BEHAVIOR OF PRECAST CONCRETE TUNNEL SEGMENTS REINFORCED WITH GFRP BARS UNDER PUNCHING AND SHEAR LOADS

ÉTUDE DU COMPORTEMENT DES VOUSSOIRS DE TUNNEL PRÉFABRIQUÉS EN BÉTON ARMÉ D’ARMATURE EN PRFV SOUMIS À DES CHARGES DE POINÇONNEMENT ET DE CISAILLEMENT

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DEDICATION

In the name of Allah, the most merciful, the most compassionate

This dissertation work is dedicated to my parents (may Allah mercy them) who had always loved me unconditionally

To my beloved wife and my son who sacrifice everything for the succession of this work

To my brother and sister
ABSTRACT

Tunnels are one of the types of infrastructure extensively developed in recent decades to improve the mobility of people and goods, as well as for utility purposes. Substituting the conventional in situ lining technique with precast concrete tunnel lining (PCTL) segments in infrastructure projects maintains time and emphasizes workers’ safety with superior quality. The main durability issue facing PCTL segments is the corrosion of the steel reinforcement due to harsh environmental conditions. Corrosion of embedded reinforcing bars and the related deterioration of PCTL segments are the primary modes that accelerate failure and can jeopardize structural integrity. This premature degradation requires costly repairs and rehabilitation for tunnels. The use of glass fiber-reinforced polymer (GFRP) bars in PCTLs as an alternative to corroding steel bars has emerged as a realistic, cost-effective solution, particularly when high resistance to environmental attack is required. In recent years, few studies have been conducted on bending and thrust tests to simulate the provisional and permanent loads that affect the life cycle of GFRP-reinforced PCTLs. Additionally, current design provisions do not address the design of PCTL segments reinforced internally with FRP bars. To date, the punching and shear behavior of GFRP-reinforced PCTL segments with different parameters have not been investigated.

In this research study, the experimental program consisted of two phases. The first phase was conducted on an investigation of the punching behavior of PCTL segments reinforced with GFRP bars and ties. A total of eight full-scale rhomboidal PCTL segments measuring 1500 mm in width and 250 mm in thickness, and with an arc length of 3100 mm and 2100 mm were designed, cast, and tested under concentrated load until failure. The second phase included testing of nine (five convex and four concave) full-scale PCTL segments to investigate the shear behavior of tunnel segments under fabrication and service loads. Segments were constructed with a rhomboidal shape measuring 2100 x 1500 x 250 mm and subjected to line loading on their extrados and intrados surfaces up to failure. The test parameters were the reinforcement type (GFRP and steel), reinforcement ratio, concrete strength (NSC and HSC), transverse reinforcement configuration (closed versus bars), arc length, and shear reinforcement. The test results were carefully analyzed in terms of punching and shear strength, cracking behavior, failure modes, load-deflection response, and strains in reinforcement and concrete. The effects of test parameters on punching and shear behavior were discussed and evaluated. A theoretical prediction of the punching...
capacity, shear capacity, and deflection using current models were compared to the experimental 
results obtained herein. In addition, modifications in existing models were provided to consider 
the influence of curved geometry of GFRP-reinforced PCTL segments with closed ties. The results 
reveal that the GFRP-reinforced PCTL segment was comparable with its steel counterpart with the 
same reinforcement ratio and satisfied serviceability limits. Increasing the reinforcement ratio and 
concrete strength increased the punching and shear strengths. Decreasing the segment arc length 
and using shear reinforcement enhanced the punching response. The shear behavior can be further 
improved by using closed ties. The theoretical outcomes show the suitability of using current FRP 
design provisions for predicting the punching capacity of PCTL segments reinforced with GFRP 
bars. The modified models of shear capacity and deflection accurately predicted the experimental 
results.

**Keywords:** precast concrete tunnel lining (PCTL) segments; glass fiber-reinforced polymer 
(GFRP) bars; punching; shear; reinforcement ratio; concrete strength; shear reinforcement; 
prediction; deflection; modified models.
RÉSUMÉ

Les tunnels sont l'un des types d'infrastructures largement développés au cours des dernières décennies pour améliorer la mobilité des personnes et des marchandises, ainsi qu'à des fins utilitaires. Le remplacement de la technique conventionnelle de revêtement in situ par des voussoirs de revêtement de tunnels en béton préfabriqué (RTBP) permet de gagner du temps et de mettre l'accent sur la sécurité des travailleurs avec une qualité supérieure. Le principal problème auquel sont confrontés les voussoirs de RTBP est la corrosion de l'armature d'acier due aux conditions environnementales. La corrosion des barres d'armature d'acier et la détérioration des voussoirs de RTBP constitue la principale cause de défaillance qui pourrait mettre en péril l'intégrité structurelle du tunnel. Cette dégradation due à la corrosion d'armature d’acier nécessite des réparations et des réhabilitations coûteuses pour les tunnels. L'utilisation de barres d’armature en polymère renforcé de fibres de verre (PRFV) dans les voussoirs de RTBP comme alternative aux barres d'acier constitue une bonne solution, en particulier lorsqu’une résistance élevée aux agressions environnementales est requise. Très peu d'études et de travaux de recherche ont été menés sur le comportement en flexion et de poussée de voussoirs de revêtement de tunnels en béton préfabriqué (RTBP) armé d’armature en polymère renforcé de fibres de verre (PRFV). Ces charges de flexion et de poussée représentent les charges provisoires et permanentes dans les structures de revêtement de tunnels. En outre, les dispositions de conception actuelles dans les normes ne sont pas applicables à la conception de voussoirs de revêtement de tunnels en béton préfabriqué (RTBP) armé d’armature en polymère renforcé de fibres de verre (PRFV). La résistance et le comportement des voussoirs de revêtements de tunnels en béton préfabriqué (RTBP) armé d’armature en PRFV sous des charges de poinçonnement de l’effort tranchant (cisaillement) est un domaine dans lequel aucun résultat de recherche expérimentale n'est disponible.

Le programme expérimental de cette thèse comprend deux phases. La première phase a consisté à étudier le comportement au poinçonnement de voussoirs de RTBP armé d’armature longitudinale et transversale en PRFV. Un total de huit voussoirs de RTBP à pleine échelle ayant une longueur de 3100 mm et 2100 mm, une largeur de 1500 mm et une épaisseur de 250 mm ont été construits et testés sous des charges concentrées jusqu'à la rupture. La deuxième phase comprenait, quant à elle, neuf voussoirs de RTBP à pleine échelle (cinq convexes et quatre concaves) ayant une
longueur de 2100 mm, une largeur de 1500 mm et une épaisseur de 250 mm pour étudier le comportement à l’effort tranchant. Les spécimens ont été soumis à une charge linéaire sur leurs surfaces extrados et intrados jusqu'à la rupture. Les paramètres d'essai comprenaient le type d'armature (PRFV et acier), le taux d'armature, la résistance du béton (béton a résistance normale BRN et béton a haute résistance BHR), la configuration de l'armature transversale ((cadres fermés par rapport aux cadres doubles en forme de U), la longueur de l'arc et l'armature transversale. Les résultats des essais ont été analysés en termes de résistance au poinçonnement et à l’effort tranchant, de comportement à la fissuration, de modes de rupture, de réponse charge-déformation et de déformations dans l'armature et le béton. Les effets des paramètres d'essai sur le comportement au poinçonnement et à l’effort tranchant ont été discutés et analysés. Une prédiction théorique de la résistance au poinçonnement et à l’effort tranchant cisaillement ainsi que de la déflexion en utilisant des modèles existants a été réalisée et comparée aux résultats expérimentaux. En outre, des modifications ont été apportées aux modèles existants afin de prendre en compte l'influence de la géométrie desvoussoirs de revêtement de tunnels en béton préfabriqué (RTBP) armé d’armature en PRFV. Les résultats obtenus ont montré que le comportement de voussoirs de revêtement de tunnels en béton préfabriqué (RTBP) armé d’armature en PRFV est comparable à celui des voussoirs de revêtement de tunnels en béton préfabriqué (RTBP) armé d’armature d’acier pour un taux d’armature équivalent. L'augmentation du taux d’armature et de la résistance du béton a comme attendu fait augmenter les résistances au poinçonnement et à l’effort tranchant. Par ailleurs, les résultats obtenus ont montré que la diminution de la longueur de l'arc du voussoir et l'utilisation d’armature transversale ont fait accroître la résistance au poinçonnement. Aussi, le comportement et la résistance à l’effort tranchant se trouvent amélioré par l’utilisation d’armature transversale constituée de cadres fermés. Enfin, les analyses théoriques ont montré que l'utilisation des équations dans les normes de conception traitant de l’armature en PRFV prédisent très bien la résistance au poinçonnement des voussoirs de revêtement de tunnels en béton préfabriqué (RTBP) armé d’armature en PRFV. De nouveaux modèles ont été mis au point pour mieux prédire la résistance à l’effort tranchant et de déflexion de voussoirs de revêtements de tunnels en béton préfabriqué (RTBP) armé d’armature en PRFV.

**Mots clés :** des voussoirs de revêtements de tunnels en béton préfabriqué (RTBP); barres en polymère renforcé de fibres de verre (PRFV); résistance au poinçonnement; résistance à l’effort
tranchant; taux d'armature; résistance en compression du béton; armature de cisaillement; déflexion; modélisations, codes de conception.
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NOTATION

