
Figure C.101  Strain in nº14 stirrup on active side stem 2 at failure for the central loading test

Figure C.102  Strain in slab bar at 1875 mm from active side at failure for the central loading test
Figure C.103  Strain in slab longitudinal bar at 1555 mm from active side at failure for the central loading test

Figure C.104  Strain in slab bar no8 at mid-span at failure for the central loading test
C.3 ST FRP panel

C.3.1 Strain gauge positions in the panel

Figure C.105 Stem strain gauge positions for the ST FRP panel
Figure C.106  LVDT's position for the ST FRP panel
C.3.2 Central loading until cracking

Figure C.107 Strain in the slab at mid-span until cracking for the central loading test

Figure C.108 Strain on the top slab at mid-span until cracking for the central loading test
Figure C.109  Strain on the stem at mid-span until cracking for the central loading test

Figure C.110  Strain on tendons at mid-span until cracking for the central loading test
Figure C.111  Strain on concrete at the same level as the tendons at mid-span until cracking for the central loading test

Figure C.112  Strain on stem 2 tendons at passive side until cracking for the central loading test
Figure C.113  Strain on tendon at 1500 mm from active side until cracking for the central loading test

Figure C.114  Strain in n°6 stirrup on passive side until cracking for the central loading test
Figure C.115  Strain in nº4 stirrup on active side until cracking for the central loading test

Figure C.116  Strain in nº6 stirrup on active side until cracking for the central loading test
Figure C.117  Strain in n°7 stirrup on active side until cracking for the central loading test

Figure C.118  Strain in 45° strain gauges on concrete at active support until cracking for the central loading test
Figure C.119  Strain in nº14 stirrup on passive side until cracking for the central loading test

Figure C.120  Strain in slab longitudinal bar at 1555 mm from passive side until cracking for the central loading test
C.3.3 Intermediate loading until cracking

Figure C.121 Strain in the slab at mid-span until cracking for the intermediate loading test

Figure C.122 Strain on the top slab at mid-span until cracking for the intermediate loading test

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Figure C.123  Strain on the stem at mid-span until cracking for the intermediate loading test

Figure C.124  Strain on tendons at mid-span until cracking for the intermediate loading test
Figure C.125  Strain on concrete at the same level as the tendons at mid-span until cracking for the intermediate loading test

Figure C.126  Strain on tendon at 1500 mm from active side until cracking for the intermediate loading test
Figure C.127 Strain in n°6 stirrup on passive side until cracking for the intermediate loading test

Figure C.128 Strain in n°4 stirrup on active side until cracking for the intermediate loading test
Figure C.129  Strain in no.6 stirrup on active side until cracking for the intermediate loading test

Figure C.130  Strain in no.7 stirrup on active side until cracking for the intermediate loading test
Figure C.131  Strain in 45° strain gauges on concrete at active support until cracking for the intermediate loading test

Figure C.132  Strain in nº14 stirrup on passive side until cracking for the intermediate loading test
Figure C.133  Strain in slab longitudinal bar at 1555 mm from passive side until cracking for the intermediate loading test.
C.3.4 Shear loading until cracking

Figure C.134 Strain in the slab at mid-span until cracking for the shear loading test

Figure C.135 Strain on the top slab at mid-span until cracking for the shear loading test
Figure C.136  Strain on the stem at mid-span until cracking for the shear loading test

Figure C.137  Strain on tendons at mid-span until cracking for the shear loading test
Figure C.138  Strain on concrete at the same level as the tendons at mid-span until cracking for the shear loading test

Figure C.139  Strain in n°6 stirrup on passive side until cracking for the shear loading test
Figure C.140 Strain in nº4 stirrup on active side until cracking for the shear loading test

Figure C.141 Strain in nº6 stirrup on active side until cracking for the shear loading test
Figure C.142  Strain in n°7 stirrup on active side until cracking for the shear loading test

Figure C.143  Strain in 45° strain gauges on concrete at active support until cracking for the shear loading test
Figure C.144  Strain in nº14 stirrup on passive side until cracking for the shear loading test

Figure C.145  Strain in slab longitudinal bar at 1555 mm from passive side until cracking for the shear loading test
C.3.5 Central loading until failure

Figure C.146 Strain in the slab at mid-span at failure for the central loading test

Figure C.147 Strain on the top slab at mid-span at failure for the central loading test
Figure C.148  Strain on the stem at mid-span at failure for the central loading test

Figure C.149  Strain on tendons at mid-span at failure for the central loading test
Figure C.150  Strain on concrete at the same level as the tendons at mid-span at failure for the central loading test

Figure C.151  Strain in nº6 stirrup on passive side at failure for the central loading test
Figure C.152 Strain in n°4 stirrup on active side at failure for the central loading test.