$A_f$ = nominal cross-sectional area (mm²)
$A_{fr,\text{min}}$ = minimum amount of FRP shear reinforcement within spacing $s$ (mm²)
$b_w$ = width of a section (mm)
$b_{o;0.5d}$ = critical perimeter at distance of 0.5$d$ from plate face (mm)
$b_{o;1.5d}$ = critical perimeter at distance of 1.5$d$ from plate face (mm)
$d$ = average effective segment depth (mm)
$d_h$ = nominal diameter (mm)
$d_c$ = effective depth of segment according to CSA S806-12 (max of 0.72$h$ or 0.9$d$) (mm)
$E_c$ = elastic modulus of concrete (MPa) (=4700 $\sqrt{f'_c}$)
$E_f$ = elastic modulus of FRP (GPa)
$E_s$ = elastic modulus of steel (MPa)
$f'_c$ = average cylinder concrete compressive strength (MPa)
$f_{cu}$ = average cubic concrete compressive strength (MPa)
$f_{tu}$ = ultimate tensile strength of FRP (MPa)
$f_{tv}$ = stress in FRP ties (MPa)
$h$ = thickness of segment (mm)
$K_{\text{pre}}$ = pre-cracking stiffness (kN/mm)
$K_{\text{post}}$ = post-cracking stiffness (kN/mm)
$k$ = the ratio of the neutral-axis depth to the reinforcement depth
$M_d$ = design bending moment (kN.m)
$M_{\text{exp}}$ = experimental bending moment (kN.m)
$M_{\text{theo}}$ = theoretical bending moment (kN.m)
$l$ = distance between reaction forces acting on centerline (mm)
$n_f$ = modular ratio ($E_f/E_c$)
$u$ = perimeter of the loaded area (mm)
$V_c$ = ultimate punching-shear capacity provided by concrete (kN)
$V_{cf}$ = calculated shear capacity carried by concrete (kN)
$V_{\text{csc}}$ = shear load carried by a critical shear crack (kN)
$V_{cr}$ = first cracking load (kN)
$V_{\text{exp}}$ = experimental shear load (kN)
$V_{\text{pred}}$ = predicted shear capacity (kN)
$V_{r,c}$ = concrete shear strength (kN)
$V_{sf}$ = calculated shear capacity carried by closed ties (kN)
$V_u$ = ultimate punching shear load (measured in test) (kN)
$v_c$ = factored shear resistance provided by concrete (MPa)
$v_r$ = factored shear resistance (MPa)
$v_u$ = ultimate shear stress calculated at plate face (MPa)
$W_{\text{tmax}}$ = maximum crack width (mm)
$x$ = horizontal distance from vertical axis to the point of reaction on centerline (mm)
$\Delta$ = vertical distance from the centerline of segment to supports’ reactions (mm)
$\Delta_{cr}$ = deflection corresponding to cracking load (mm)
$\Delta_u$ = deflection corresponding to ultimate load (mm)
$\Delta V$ = deviation forces along arc-shaped elements (kN)
$\varepsilon_{fu}$ = ultimate tensile Strain ($\mu$e)
$\varepsilon_{max}$ = ultimate measured reinforcement strain ($\mu$e)
$\varepsilon_{cmax}$ = ultimate measured concrete strain ($\mu$e)
$\rho_b$ = balanced reinforcement ratio
$\rho$ = reinforcement ratio = $A_f / (1500 \times d)$
$\rho_{fv}$ = shear reinforcement ratio
$\alpha_s$ = 4 for interior columns
$\beta_c$ = factor that adjusts $v_c$ for support dimensions (=1 for circular plate)
$\lambda$ = factor to account for the concrete density
$\gamma_b$ = factor of safety (=1.3)
$\phi$ = inclination angle of support reaction to the vertical axis during testing (degree)
$\phi_c$ = resistance factor for concrete
CHAPTER 1
INTRODUCTION

1.1 General

Precast concrete tunnel lining (PCTL) segments are currently widely used in infrastructure projects (i.e., roadways, subways, gas pipelines, and wastewater tunnels) because they save time, satisfy the safety requirements, and offer high quality. The use of PCTL segments in tunneling projects has been increasing because of its efficient and economical application compared to the conventional in situ lining technique (Elliott et al. 2002). Tunnel function depends on the efficiency, structural performance, and durability of its lining systems. The objective of designing PCTL segments is to develop suitable criterions to make the tunnel structures stable during their intended lives. Tunnel structures built with steel-reinforced concrete are designed for 100 years of service life. The main durability issue facing PCTL segments, however, is the corrosion of the steel reinforcement due to harsh environmental conditions. The durability of underground structures refers to their resistance to surrounding exposure effects such as seawater or the presence of chloride-contaminated water. The penetration of chloride ions through the surfaces of PCTLs—whether from groundwater or deicing salts—can disturb the passive layer around the reinforcing bars, resulting in corrosion initiation and a reduction in load-carrying capacity (Abbas et al. 2014). Corrosion of embedded reinforcing bars can induce internal pressures in the concrete. Once that occurs, the concrete cover could spall due to micro- and macrocracking, which might lead to catastrophic failure. In fact, the corrosion of steel reinforcement and the related deterioration of PCTL segments are the primary modes that accelerate failure and can jeopardize structural integrity, requiring costly repairs and rehabilitation. Adopting glass fiber-reinforced polymer (GFRP) bars in PCTL segments as an alternative to corrodible steel reinforcement is considered a realistic and cost-effective solution, particularly in harsh environmental conditions surrounding tunnels. GFRP materials present high tensile strength, high strength-to-weight ratio, long service life, and neutrality to electrical and magnetic disturbances, while improving the life-cycle cost efficiency (ACI Committee 440; Benmokrane et al. 2021). These nonmetallic properties can reduce the concrete cover and create
dielectric joints in PCTLs. Consequently, segments reinforced with GFRP bars avoid concrete crushing during handling and guarantee an effective remedy to corrosion (Caratelli et al. 2016).

ACI 544.7R-16 and 533.5R-20 divided the loading conditions of PCTLs into three main stages: production and transient stage loads, loads for the construction stage, and loads for service stages. Recently, limited studies have been conducted to investigate the possibility of using GFRP reinforcement in tunnel segments (Moccichino et al. 2010; Caratelli et al. 2016, 2017; Spagnuolo et al. 2017, 2018; Meda et al. 2019; Hosseini et al. 2022a, 2022b; Ibrahim et al. 2023). Bending and edge thrust tests were conducted on full-scale PCTLs to assess their structural response. All these studies revealed the effectiveness and safety of employing GFRP bars in PCTL segments, especially for tunnel parts that have to be eventually removed (openings, niches, vent channels, and stations) to facilitate demolition and disposal. The laboratory results demonstrate the enhanced structural and durability performance of the GFRP-reinforced PCTL segments in terms of cracking behavior, deflection, and ultimate strength. Moreover, substituting GFRP reinforcement for conventional steel reinforcement satisfied the structural requirements. The punching phenomenon is one of the complex circumstances that can be induced by rock expansion or other geotechnical conditions above or beneath the tunnel. On the other hand, shear force is one of the straining actions that can be induced by the provisional and permanent loads on tunnels.

In the literature, very limited data is available on the punching and shear responses of PCTL segments. Furthermore, the current design provisions in ACI 544.7R-16, ACI 533.5R-20, CSA S806-12, and ACI 440.11-22 do not address the design of PCTL segments reinforced internally with FRP bars. The curved geometry of PCTLs results in the applied load and stresses around the loaded region differing from those of flat slabs and non-curvature beams. The punching and shear behavior of these elements can, however, be used to understand the response of tunnel segments. GFRP-reinforced structures had lower punching and shear capacities than their counterparts reinforced with steel bars due to the lower modulus of GFRP bars (Eladawy et al. 2020; Ali et al. 2017). Therefore, experimental and theoretical investigations to study the structural performance of GFRP-reinforced PCTL segments subjected to punching and shear loads are needed.
1.2 Research Significance

Using GFRP bars in PCTL segments is considered an alternative solution to corroding steel reinforcement, primarily due to their durability and cost-effectiveness in the harsh environmental conditions surrounding tunnels. No research, however, seems to have investigated PCTL segments reinforced with GFRP bars and closed ties under punching and shear loads. In addition, the results for GFRP-RC slabs/beams cannot be directly applicable to GFRP-reinforced PCTL segments—given their curved geometry—particularly when investigating the effect of their numerous parameters. Furthermore, current tunnel design provisions in (ACI 544.7R-16, ACI 533.5R-20, and fib bulletin 83) do not consider the shear resistance of transverse reinforcement PCTL segments and are often designed as a minimum reinforcement for shrinkage and temperature resistance. This study—which presents the first experimental results of full-scale precast RC segments reinforced with GFRP bars and closed ties—contributes to understanding the general punching and shear behavior of such elements. It also aimed to evaluate the effects of various parameters on punching strength as well as the contributions of transverse reinforcement and the arched shape on GFRP-reinforced PCTL segments. A theoretical investigation was conducted to examine the accuracy of the current models and propose modifications for tunnel segments. The findings pave the way for the use of PCTL segments reinforced with curvilinear GFRP bars in infrastructure applications.

1.3 Research Objectives

The aim of this work is to evaluate the feasibility of using GFRP bars as internal reinforcement for PCTL segments, both experimentally and theoretically. This study consists of two phases. In Phase I, eight GFRP-reinforced PCTL segments of different configurations were designed, fabricated, and tested under punching loads, considering the effect of different parameters on the behavior of tunnels. Phase II involved investigating the shear behavior of nine precast RC segments reinforced with GFRP bars and ties with different variables such as reinforcement type and ratio, concrete strength, and configuration of transverse reinforcement. The specific objectives of this research can be summarized as follows:
1- To assess the structural performance of full-scale precast concrete tunnel lining segments reinforced with FRP bars under different loading conditions (punching and shear loads) compared to conventional members reinforced with steel reinforcement.

2- To optimize the FRP reinforcement details and concrete dimensions of concrete tunnel lining segments under different loading conditions.

3- To investigate the effect of different parameters of reinforcement ratio, transverse configurations, and concrete type on the structural performance of GFRP-reinforced PCTL segments.

4- To assess the contribution of closed ties on the shear strength of PCTL segments and develop recommendations for tunnel design provisions.

5- To evaluate the validity of the current analytical and design approaches for punching and shear to FRP reinforced precast concrete tunnel lining segments.

6- To develop design recommendations for North American standards for the use of composite reinforcing bars in precast concrete tunnel lining segments.

7- To propose an analytical model that can accurately predict the shear capacity of concave and convex GFRP-reinforced PCTL segments.

### 1.4 Research Methodology

To achieve the aforementioned objectives, comprehensive experimental and theoretical programs were conducted. The research program comprised two phases summarized as follows:

#### 1.4.1 Phase I: PCTL Segments Reinforced with GFRP Bars and Ties under Punching Loads

This phase was designed and prepared to provide experimental data on the punching behavior of GFRP-reinforced PCTL segments induced by soil conditions, such as rock expansion or the geotechnical conditions surrounding a tunnel. Eight full-scale rhomboidal PCTL specimens measuring 1500 mm in width and 250 mm in thickness were constructed and tested up to failure. The test parameters included the reinforcement type (GFRP versus steel), reinforcement ratio (0.46% or 0.86%), concrete strength (NSC and HSC), stirrups as shear reinforcement, and segment length (2100 mm or 3100 mm). The segments were instrumented with a series of strain gauges, potentiometers (LPOTs), and linear variable differential transformers (LVDTs) to monitor the overall movement of the specimens, and to gather dense measurements of local deformations.
needed for continuum analyses. The load–deflection response, failure mode, punching strength, cracking behavior, reinforcement and concrete strain, deformability, energy absorption, and effect of test variables were all evaluated in the experiments. A theoretical investigation involving a comparison between predictions of the FRP design provisions and experimental results was performed to assess the applicability of the current punching equations for PCTL segments.

1.4.2 Phase II: PCTL Segments Reinforced with GFRP Bars and Closed Ties under Shear Loads

This phase evaluated the behavior and shear strength of PCTL segments reinforced with GFRP bars with and without closed shear ties. Such loading in tunnels simulates fabrication and service loads acting on the intrados and extrados surfaces. A total of nine (five convex and four concave) full-scale rhomboidal PCTL specimens with a 1500 x 250 mm rectangular cross section and an arched length of 2100 mm were constructed and tested under three-point loading up to failure. The testing variables were the configuration of the transverse reinforcement (bars versus closed ties), the spacing of closed shear ties, the longitudinal reinforcement ratio, concrete strength, reinforcement type (GFRP and steel). The results were discussed in terms of deflection behavior, cracking patterns, failure mechanisms, shear capacities, and strain in the reinforcement and concrete. The effect of the test parameters was also discussed herein. Experimental results were employed to review and verify North American code provisions and existing models with some amendments to meet the requirements of designing tunnel segments reinforced with GFRP bars in terms of deflection at the service state and checking shear strength at the ultimate limit state. The contribution of curved geometry and tie resistance in the modified compression field theory (MCFT), plasticity theory (PT), and critical shear crack theory (CSCT) was also introduced in the theoretical approach to predict the shear capacity of GFRP-reinforced PCTLs.

1.5 Thesis Organization

The thesis consists of eight chapters. The contents of each chapter can be summarized as follows:

Chapter 1 presents the introduction, research significance, research objectives, and methodology of this research.

Chapter 2 introduces a review of the relevant literature, starting with an introduction to fiber-reinforced polymers (FRP), followed by an exploration of the main characteristics of FRP
reinforcement used as internal reinforcement. Subsequently, a brief overview of tunnel construction and the applied loads on tunnel segments in different loading stages is presented. Then, a summary of previous research on the GFRP-reinforced PCTL segments is presented. The punching and shear strengths of FRP-RC slabs and beams are reviewed. Finally, the code provisions related to the punching and shear behavior of FRP-RC members are also presented.

Chapter 3 describes the details of the experimental program undertaken in the present study. Detailed information is provided on the specimens’ details, material properties, specimens’ production, instrumentation, and test setup of punching and shear tests.

Chapter 4 (1st article) presents first experimental data on the punching behavior of five PCTL segments reinforced with GFRP bars, and one control segment reinforced with conventional steel bars for comparison purposes. The contributions of a unique configuration of shear stirrups and curved shape of PCTLs on the punching strength were discussed. The paper demonstrates the effect of segment arc length and reinforcement type/ratio on the general behavior of PCTL segments under punching loads. The experimental punching capacity was compared to the current FRP punching design equations in the design guidelines and codes to evaluate their applicability.

Chapter 5 (2nd article) determines the structural performance of four full-scale precast RC segments reinforced with GFRP bars and ties. The test parameters were the longitudinal reinforcement ratio and concrete type (NSC and HSC). The effect of these variables on the experimental results on the punching behavior were analyzed and evaluated. This paper shows more discussions based on failure mode, load-deflection response punching strength, cracking behavior, reinforcement and concrete strain, deformability, and energy absorption. punching-capacity predictions of GFRP-reinforced PCTL segments were performed according to the equations available for FRP-reinforced concrete slabs to assess their accuracy. Additionally, the equations proposed by researchers based on their experimental or analytical studies were evaluated.

Chapter 6 (3rd article) aims at investigating the experimental and analytical shear behavior of convex PCTL segments reinforced GFRP bars with and without closed ties. The testing parameters included the configuration of the transverse reinforcement (bars versus closed ties), the spacing of closed shear ties, the longitudinal reinforcement ratio, and concrete strength. The load–deflection behavior, shear crack width, failure mechanisms, shear capacities, and strain in the reinforcement
and concrete were discussed. This paper presents first experimental evidence on the contribution of transverse reinforcement on the shear and strength of PCTL segments. The critical shear crack theory (CSCT) was introduced and modified to predict the shear capacity of curved PCTL segment reinforced with GFRP bars and closed ties. A comparison between the test results and predictions made with the CSCT was performed in the theoretical approach to evaluate the applicability of existing FRP design code provisions.

Chapter 7 (4th article) reports on the experimental results of concave PCTL segments reinforced with GFRP bars and ties under shear loading. The type and amount of the longitudinal reinforcement and the concrete type were considered the test parameters. The test specimens comprised three GFRP-reinforced PCTL segments, and one specimen entirely reinforced with steel reinforcement as a reference specimen. Experimental results were employed to review and verify North American code provisions and existing models with some amendments to meet the requirements of designing tunnel segments reinforced with GFRP bars in terms of deflection at the service state and checking shear strength at the ultimate limit state. Measured deflection was used to conduct a comparison of the experimental values of the effective moment of inertia to predictions with current models to evaluate their capability.

Chapter 8 provides a summary, conclusions, and recommendations for future study.
CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

The application of glass fiber reinforced polymer (GFRP) reinforcement in concrete structures has encountering an increasing interest worldwide, for several applications in civil engineering. Therefore, the possibility of replacing traditional steel reinforcement with GFRP cages in precast concrete tunnel lining (PCTL) segments has been investigated by the authors in previous studies (Caratelli et al. 2016, 2017; Spagnuolo et al. 2017, 2018; Meda et al. 2019). Bending and edge thrust tests were conducted on full-scale GFRP-reinforced PCTL segments to assess their structural response. To specify the research parameters and the current needs, an elaborate review was presented herein for the important research related to the subject.

2.2 FRP Reinforcement

“FRP” is an acronym for Fiber Reinforced Polymers, which some also call Fiber Reinforced Plastics. FRP is a specific type of composite material used to describe a judicious combination of two or more materials to yield a product that is more efficient than its constituents. One constituent is called the reinforcing or Fiber phase (one that provides strength); the other in which the fibers are embedded is called the matrix phase. The matrix, such as a cured resin-like epoxy, polyester, vinyl ester, or other matrix acts as a binder and holds the fibers in the intended position, giving the composite material its structural integrity by providing shear transfer capability. Since the 1940s, the aerospace industry has pioneered the use of FRP materials. Today their potential is being harnessed for many uses. Advanced composite materials, so called because of their many desirable properties, such as high performance, high strength-to-weight and high stiffness-to-weight ratios, high energy absorption, and outstanding corrosion and fatigue damage resistance are now increasingly used for civil engineering infrastructure projects.
FRP reinforcing bars are manufactured from continuous fibers (such as carbon, glass, aramid, and basalt) embedded in matrices (thermosetting or thermoplastic). The mechanical properties of the final FRP reinforcement depend on different parameters. This includes fiber type, quality, volumetric ratio, adhesion resin and most importantly the manufacturing process. FRP materials are anisotropic and are characterized by high tensile strength with no yielding only in the direction of the reinforcing fibers (ACI 440.11-22). The tensile behavior of FRP bars consisting of one type of fiber material is characterized by a linearly elastic stress-strain relationship until failure. Compared to ductile steel, FRPs generally have higher tensile capacity and relatively lower modulus of elasticity. Table 2.1 summarizes the typical mechanical tensile properties of FRP bars compared to those of steel reinforcement.

<table>
<thead>
<tr>
<th>Fiber Type</th>
<th>Nominal yield strength, MPa</th>
<th>Tensile strength, MPa</th>
<th>Modulus of Elasticity, GPa</th>
<th>Yield strain, percent</th>
<th>Rupture strain, percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>276 to 517</td>
<td>483 to 690</td>
<td>200</td>
<td>0.14 to 0.25</td>
<td>6.0 to 12.0</td>
</tr>
<tr>
<td>AFRP</td>
<td>N.A.</td>
<td>1720 to 2540</td>
<td>41 to 125</td>
<td>N.A.</td>
<td>1.9 to 4.4</td>
</tr>
<tr>
<td>CFRP</td>
<td>N.A.</td>
<td>600 to 3690</td>
<td>120 to 580</td>
<td>N.A.</td>
<td>0.5 to 1.7</td>
</tr>
<tr>
<td>GFRP</td>
<td>N.A.</td>
<td>483 to 1600</td>
<td>35 to 51</td>
<td>N.A.</td>
<td>1.2 to 3.1</td>
</tr>
</tbody>
</table>

2.3 Tunneling

Tunneling has been a form of creating underground infrastructure for thousands of years. Throughout this period, there have been many advancements in technology that have improved the safety, efficiency, and overall productivity of tunneling systems. Many different applications of tunnels exist, serving different purposes. The most common form, transportation tunnels (i.e., roadway, subway, and pedestrian), offer a more convenient means of access from one location to another or in some cases to locations not previously accessible. Other applications of tunneling are pressure tunnels, which are a crucial component in the operation of hydroelectric power plants. Their purpose is to divert water from an upstream reservoir to a hydro-electric powerhouse, carrying very large outward pressures. Telecommunication cables are also routed through tunnels for their installation.
2.3.1 Tunnel Boring

Tunnel linings are commonly constructed in a circular shape using tunnel boring machines (TBMs). A TBM is a circular cross-section machine used to excavate tunnels through a variety of soils and rock strata. In unstable ground conditions, concrete segments are erected in place, forming a lining as the TBM advances. The tunnel lining contributes greatly to the serviceability, increased mechanical and ultimate capacity of the excavated area, and reduction of uncertainties during and after construction by increasing the factor of safety (Mashimo et al. 2002). The TBM has the advantage of limiting disturbances to the surrounding ground, which is crucial when tunneling in soft, unstable soils. It is also favorable when tunneling in residential areas, where drill and blast techniques may cause disturbances to the local population. Furthermore, by producing a smooth tunnel wall, the cost of lining the tunnel is greatly reduced. The main disadvantage of using a TBM as a method of tunneling is the large overhead costs due to the complexity of this machinery. However, as modern tunnels become longer, TBMs prove to be a more economical solution compared to D&B due to their efficiency and resulting shorter project time frame. The TBM has revolutionized the tunneling industry by making tunneling safer, a more economical solution for creating underground space, and opening the possibility of creating tunnels where it was not feasible before (Spencer et al. 2009). Figure 2.1 shows a typical TBM during boring tunnels.

![Tunnel boring machine (TBM).](image)

Figure 2.1 Tunnel boring machine (TBM).
2.3.2 Tunnel Linings

An important component of tunnel infrastructure is the tunnel lining system. The functionality of tunnels significantly depends on the structural and durability performance of their lining systems. Tunnel linings act as protective barriers against large overburden loads and complex geotechnical surrounding exposure conditions. Tunneling operations consist of the excavation of soil or rock, which results in increased in situ stresses due to the loose native soil. As such, additional reinforcement is employed in areas of low in-situ strength or where cover is inadequate. Reinforcing the tunnel walls is often done with rock bolts, wire mesh, and concrete lining, depending on the in-situ conditions present (i.e., soil or rock type, stress state, water table location). In the application of pressure tunnels, steel linings grouted with concrete may be used. Figure 2.2 illustrates the three basic types of linings: the lined section, the semi-lined section, and the unlined section.