Figure C.153 Strain in n°6 stirrup on active side at failure for the central loading test.
Figure C.154  Strain in n°7 stirrup on active side at failure for the central loading test

Figure C.155  Strain in 45° strain gauges on concrete at active support at failure for the central loading test
Figure C.156  Strain in n°14 stirrup on passive side at failure for the central loading test

Figure C.157  Strain in slab longitudinal bar at 1555 mm from passive side at failure for the central loading test
C.4 ST steel panel

C.4.1 Strain gauge positions in the panel

Figure C.158 Stem strain gauge positions for the ST steel panel
Figure C.159  LVDT's position for the ST steel panel
C.4.2 Central loading until cracking

Figure C.160 Strain in the slab at mid-span until cracking for the central loading test

Figure C.161 Strain on the top slab at mid-span until cracking for the central loading test
Figure C.162  Strain on the stem at mid-span until cracking for the central loading test

Figure C.163  Strain on tendons at mid-span until cracking for the central loading test
Figure C.164  Strain on concrete at the same level as the tendons at mid-span until cracking for the central loading test.

Figure C.165  Strain on tendons at passive side until cracking for the central loading test.
Figure C.166  Strain on tendon at 1500 mm from active side until cracking for the central loading test

Figure C.167  Strain in n°6 stirrup on active side until cracking for the central loading test
Figure C.168  Strain in nº4 stirrup on passive side until cracking for the central loading test

Figure C.169  Strain in nº6 stirrup on passive side until cracking for the central loading test
Figure C.170  Strain in nº7 stirrup on passive side until cracking for the central loading test

Figure C.171  Strain in 45° strain gauges on concrete at passive support until cracking for the central loading test
Figure C.172  Strain in n°14 stirrup on active side until cracking for the central loading test

Figure C.173  Strain in slab longitudinal bar at 1555 mm from passive side until cracking for the central loading test
C.4.3 Intermediate loading until cracking

The intermediate loading test until cracking was not performed for the ST steel panel due to time restriction in the laboratory schedule, so no load to strain graph are presented in this section.

C.4.4 Shear loading until cracking

Figure C.174 Strain in the slab at mid-span until cracking for the shear loading test
Figure C.175  Strain on the top slab at mid-span until cracking for the shear loading test

Figure C.176  Strain on the stem at mid-span until cracking for the shear loading test
Figure C.177  Strain on tendons at mid-span until cracking for the shear loading test

Figure C.178  Strain on concrete at the same level as the tendons at mid-span until cracking for the shear loading test
Figure C.179  Strain on tendons at passive side until cracking for the shear loading test

Figure C.180  Strain on tendon at 1500 mm from active side until cracking for the shear loading test
Figure C.181 Strain in n°6 stirrup on active side until cracking for the shear loading test

Figure C.182 Strain in n°4 stirrup on passive side until cracking for the shear loading test
Figure C.183 Strain in nº6 stirrup on passive side until cracking for the shear loading test.

Figure C.184 Strain in nº7 stirrup on passive side until cracking for the shear loading test.
Figure C.185  Strain in 45° strain gauges on concrete at passive support until cracking for the shear loading test

Figure C.186  Strain in n°14 stirrup on active side until cracking for the shear loading test
Figure C.187  Strain in slab longitudinal bar at 1555 mm from passive side until cracking for the shear loading test
C.4.5 Central loading until failure

Figure C.188 Strain in the slab at mid-span at failure for the central loading test

Figure C.189 Strain on the top slab at mid-span at failure for the central loading test
Figure C.190  Strain on the stem at mid-span at failure for the central loading test

Figure C.191  Strain on tendons at mid-span at failure for the central loading test
Figure C.192  Strain on concrete at the same level as the tendons at mid-span at failure for the central loading test

Figure C.193  Strain on tendons at passive side at failure for the central loading test
Figure C.194  Strain on tendon at 1500 mm from active side at failure for the central loading test

Figure C.195  Strain in n°6 stirrup on active side at failure for the central loading test
Figure C.196  Strain in n°4 stirrup on passive side at failure for the central loading test

Figure C.197  Strain in n°6 stirrup on passive side at failure for the central loading test
Figure C.198  Strain in n°7 stirrup on passive side at failure for the central loading test

Figure C.199  Strain in 45° strain gauges on concrete at passive at failure cracking for the central loading test
Figure C.200  Strain in n°14 stirrup on active side at failure for the central loading test

Figure C.201  Strain in slab longitudinal bar at 1555 mm from passive side at failure for the central loading test