![Tunnel lining reinforcement sections](image)

Figure 2.2 Tunnel lining reinforcement sections (Rancourt et al. 2007).

2.3.3 Precast Concrete Tunnel Lining (PCTL) Segments

The use of precast concrete tunnel lining (PCTL) systems in tunneling projects has been increasing as a result of their efficient and economical application in comparison with the conventional in-situ lining technique (Elliott 2002). PCTL segments allow speedy construction along with superior quality due to enhanced control during precast segment fabrication in precast plants. Moreover, the fabrication of PCTLs includes repetitive steps of batching and casting of concrete, which
ultimately results in a reduction in wastage compared to traditional in-situ concrete lining (Hariyanto et al. 2005). Multi-disciplinary skills are required for the design of PCTL segments in order to meet their structural and durability performance requirements. Thus, a detailed life-cycle analysis is required in order to calculate the total fabrication and installation cost of PCTL systems that satisfy specific design performance criteria (Hung et al. 2009).

2.3.4 Forces Acting on Precast Tunnel Segments

Tunnel provisions (ACI 544.7R-16; 533.5R-20; fib bulletin 83) divided the loading conditions of PCTLs into three main stages: production and transient stage loads, loads for the construction stage, and loads for service stages. During production and transient stages, the internal forces and stress from demolding, storage, transportation, and handling are used in designing PCTLs. Construction loads include the jacking thrust loads created by the tunnel-boring machine (TBM) on the circumferential ring joints and the pressures exerted against the exterior of the completed rings during grouting operations. The final service stages represent the long-term loads imposed on the lining by earth pressure, groundwater, and surcharges. Other loads should be considered in the service stage based on the ground conditions, the tunnel function, and any special circumstances.

2.3.4.1 Demolding

The design calculations of demolding stage shall include the design strength of concrete at the time that the process will be implemented. It should be noted that in many of the projects, the compressive strength of concrete is expected to be 15 MPa after 3.5-4 hours of concrete pouring (ACI 544.7R-16). Early-age concrete strength, both in tension and compression, is important. The bending moment results from the self-weight of the segment. The demolding phases should be analyzed in both SLS and ULS.

2.3.4.2 Storage

After demolding of segments, they will be stored in the storage yard. The segments are stored on top of each other with wood support between them and at the ground floor. In theory, if the wood supports are placed in straight line, the segment will not be subjected to any bending moment. However, in practice this may not be the case. Therefore, for design purposes 100mm eccentricity
is accepted between the wood supports (ACI 544.7R-16). This eccentricity may be inside or outside of the wood blocks.

2.3.4.3 Transportation

The transportation of the segments is shown in Fig 2.3. Capacity checks for transportation stage is similar to the ones in demolding and storage. The cracks should be avoided as much as possible. During the transportations, segments are subjected to different dynamic shock loadings. The dead load should be multiplied by the load factor as well as the dynamic factor (typically 2.0) to simulate the transportation loadings.

![Transportation of tunnel segments](image)

Figure 2.3 Transportation of tunnel segments (ACI 544.7R-16).

2.3.4.4 Handling

Segment handling from the storage area is usually done with special devices such as vacuum lifters. The loading case for handling is very similar to the demolding load case, and therefore, the design forces can be calculated using the same formulations. The dynamic factor is equal to 2 for handling loads.

2.3.4.5 Assembling the Segmental Ring

The introduction of mechanically excavated tunnel technology through a tunnel boring machine (TBM) is shown in Fig. 2.4. The segments are subjected to significant bursting, spalling, and compressive stresses during the advance of the TBM. Due to the thrust of the TBM, compressive stress occurs under the jacks. The high compressive forces result in spalling and bursting tensile stresses in the segment.
2.3.4.6 TBM Backup Load

TBM backup load is the self-eight of the backup equipment train behind the TBM Shield. The applied load is a concentrated variable load onto the segmental tunnel lining. Typical analysis results including deformations, axial force, bending moment, and shear forces. Results should confirm that PCTL segments can withstand these loads which induced by TBM backup. Punching strength should be also evaluated following the current provisions.

2.3.4.7 Earth pressure, groundwater, and surcharge

Precast concrete segments are used to withstand various loads from vertical and horizontal earth pressure, groundwater, self-weight, surcharge, and ground reaction loads. In accordance with load and resistance factor design (LRFD) principles, load factors and load combinations from AASHTO DCRT-1 are used to compute the ultimate limit state (ULS) and serviceability limit states (SLS).

2.3.4.8 Other loads

Other loads should also be considered that may result in failure of the liner to include earthquake, fire, explosion, breakouts at cross passageways, portals, shafts, and excessive longitudinal bending moments.
2.3.5 GFRP-Reinforced Precast Tunnel Segments

A few studies have been carried out to investigate the possibility of using GFRP bars in tunnel linings (Moccichino et al. 2010; Caratelli et al. 2016, 2017; Spagnuolo et al. 2017, 2018; Meda et al. 2019; Hosseini et al. 2022a, 2022b; Ibrahim et al. 2023). These researchers believed that GFRP bars could be a suitable solution for tunnel segments for several reasons. Firstly, GFRP rebars have a non-corrosive property, making them ideal for overcoming the durability problems caused by aggressive environments in wastewater tunnels or hydraulic tunnels. Moreover, the use of these non-metallic reinforcing bars in precast tunnel segments allows for a reduced concrete cover thickness, which is significantly important for preventing possible cracks during the construction, production, and transient stages of tunnel segments due to the increased bending capacity of the segments. Furthermore, the use of GFRP bars in tunnels reduces stray currents since they are non-conductive materials. This also provides cathodic protection, which is essential in ordinary tunnels. Finally, GFRP reinforcing bars are convenient for tunnel sections to be demolished due to their easy disposal. Cross-passage or emergency exit sections are typical examples of such tunnel sections that may require modification after construction. The results demonstrate the enhanced structural and durability performance of the GFRP-reinforced PCTL segments in terms of cracking behavior, deflection, and ultimate strength. Moreover, substituting GFRP reinforcement for conventional steel reinforcement satisfied the structural requirements.

2.3.6 Full-scale Tests of GFRP-Reinforced PCTL Segments

Tunnel provisions (ACI 544.7R-16; ACI 533.5R-20) address that the design and performance of segments should be evaluated using full-scale tests. Bending and point load tests are conducted at full scale GFRP-reinforced tunnel segments (Table 2.2). Both tests are conducted up to failure, and strength results are compared with the actions induced during loading stages. Bending tests are performed to verify the design and performance of segments during the production stages of stripping, storage, transportation, and handling, as well as for unsymmetrical earth pressure (Hilar et al. 2012) at the service stage. Full-scale point load tests simulate the tunnel-boring machine (TBM) thrust jack forces on the segment during the excavation process (Caratelli et al. 2017), as well as the force transfer through a reduced cross section in longitudinal joints. Figure 2.7 shows bending and thrust loading tests.
Table 2. Full-scale tests on GFRP-reinforced PCTL segments.

<table>
<thead>
<tr>
<th>References</th>
<th>Approach</th>
<th>Type of test</th>
<th>Material</th>
<th>Objective</th>
</tr>
</thead>
<tbody>
<tr>
<td>Caratelli et al. (2016)</td>
<td>Experimental</td>
<td>Thrust load</td>
<td>GFRP-RC</td>
<td>Structural behavior of segment</td>
</tr>
<tr>
<td>Caratelli et al. (2016)</td>
<td>Experimental</td>
<td>Bending load</td>
<td>RC GFRP-RC</td>
<td>Ultimate bearing capacity of segments</td>
</tr>
<tr>
<td>Caratelli et al. (2017)</td>
<td>Experimental</td>
<td>Thrust load</td>
<td>RC GFRP-RC</td>
<td>Structural behavior of segments that consist of different types of reinforcement</td>
</tr>
<tr>
<td>Spangnuolo et al. (2016)</td>
<td>Experimental Analytical</td>
<td>Bending load</td>
<td>RC GFRP-RC</td>
<td>Ultimate bearing capacity of segments</td>
</tr>
<tr>
<td>Tengilimoğlu et al. (2019)</td>
<td>Experimental</td>
<td>Thrust load</td>
<td>RC GFRP-PFRC</td>
<td>Structural behavior of segments that consist of different types of reinforcement</td>
</tr>
<tr>
<td>Hosseini et al. (2022)</td>
<td>Experimental Analytical</td>
<td>Bending load</td>
<td>GFRP-RC GFRP-PFRC</td>
<td>Structural behavior of segments that consist of reinforcement’s type and ratio</td>
</tr>
</tbody>
</table>

Figure 2. 5 (a) bending and (b) Thrust load tests (Caratelli et al. 2017).

2.4 Punching Capacity

The punching phenomenon emerges in concrete flat structures exposed to high bending moment and concentrated shear stress loads, either supported on a column or subjected to a point load.
2.4.1 Punching Failure Mechanism

2.4.1.1 In the Case of Planner without Transverse Reinforcement

The punching failure is characterized by low stresses in the longitudinal reinforcement and the development of a diagonal crack with variable inclination, starting from the root of the column to the tension face of the slab (Fig. 2.6). The inclination of the failure surface is dependent on the geometry of the member (depth, slenderness, column dimension to slab thickness) and the characteristics of the structural parameters (material strengths, aggregate distribution, dimension, reinforcement layout, etc.). After the diagonal tension cracking occurs in the vicinity of the critical section of the slab around the perimeter of the load area, the slab carries the shear forces by shear across the compression zone, aggregate interlock, and dowel action.

![Diagram of punching failure mechanism](image)

Figure 2.6 Typical cracks at interior slab-column connections (Eladawy et al. 2019).

2.4.1.2 In the Case of Planner with Transverse Reinforcement

The effective solution to enhance the ductility and punching shear strength of slab-column connections is to use shear reinforcement in the slab in the vicinity of columns (Eladawy et al. 2019). The principal effect of shear reinforcement is to restrain the discontinuity of the slab at the shear crack, concentrating rotation at the vertical crack at the face of the column. Once inclined shear cracks develop, the shear reinforcement transfers most of the forces across the shear cracks.
and delays further widening. This, in turn, increases the punching shear and deformation capacity of the slab (Rizk et al. 2011). Figure 2.7 shows several potential failure modes of slabs with shear reinforcement.

![Figure 2.7 Failure modes: (a) crushing struts, (b) punching inside shear-reinforced zone, (c) punching outside, (d) concrete core delamination, and (e) flexural bars yielding.](image)

### 2.4.2 Research on Punching Behavior of RC Members

*Matthys and Taerwe (2000)* investigated the punching shear behavior of two-way concrete slabs reinforced with an FRP grid. A total of seventeen punching tests were performed on square slabs with a side length of 1000 mm and a total slab thickness of 120 or 150 mm. The investigated parameters were the flexural reinforcement ratio, slab thickness, and loaded area. The test results revealed a strong interaction between shear and flexural effects, with most slabs exhibiting a punching cone failure. For FRP-RC specimens with similar flexural strength as the steel-reinforced reference specimens, the obtained punching load and stiffness in the cracked state were considerably lower. However, for the FRP-RC specimens with an increased reinforcement ratio or an increased slab depth, the behavior of the slabs was comparable to steel-reinforced reference slabs.

*Ospina et al. (2003)* examined the punching shear behavior of four isolated full-scale slab-column systems reinforced with GFRP bars and subjected to concentric gravity loading. The main variables were the slab reinforcement material, the type of reinforcing mat, and the slab reinforcement ratio. The results showed that the punching failure in FRP-reinforced specimens is
affected by the elastic stiffness of the FRP mat as well as its bond characteristics. They suggested that concrete crushing did not necessarily trigger punching shear failure in steel or FRP-reinforced concrete slabs. They also proposed a punching equation for FRP-RC slabs as follows:

\[
V_c = 2.77(\rho_f f_c')^{0.33} (E_f / E_s)^{0.5} b_{0.15d} d
\]  

(2.1)

Nguyen–Minh and Rovňák (2013) studied the punching shear behavior of concrete two-way slabs reinforced with GFRP bars. A total of six full-scale slab-column connections (2200×2200×150 mm), consisting of three GFRP-reinforced slabs and three control steel-reinforced slabs, were tested. They concluded that increasing the GFRP reinforcement ratio in tested slabs subjected to punching shear loads has proven to have the following benefits: (i) an increase in punching shear resistance (up to 36%); (ii) reduction of deflections (up to 35%). In comparison with steel-reinforced slabs, the GFRP-reinforced ones have a smaller resistance (up to 38%); higher displacement (up to 2.6 times); and larger crack width (up to 34%). The size factor and the effect of the span to effective depth ratio were taken into account in the calculations of the punching resistance of the FRP-reinforced slab-column connections, and a new equation was proposed as Eq. 2.2:

\[
V_c = (400 / d)^{0.5} \left( (0.8 / (L_i / d)) - (c_i / d) \right) (0.01 \rho_f)^{0.33} E_f^{0.33} f_{c'} b_{cr,i} d
\]  

(2.2)

Hassan et al. (2013) investigated the punching–shear behavior of two-way concrete slabs reinforced with glass fiber reinforced polymer (GFRP) bars of different grades. A total of 10 full-scale interior slab-column specimens measuring 2500×2500 mm with thicknesses of either 200 or 350 mm and 300 × 300 mm square column stubs were fabricated with normal and high-strength concretes. The specimens were tested under monotonic concentric loading until failure. The main variables were the slab reinforcement material, the GFRP bars grades, the slab reinforcement ratio, and concrete compressive strengths. The experimental results showed that all specimens exhibited typical punching failure (Fig 2.8). Increasing the GFRP reinforcement ratio yielded higher punching–shear capacities, lower strains in the reinforcement, and smaller slab deflections. Moreover, using high compressive strength concrete (HSC) for the GFRP–RC specimens improved the punching capacity and enhanced the load–deflection relationships.
Abbas et al. (2014) studied the punching behavior of precast concrete tunnel linings systems (PCTL). A total of two skewed full-scale RC and SFRC segments, the length, width and thickness of segments were 2120 mm, 1500 mm and 235 mm, respectively. The conventional rebar cage in RC segments was completely replaced with steel fibers in the SFRC segments. The test results showed that the ultimate load carrying capacity of RC PCTL segments was higher than that of similar SFRC segments. However, the SFRC segments showed enhanced cracking behavior and exhibited a steady drop in load carrying capacity due to the crack bridging action of steel fibers. The RC segments showed more abrupt decrease in load carrying capacity after reaching their peak load and failed in a more brittle manner (Fig 2.9). It was observed that the crack widths in SFRC segments were smaller than that of the RC segments. Moreover, both the RC and SFRC segments satisfied the design criteria for punching loads.

Figure 2. 8 Typical punching failure (Hassan et al. 2013).

Figure 2. 9 Brittle failure of RC-PCTL segment (Abbas et al. 2014).
2.4.3 Codified Punching Equations of FRP-RC Members

2.4.3.1 ACI 440.11 (2022)

The punching capacity of FRP-reinforced concrete slabs was provided based on the mechanistic beam shear model, which accounts for reinforcement stiffness with other modifications to involve the effect of shear transfer, as shown in Eq. (2.3)

\[ V_c = \frac{4}{5} \sqrt{f_c' b_{o,0.5d} (kd)} \]  
\[ k = \sqrt{2\rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f \]

where \( b_o \) is the perimeter of critical section computed at \( d/2 \) away from plate face with the same shape (mm); \( f_c' \) is the specified compressive strength of the concrete (MPa); \( k \) is the ratio of the neutral-axis depth to reinforcement depth; \( d \) is the effective depth of the bottom flexural bars; \( \rho_f \) is the fiber flexural reinforcement ratio; and \( n_f \) is the ratio between the modulus of elasticity of the FRP bars and concrete.

2.4.3.2 CAN/CSA S806 (2012)

The factored shear–stress resistance of concrete due to punching is the smallest of Eqns. (2.4) to (2.6)

\[ V_r = V_c = \left(1 + \frac{2}{\beta_c}\right) \left[0.028\phi_c (E_f \rho_f f_c')^{\frac{1}{3}}\right] b_{o,0.5d} d \]  
\[ V_r = V_c = \left[\left(\frac{\alpha_s d}{b_o}\right) + 0.19\right] 0.147\phi_c (E_f \rho_f f_c')^{\frac{1}{3}} b_{o,0.5d} d \]  
\[ V_r = V_c = 0.056\phi_c (E_f \rho_f f_c')^{\frac{1}{3}} b_{o,0.5d} d \]

where the value of \( f_c' \) used in these equations shall not exceed 60 MPa; \( \beta_c \) is the rectangularity of the plate; and \( \alpha_s \) is the coefficient equal to 4 for interior columns.
2.4.3.3  JSCE (1997)

When the eccentricity of the load is negligible, the punching-shear capacity of planner members is determined by Eq. (2.7)

\[ V_c = \beta_d \beta_p \beta_r f_{pcd} b_{o,0.5d} d / \gamma_b \]  
\( (2.7a) \)

\[ \beta_d = (1000 / d)^{1/4} \leq 1.5 \]  
\( (2.7b) \)

\[ \beta_p = (100 \rho E_f / E_s)^{1/3} \leq 1.5 \]  
\( (2.7c) \)

\[ \beta_r = 1 + 1 / (1 + 0.25u / d) \]  
\( (2.7d) \)

\[ f_{pcd} = 0.2 \sqrt{f'_c} \leq 1.2 \text{ Mpa} \]  
\( (2.7e) \)

where \( u \) is the peripheral length of loaded area (mm) and \( \gamma_b \) is the member factor equal to 1.3.

2.4.3.4  AASHTO (2018)

For sections without shear reinforcement, the nominal punching resistance of concrete \( V_c \) in kips shall be taken as Eq. (2.8)

\[ V_c = 0.316 k \sqrt{f'_c b_{o,0.5d} d} \]  
\( (2.8) \)

where \( b_{o,0.5d} \) and \( d \) are in inches and \( f'_c \) are in ksi.

2.5  Shear Behavior of FRP-RC Members with and without Stirrups

2.5.1  Shear Transfer Mechanism

In literature, reinforced concrete members without shear reinforcement resist shear through various mechanisms, including shear resistance of the uncracked concrete compression zone, aggregate interlock, residual tensile stress across the cracks, dowel action of longitudinal reinforcement, and arch action (Ali et al. 2017). The combined contribution from these mechanisms is referred to as the concrete contribution to the shear strength, denoted as \( V_{cf} \). Compared to steel-reinforced
members, the shear strength of RC members reinforced with FRP bars as flexural reinforcement is lower due to their wider and deeper cracks, which in turn, reduces the contribution of un-cracked concrete. In addition, lower modulus of elasticity of FRP bars and their weaker contribution in dowelling action leads to a lower shear capacity of GFRP-RC members (ACI 440.11-22). The contribution of FRP stirrups to shear resistance ($V_f$) is based on the stresses created in the stirrups, as FRP is linearly elastic until failure, does not yield, and has an ultimate tensile strength of the bent segment that is considerably less than that of a straight part (Kim et al. 2015). Accordingly, FRP design guidelines restrict the strain permitted in FRP stirrups to lower levels in the ultimate state.

2.5.2 Research on Shear Behavior of RC Members

Nagasaka et al. (1993) tested FRP-RC beams with cross-sections of 250 x 300 mm considering the type of FRP reinforcement (carbon, aramid, glass, or hybrid) and the reinforcement ratio of the stirrup (0%, 0.5%, 1.0%, and 1.5%). The results indicated that the ultimate shear capacity increased with increasing reinforcement ratio of the FRP stirrup, while it was not significantly affected by the type of stirrup. Further, the results demonstrated that the ultimate shear capacity of beams can be effectively estimated using the tensile strength of the bent part of the stirrup.

Shehata et al. (2000) tested a total of ten large-scale RC beams with a depth of 560 mm considering the material used for the stirrups (steel, carbon fiber reinforced polymer (CFRP), and glass fiber reinforced polymer (GFRP) and the stirrup spacing (d/2, d/3, and d/4). The results indicated that the shear deformation is affected by the bond characteristics and elastic modulus of the stirrup material. They also suggested a stirrup strain limit of 0.002 to control the crack width in both GFRP and CFRP.

Ali et al. (2016) investigated the behavior and shear strength of full-scale FRP-reinforced circular members without web reinforcement. The specimens, which measured 3,000 mm in length by 500 mm in diameter, were tested under four-point bending. The test parameters included the longitudinal reinforcement ratio and the modulus of elasticity of the reinforcing bars. The results indicated that the concrete contribution to the shear strength is proportional to the axial stiffness of the longitudinal reinforcing bars. The higher reinforcement ratio or the modulus of elasticity of the reinforcing bars, the higher the obtained shear strength.
2.5.3 Shear Design Provisions for FRP-RC Members

2.5.3.1 ACI 440.11 (2022)

The concrete shear capacity $V_{cf}$ of flexural members using FRP as main reinforcement can be determined as follows:

$$V_{cf} = \frac{2}{5} \lambda_s \sqrt{f'c} b_w (kd)$$

(2.9a)

$$\lambda_s = \sqrt{\frac{2}{1+\frac{d}{10}}} \leq 1$$

(2.9b)

where $b_w$ is width of member and $\lambda_s$ is size effect modification factor equal 1 if $A_{f_v} > A_{f_v,min}$.

The shear resistance $V_{sf}$ provided by FRP stirrups perpendicular to the axis of the member can be evaluated as follows:

$$V_{sf} = A_{f_v} f_{f_v} d / S$$

(2.10)

where $s$ is spacing of stirrups; $A_{f_v}$ is area of shear reinforcement within a distance $s$; $f_{f_v}$ is the design tensile strength and shall not exceed the smaller of $f_{f_v}$ and 0.005 $E_{f_v}$.

2.5.3.2 CAN/CSA S806 (2012)

Sections having an effective depth not exceeding 300 mm and with no axial load acting on them, the concrete contribution is calculated as follows:

$$V_{cf} = 0.05 \lambda \phi k_m k_r \sqrt{f'c} b_w d_v$$

(2.11a)

$$k_m = \left(\frac{V_{f_v} d}{M_{f_v}}\right)^{1/2} \leq 1$$

(2.11b)

$$k_r = 1 + \left(\frac{E_{f_v}}{\rho_{f_v}}\right)^{1/3}$$

(2.11c)

where $\lambda$ is the concrete density factor and is 1.0 for normal density concrete; $\phi$ is the material resistance factor; $k_m$ and $k_r$ are factors accounting for the effects of moment to shear ratio and longitudinal reinforcement rigidity, respectively, on the shear strength of the section under consideration.

For members with FRP transverse reinforcement perpendicular to the member axis shall be calculated as follows:

$$V_{sf} = \phi f_{f_v} A_{f_v} f_{f_v} d_v \cot \theta / S$$

(2.12a)

$$f_{f_v} = 0.005 E_{f_v} \leq 0.4 f_{w}$$

(2.12b)
where \(d_v\) is the effective shear depth and calculate the greater of 0.72h or 0.9d.

### 2.5.3.3 CAN/CSA S6-19 (2019)

The shear strength of concrete members is based on the modified compression field theory (MCFT). For FRP reinforced concrete members, the following modifications are introduced in the general method to reflect using FRP reinforcement instead of steel reinforcement:

\[
V_{cf} = 2.5 \beta f_y f_y d_v b_w \frac{\cot \theta}{S} \quad (2.13a)
\]

\[
\beta = 0.4 \left(1 + 1500 \varepsilon_v\right) \frac{1300}{(1000 + s_{ze})} \quad (2.13b)
\]

For members with FRP transverse reinforcement perpendicular to the member axis shall be calculated as follows:

\[
V_{sf} = \phi f_y A_p f_y d_v \frac{\cot \theta}{S} \quad (2.14a)
\]

\[
f_y = 0.004 E_y \leq f_{u, bent} \quad (2.14b)
\]

\[
f_{u, bent} = (0.05 r_y / d_b + 0.3) f_{fu} / 1.5 \quad (2.14c)
\]

\[
\theta = (29 + 7000 \varepsilon_f)(0.88 + s_{ze} / 2500) \quad (2.14d)
\]
CHAPTER 3
EXPERIMENTAL PROGRAM

3.1 Synopsis

A survey on the body of the literature performed in Chapter 2 revealed that the punching and shear behavior of PCTL segments reinforced with GFRP bars and ties have not been investigated. Therefore, a comprehensive research program was carried out in the Department of Civil Engineering at the University of to investigate the structural performance of precast concrete tunnel lining segments reinforced with curvilinear GFRP bars. Punching and shear tests—in addition to static flexural loading (Hosseini et al. 2022a, 2022b); quasi-static cyclic flexural loading (Ibrahim et al. 2023); and thrust loading—were conducted on full-scale GFRP-reinforced PCTL segments. This experimental program presents the effect of different variables and design parameters on the structural behavior of GFRP-reinforced PCTL segments under punching and shear loads. This chapter presents the outline of the experimental research program carried out in this study. Detailed information is provided on the specimens’ details, material properties, specimens’ fabrication processes, test setup, instrumentations, and loading procedure.

3.2 Material Properties

Three materials were used in fabricating the test specimens, these materials are concrete, GFRP, and steel reinforcement (for control specimens).

3.2.1 GFRP Reinforcement

The sand-coated GFRP reinforcement used in this study was manufactured by pultruding 78% (by weight) boron-free glass fibers impregnated with thermoset vinyl-ester resin, additives, and fillers. Newly developed curvilinear (Pultrall 2019) No.5 (15 mm) GFRP bars were employed to reinforce the GFRP-reinforced PCTL segments in the top and bottom longitudinal direction. The radius of the bottom and top mats was of 3305 mm and 3405 mm, respectively. The tensile strength, modulus of elasticity, and ultimate tensile strain of the flexural curvilinear bars were calculated according to ASTM D7205-21 (Table 3.1). Transverse reinforcement was provided as No. 4 (13
mm) GFRP straight bars and closed ties. The bent tensile strength of the closed ties was determined according to AASHTO (2018) and CSA S6-19. Number 4 (13 mm) and No. 5 (15 mm) U-shaped GFRP bars served as shear reinforcement and were used to ensure proper anchorage for the longitudinal reinforcement, respectively. All GFRP bars and ties had sand-coated surfaces to improve the bonding between the bars and the surrounding concrete. Figure 3.1 shows an overview of the GFRP bars and ties.

![Figure 3.1 GFRP reinforcement: (a) curvilinear bars, (b) U-shaped closing bars, and (c) transverse reinforcement (closed ties or bars).](image)

### 3.2.2 Steel Reinforcement

Deformed 15M (16 mm) curvilinear steel bars were used as longitudinal reinforcement and No. 10M (11.3 mm) steel ties were provided as transverse reinforcement. As shown in Table 3.1, the mechanical properties of utilized steel bars were determined in accordance with ASTM A615/A615M-20.

<table>
<thead>
<tr>
<th>Reinforcement Type</th>
<th>Reinforcement Shape</th>
<th>Bar Size</th>
<th>$d_b$ (mm)</th>
<th>$A_f$ (mm$^2$)</th>
<th>$E_f$ (GPa)</th>
<th>$f_{fu}$ (MPa)</th>
<th>$\varepsilon_{fu}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GFRP</td>
<td>Curvilinear bars #5</td>
<td>15.0</td>
<td>199</td>
<td>55.1 ± 1.25</td>
<td>1115 ± 60</td>
<td>2.0 ± 0.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Closed stirrups #4</td>
<td>13.0</td>
<td>129</td>
<td>55.6 ± 1.6</td>
<td>1248 ± 74</td>
<td>2.2 ± 0.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>U-shaped closing bars #5</td>
<td>15.0</td>
<td>199</td>
<td>53.5 ± 1.1</td>
<td>1283 ± 42</td>
<td>2.4 ± 0.1</td>
<td></td>
</tr>
<tr>
<td>Steel</td>
<td>Curvilinear bars 15 M</td>
<td>16.0</td>
<td>200</td>
<td>200.0</td>
<td>$f_y = 480 \pm 15$</td>
<td>$\varepsilon_y = 0.24$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Closed stirrups 10 M</td>
<td>11.3</td>
<td>100</td>
<td>200.0</td>
<td>$f_y = 480 \pm 10$</td>
<td>$\varepsilon_y = 0.24$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>U-shaped closing bars 15 M</td>
<td>16.0</td>
<td>200</td>
<td>200.0</td>
<td>$f_y = 480 \pm 15$</td>
<td>$\varepsilon_y = 0.24$</td>
<td></td>
</tr>
</tbody>
</table>
3.2.3 Concrete Properties

Two types of concrete were used in this study classified as normal-concrete (NSC) and high-strength concrete (HSC). The two mixes had a target 28-day compressive strength of 40 MPa for the NSC and 70 MPa for the HSC. The concrete compressive strength for all specimens was determined based on the average test results of six concrete cylinders measuring 100 x 200 mm tested on the same day of testing. Table 3.2 gives the actual concrete compressive strength for all test specimens.

3.3 Test Matrix, Parameters, and Specimens Details

3.3.1 Phase I: Punching Test

A total of eight full-scale PCTL segments were constructed and tested under punching loads. The test matrix was arranged to investigate the influence of the reinforcement type (GFRP or steel), reinforcement ratio, shear reinforcement, specimen length, and concrete strength (Fig. 3.2). Table 3.2 presents the test matrix and reinforcement details of the tested specimens. Segments were labeled with a letter denoting the reinforcement type (G for GFRP and S for steel bars and ties). The subscripts (x%) stand for the longitudinal reinforcement ratio of the top and bottom meshes, followed by the specimen arc length in meters. SR denotes shear reinforcement while the letter H indicates high-strength concrete. The experimental program comprised seven PCTL segments reinforced with GFRP bars and one reinforced with steel for comparison. Two different amounts of longitudinal reinforcement were used to reinforce the GFRP specimens (7 and 13 No. 5 bars, with reinforcement ratios of 0.46% and 0.86%, respectively) for the top and bottom meshes. The segments were designed to have a flexural reinforcement ratio greater than the balanced reinforcement ratio (ACI 440.11-22). For each reinforcement ratio of short specimens, one specimen was fabricated using a concrete strength of 40 MPa (NSC), while the other was cast with a concrete strength of 70 MPa (HSC). All GFRP segments were reinforced with No. 4 ties in the transversal direction at a spacing of 200 mm. The control specimen was reinforced longitudinally with 7 No. 15M steel bars with a reinforcement ratio of 0.47% and transversely with 10M steel ties at 200 mm. One of the GFRP segments had five discrete branches of No. 4 GFRP U-shaped stirrups as shear reinforcement to assess the stirrups’ contribution to punching and deformation capacity. The shear reinforcement stirrups for typical slabs/plates reinforced with straight bars are
Chapter 3: Experimental Program

arranged in a cruciform pattern (ACI 318-19, CSA A23.3-19). Nevertheless, the precast tunnel segments were fabricated with difficult geometry and complex configurations. A new technique was designed to arrange the shear reinforcement stirrups by adding the discrete stirrups symmetrically to the closed ties spaced at $0.25d$ and $2.25d$ (where $d$ is the average effective depth of the segment) from the loading-plate face. The rhomboidal PCTL segment considered herein is one of seven different shapes of segments to comprising full parallel rings with an internal diameter of 6500 mm. Two average arc lengths of the tested segments were 2100 mm and 3100 mm, while the width and thickness were 1500 mm and 250 mm, respectively. The clear concrete cover was kept constant at 40 mm for all segments (ACI 533.5R-20).

![Assembled GFRP cages](image)

Figure 3. 2 Assembled GFRP cages.
3.3.2 Phase II: shear Test

The five convex full-scale GFRP-reinforced PCTL segments were designed, constructed, and tested under an increasing monotonically shear load. The test matrix was arranged to assess their contribution to the shear strength of tunnel segments (Table 3.2). Each specimen was labeled with the letter G, indicating GFRP reinforcement, both longitudinally and transversely. The subscript in parenthesis refers to the flexural reinforcement ratio. T(x) or C(x) denote the spacing of the transverse bars or closed stirrups, respectively, whereas the letter H stands for high-strength concrete. The effect of shear stirrups was investigated with No. 4 closed GFRP ties and transverse bars at a spacing of 200 mm. In addition, their ratio was investigated with No. 4 closed GFRP ties spaced at 100 mm and 200 mm. Specimen G(0.86)/C100 had a tie spacing less than the maximum spacing limit specified in CAN/CSA S806-12 (CSA 2012), which represents the minimum of 0.6dV cot θ and 400 mm. The closed-tie spacing in G(0.46)/C200, G(0.86)/C200, and G(0.86)/C200-H was as per that recommended in tunnel standards (ACI 2020 and ITA 2019). Furthermore, a control specimen (G(0.46)/T200) with transverse bars was included in the test variables for comparison. Two longitudinal reinforcement ratios of 0.46% and 0.86% (7 and 13 No. 5 GFRP bars, respectively) were used for the top and bottom meshes. Segments with a high longitudinal reinforcement ratio were fabricated with NSC and HSC.

Four concave real-scale PCTL segments with closed ties were fabricated, cast, and tested under a monotonically increasing line load. The experimental program comprised three PCTL segments reinforced with GFRP bars and ties, while the fourth segment was reinforced with steel for comparison. Both the top and bottom meshes of two of the GFRP-reinforced PCLTs had a flexural reinforcement ratio of 0.46% (7 No. 5 at 250 mm spacing). Another segment had a reinforcement ratio of 0.86% (13 No. 5 at 125 mm spacing). Two of the three segments with the low reinforcement ratio were fabricated with normal-strength concrete; one was made with high-strength concrete. The GFRP-reinforced specimens were had No. 4 closed ties at a pitch of 200 mm. The control segment was reinforced longitudinally with 7 M15 steel bars (0.47%) and transversely with M10 steel ties at a pitch of 200 mm. Each segment was identified with a label consisting of a letter and a number (Table 3.2). The letter indicates the type of reinforcement (G for GFRP and S for steel longitudinal bars and stirrups). The number denotes the percentage of the
flexural reinforcement ratio. The high-strength concrete segment bears the letter H. Figure 3.3 shows the geometry and reinforcement details of the testing specimens.

Figure 3.3 Typical dimensions and reinforcement details of all tested segments (dimensions in mm).
### Table 3.2 Details of tested specimens.

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Specimen ID¹</th>
<th>Conc. Type</th>
<th>Reinf. Type</th>
<th>Arc Length (mm)</th>
<th>Longitudinal Reinforcement</th>
<th>ρf (%)</th>
<th>ρb (%)</th>
<th>Transverse Reinforcement</th>
<th>Configuration</th>
<th>fc' (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Punching</td>
<td>G(0.46)-2.1</td>
<td>NSC</td>
<td>GFRP</td>
<td>7 No. 5</td>
<td>0.46 0.33 No. 4 @200 mm</td>
<td>Closed ties</td>
<td>47.8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>G(0.86)-2.1</td>
<td>NSC</td>
<td>GFRP</td>
<td>13 No. 5</td>
<td>0.66 0.31 No. 4 @200 mm</td>
<td>Closed ties</td>
<td>41.6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>G(0.46)-2.1-SR²</td>
<td>NSC</td>
<td>GFRP</td>
<td>7 No. 5</td>
<td>0.46 0.32 No. 4 @200 mm</td>
<td>Closed ties</td>
<td>47.1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>G(0.46)-3.1</td>
<td>NSC</td>
<td>GFRP</td>
<td>7 No. 5</td>
<td>0.46 0.33 No. 4 @200 mm</td>
<td>Closed ties</td>
<td>51.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>G(0.86)-3.1</td>
<td>NSC</td>
<td>GFRP</td>
<td>13 No. 5</td>
<td>0.48 0.34 No. 4 @200 mm</td>
<td>Closed ties</td>
<td>48.8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>G(0.46)-2.1-H</td>
<td>HSC</td>
<td>GFRP</td>
<td>7 No. 5</td>
<td>0.46 0.45 No. 4 @200 mm</td>
<td>Closed ties</td>
<td>70.7</td>
<td></td>
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</tr>
<tr>
<td></td>
<td>G(0.86)-2.1-H</td>
<td>HSC</td>
<td>GFRP</td>
<td>13 No. 5</td>
<td>0.86 0.42 No. 4 @200 mm</td>
<td>Closed ties</td>
<td>66.9</td>
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<td></td>
</tr>
<tr>
<td>S(0.47)-2.1</td>
<td>NSC</td>
<td>Steel</td>
<td>7-15M</td>
<td>0.47 3.02 10M @200 mm</td>
<td>Closed ties</td>
<td>40.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear (Convex)</td>
<td>G(0.46)/T200</td>
<td>NSC</td>
<td>GFRP</td>
<td>7 No. 5</td>
<td>0.46 0.35 No. 4 @200 mm</td>
<td>Bars</td>
<td>51.5</td>
<td></td>
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<td></td>
</tr>
<tr>
<td></td>
<td>G(0.46)/C200</td>
<td>NSC</td>
<td>GFRP</td>
<td>7 No. 5</td>
<td>0.46 0.34 No. 4 @200 mm</td>
<td>Closed ties</td>
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</tr>
<tr>
<td></td>
<td>G(0.86)/C100</td>
<td>NSC</td>
<td>GFRP</td>
<td>13 No. 5</td>
<td>0.86 0.34 No. 4 @100 mm</td>
<td>Closed ties</td>
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<td></td>
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<td>NSC</td>
<td>GFRP</td>
<td>13 No. 5</td>
<td>0.86 0.34 No. 4 @200 mm</td>
<td>Closed ties</td>
<td>50.2</td>
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<tr>
<td></td>
<td>G(0.86)/C200-H⁵</td>
<td>HSC</td>
<td>GFRP</td>
<td>13 No. 5</td>
<td>0.86 0.45 No. 4 @200 mm</td>
<td>Closed ties</td>
<td>70.1</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Shear (Concave)</td>
<td>G-0.46</td>
<td>NSC</td>
<td>GFRP</td>
<td>7 No. 5</td>
<td>0.46 0.33 No. 4 @200 mm</td>
<td>Closed ties</td>
<td>51.4</td>
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<td></td>
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<td>GFRP</td>
<td>13 No. 5</td>
<td>0.86 0.34 No. 4 @200 mm</td>
<td>Closed ties</td>
<td>48.8</td>
<td></td>
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<td></td>
</tr>
<tr>
<td></td>
<td>G-0.46-H</td>
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<td>GFRP</td>
<td>7 No. 5</td>
<td>0.46 0.43 No. 4 @200 mm</td>
<td>Closed ties</td>
<td>70.3</td>
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<tr>
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<td>NSC</td>
<td>GFRP</td>
<td>7-15M</td>
<td>0.47 3.32 No. 4 @200 mm</td>
<td>Closed ties</td>
<td>48.3</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1 G denotes GFRP bars and stirrups and S denotes steel, with a subscript (x%) indicating longitudinal reinforcement ratio, followed by specimen arc length.

2 SR denotes shear reinforcement represented by 5 additional legs for the middle fourth ties at the punching region.

3 ρb calculated according to ACI 440-11-22 and ACI 318-19 for GFRP and steel-reinforced concrete slabs, respectively.

4 fc' based on six 100 × 200 mm cylinder tests.

5 Tx or Cx refers to the spacing of transverse bars or closed stirrups, and the suffix H stands for high-strength concrete.

### 3.4 Fabrication of Test Specimens

GFRP and steel cages were assembled for the various configurations at the University of Sherbrooke. The specimens were shipped to a precast plant (Sym-Tech precast company) in Saint-Hyacinthe (QC, Canada) for instrumentation and casting. Curved wooden formwork was designed and constructed to fabricate the PCTL segments. Concrete was cast into the specimen mold using an overhead crane bucket. The segments were compacted with an electrical vibrator to ensure adequate consistency and then leveled manually. A plastic sheet covered the entire mold to cure the segments and eliminate steam and moisture losses. The concrete strength reached sufficient strength after 24 hours. The specimens were stripped and then placed in a storage yard. Figure 3.4 illustrates the fabrication process of the PCTL segments.
3.5 Instrumentations

In punching test, each PCTL specimen was equipped with two instrumented bars (curvilinear and closed tie) in the orthogonal directions (Fig. 3.5(a)). The instrumentation consisted of five electrical strain gauges attached to the middle bars in the bottom mat (tension side) and one additional strain gauge for the longitudinal direction of the long span. To obtain the longitudinal
and transverse strain profiles, the gauges were located at 0, 100, 200, 300, and 400 mm from the loading plate face. Two strain gauges in each orthogonal direction were glued to the top of the reinforcement (compression side) at the plate face. Strain in the shear stirrups was measured through two gauges mounted at mid-height of the vertical legs. In addition, five concrete strain gauges were mounted on the concrete surface. The deflection at mid- and quarter-span was measured with five linear potentiometers (LPOTs), both longitudinal and transverse.

In shear test, strain in the flexural bars and closed ties was measured with an electric strain gauge with a gauge length of 10 mm. The two central curvilinear bars in each convex PCTL segment were instrumented with a single resistance gauge at mid-span (Fig. 3.5(b)). Four strain gauges were placed at the mid-height of the stirrups’ vertical legs. Two concrete strain gauges with a gauge length of 60 mm were mounted directly under loading on the extrados and lateral surfaces to measure compressive strains. In addition, four strain gauges were glued to the lateral side to measure concrete diagonal strains along the shear span. For concave segments, nine gauges were mounted on the external and internal longitudinal bars at 0, 200, and 400 mm from the segment’s centerline. Five concrete strain gauges were installed at mid- and quarter-spans to measure compressive strains. Segment deflection was monitored by five linear potentiometers (LPOTs): three distributed along the central width and two fastened at quarter-span. Two fixed high-accuracy LVDTs were installed diagonally on the lateral surface of the segments to measure shear-crack width. Flexural cracks were inspected visually, and their width was first obtained with an Elcometer crack-width ruler. Then, the cracks were highlighted and measured with three LVDTs.

![Typical Plan](image)

(a) punching  (b) shear

Figure 3.5 Locations of strain gauges.
3.6 Test Setup

In punching test, the specimens were placed on cylindrical steel supports covered with Teflon sheets to ensure free movement (Abbas et al. 2014). Two horizontal LPO Ts measured this deformation, which can be induced by the development of gaps between the segments due to imperfect placement during installation in the construction of tunnel lining segments (ACI 544.7R-16). All PCTL segments were point-loaded to failure over center-to-center spans of 1400 mm and 2400 mm for segments with small and large arc lengths, respectively. The load was applied to the segment using a circular loading plate with a diameter and thickness of 150 mm and 40 mm, respectively (Fig 3.6). The load was applied to the extrados face with a 1000 kN actuator at a controlled displacement rate of 0.5 mm/min.

The PCTL segments were loaded downward and upward under three-point loading over a simply supported center-to-center span of 1400 mm in shear tests (Figs 3.7 and 3.8). The line load was applied monotonically with an MTS 1000 kN actuator attached to a spreader beam at a stroke-controlled rate of 0.5 mm/min. Rubber sheets were placed between the beam and the tested segment to ensure smooth and uniform load distribution. Additionally, Teflon-covered steel cylinder supports were used to ease rotation and movement. The experiment was video recorded, and measurements were collected with a data-acquisition system.
Figure 3.6 Test setup of punching loading.
Figure 3. 7 Test setup of shear loading (upward).

Figure 3. 8 Test setup of shear loading (downward).
CHAPTER 4

PUNCHING-SHEAR BEHAVIOR OF GFRP-REINFORCED PRECAST CONCRETE TUNNEL SEGMENTS

Foreword

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Journal and Status:

ACI Structural Journal, under review.

Reference:


Note:

The manuscript had been slightly adjusted from the original paper by remembering the figures and tables to include the chapter number. In addition, the reference list has been moved to the appropriate section in the thesis as indicated in the table of content.
Abstract

The behavior of precast concrete tunnel lining segments (PCTLs) reinforced with glass fiber-reinforced polymer (GFRP) bars under punching loads is one area in which no research work has been conducted. This paper reports on an investigation of the punching-shear behavior of GFRP-reinforced PCTL segments induced by soil conditions, such as rock expansion or the geotechnical conditions surrounding a tunnel. Six full-scale rhomboidal PCTL specimens measuring 1500 mm (59 in.) in width and 250 mm (9.8 in.) in thickness were constructed and tested up to failure. The investigated parameters were: (1) reinforcement type (GFRP or steel); (2) reinforcement ratio (0.46% or 0.86%); (3) stirrups as shear reinforcement; and (4) segment length (2100 mm or 3100 mm [82.7 in. or 122 in.]). The experimental results are reported in terms of cracking behavior, punching-shear capacity, deflection, strain in the reinforcement and concrete, and failure modes. The results reveal that the GFRP-reinforced PCTL segment was comparable with its steel counterpart with the same reinforcement ratio and satisfied serviceability limits. Both increasing the reinforcement ratio and decreasing the segment length enhanced the punching-shear strength. The shear stirrups improved the structural performance and increased the punching and deformation capacities of the GFRP-reinforced PCTL segments. In addition, theoretical predictions of the punching-shear capacity using the current design provisions were compared to the experimental results obtained herein. The theoretical outcomes show the suitability of using current FRP design provisions for predicting the punching capacity of PCTL segments reinforced with GFRP bars.

Keywords: punching shear; precast concrete tunnel lining (PCTL) segments; glass fiber-reinforced polymer (GFRP) bars; shear stirrups; theoretical predictions; deflection and strain; segment length; reinforcement ratio.
4.1 Introduction

Tunnels are one of the types of infrastructure extensively developed in recent decades to improve the mobility of people and goods, as well as for utility purposes (i.e., hydropower, sewage, underground pipelines, and power). Tunnel lining systems play a relevant role in preserving the ground surfaces against large overburden loads and surrounding geotechnical conditions. The use of precast concrete tunnel lining (PCTL) segments in tunneling projects has been increasing because of its efficient and economical application compared to the conventional in situ lining technique (Elliott et al. 2002). PCTL segments are fabricated in precast plants to enhance control during the production process. PCTL fabrication clearly delivers significant waste reduction through the repetitive steps of batching and casting concrete. PCTL segments reduce time and enhance the safety of workers in underground conditions while providing superior quality. In essence, tunnel function depends on the efficiency, structural performance, and durability of its lining systems. The objective of designing PCTL segments is to develop suitable criteria to make the tunnel structures stable during their intended lives. Tunnel structures built with steel-reinforced concrete are designed for 100 years of service life. The main durability issue facing PCTL segments, however, is the corrosion of the steel reinforcement due to harsh environmental conditions. The durability of underground structures refers to their resistance to surrounding exposure effects such as seawater or the presence of chloride-contaminated water. The penetration of chloride ions through the surfaces of PCTLs—whether from groundwater or deicing salts—can disturb the passive layer around the reinforcing bars, resulting in corrosion initiation and a reduction in load-carrying capacity (Abbas et al. 2014). Corrosion of embedded reinforcing bars can induce internal pressures in the concrete. Once that occurs, the concrete cover could spall due to micro- and macrocracking, which might lead to catastrophic failure. In fact, the corrosion of steel reinforcement and the related deterioration of PCTL segments are the primary modes that accelerate failure and can jeopardize structural integrity. This premature degradation requires costly repairs and rehabilitation for tunnels. The Canadian Construction Association (CCA) has estimated that the global infrastructure loss would be in the vicinity of $900 billion (Mohamed et al. 2020).

Using glass fiber-reinforced polymer (GFRP) bars in PCTL segments as an alternative to corroding steel bars has emerged as a realistic, cost-effective solution, particularly when high
resistance to environmental attack is required. Compared to steel, GFRP does not experience corrosion problems, and its durability performance is a function of its constituent parts (Chen et al. 2007; El-Gamal et al. 2007; Mufti et al. 2005). GFRP also presents high tensile strength, high strength-to-weight ratio, long service life, and neutrality to electrical and magnetic disturbances, and it improves life-cycle cost efficiency (ACI 440.11-22). The use of non-metallic reinforcement such as GFRP bars can reduce the concrete cover to avoid crushing issues during segment handling because such areas are considered weak points in this kind of structure. Moreover, it creates dielectric joints in PCTLs and guarantees an effective remedy to the problem of stray currents and corrosion resulting therefrom (Caratelli et al. 2016). Recently, limited studies have been conducted to investigate the possibility of using GFRP reinforcement in tunnel segments (Moccichino et al. 2010; Caratelli et al. 2016, 2017; Spagnuolo et al. 2017, 2018; Meda et al. 2019; Hosseini et al. 2022a, 2022b; Ibrahim et al. 2023). Bending and edge thrust tests were conducted on full-scale PCTLs to assess their structural response. All these studies revealed the effectiveness and safety of employing GFRP bars in PCTL segments, especially for tunnel parts that have to be eventually removed (openings, niches, vent channels, and stations) to facilitate demolition and disposal. The laboratory results demonstrate the enhanced structural and durability performance of the GFRP-reinforced PCTL segments in terms of cracking behavior, deflection, and ultimate strength. Moreover, substituting GFRP reinforcement for conventional steel reinforcement satisfied the structural requirements. The GFRP curvilinear bars, however, had differences in mechanical and physical properties. The experimental results revealed the applicability of these bars for precast concrete segments in tunnel applications. Furthermore, the studies simulated the provisional and permanent loads that affect the life cycle of tunnels.

ACI 544.7R-16 and 533.5R-20 divided the loading conditions of PCTLs into three main stages: production and transient stage loads, loads for the construction stage, and loads for service stages. During production and transient stages, the internal forces and stress from demolding, storage, transportation, and handling are used in designing PCTLs. Construction loads include the jacking thrust loads created by the tunnel-boring machine (TBM) on the circumferential ring joints and the pressures exerted against the exterior of the completed rings during grouting operations. The final service stages represent the long-term loads imposed on the lining by earth pressure, groundwater, and surcharges. Other loads should be considered in the service stage based on the ground conditions, the tunnel function, and any special circumstances. The punching phenomenon is one
of the complex circumstances that can be induced by rock expansion or other geotechnical conditions above or beneath the tunnel. ACI 533.5R-20 and ITAWG2-19 consider that the TBM backup load induced by the self-weight of the backup equipment train behind the TBM shield applies a concentrated variable load on PCTLs and causes punching force. Nevertheless, very limited information is available in the literature on the strength and behavior of PCTL segments subjected to punching. Abbas et al. (2014) experimentally studied the punching behavior of full-scale PCTL segments reinforced with conventional steel reinforcement and steel fibers. They concluded that the ultimate punching strength of the conventional reinforced segment was much higher. The alternative solution, however, lowers production costs by eliminating the fabrication of steel cages, which is exhausting and costly.

The arched shape of the segment causes differences in the applied forces and stresses around the loaded area (Hosseini et al. 2022a). FRP-RC flat slabs/plates can, however, be used to understand the punching-shear behavior of PCTL segments. The punching failure in these elements relies on the development of internal load-resistance mechanisms. Shear reinforcement, amount and mechanical properties of flexure reinforcement, size effect, concrete characteristics, and span-to-depth ratio are considered important parameters that can influence punching resistance (Nguyen-Minh and Rovnak 2013; Hassan et al. 2017). Due to the lower modulus of GFRP bars, GFRP-reinforced slabs have demonstrated lower punching capacity than their counterparts reinforced with steel bars when the same amount of reinforcement was used (Hassan 2013). Increasing the reinforcement ratio resulted in high punching-shear capacity, reduced reinforcement and concrete strains, and decreased deflections (Bouguerra et al. 2011). Shear reinforcement—such as stirrups, studs, and bent-up bars—has been offered as an effective solution for improving the punching strength of slabs and preventing failure at low load levels. Using GFRP stirrups significantly increased the punching-shear and deformation capacity in one study (Truong et al. 2022). Nguyen-Minh and Rovnak (2013) reported that the effect of the span-to-effective depth ratio (L/d) should be considered in calculating the punching-shear resistance of FRP-RC slab-column connections. Decreasing the span-to-effective depth ratio reduced deflection and increased the punching capacity. Indeed, tunnel guidelines do not point out that there is evidence or requirements concerning the punching behavior of PCTL segments. Additionally, the punching-shear behavior of PCTLs reinforced with GFRP bars has not yet been investigated. The current study presents
experimental evidence to evaluate the behavior of GFRP-reinforced PCTL segments and the effect of different parameters under punching loads.

This presented study is a part of an ongoing extensive research program carried out at the University of Sherbrooke to investigate the structural performance of precast concrete tunnel lining segments reinforced with GFRP bars and stirrups. Punching-shear tests—in addition to static flexural loading (Hosseini et al. 2022a, 2022b); quasi-static cyclic flexural loading; and settlement—were conducted on full-scale GFRP-reinforced PCTL segments. The research project aimed to (1) assess the effect of design variables on the structural behavior of GFRP-reinforced PCTL segments; (2) improve current practices and develop more efficient design and construction approaches for using curvilinear GFRP bars and stirrups in PCTLs; and (3) determine the feasibility and efficiency of using GFRP instead of steel reinforcement under different loading conditions during production, shipping, construction, and service stages. This paper represents the first study on the structural behavior of full-scale GFRP-reinforced PCTL segments under punching-shear loading. The effects of different parameters—such as reinforcement type and ratio, shear stirrups, and segment length—were investigated.

4.2 Research Significance

Very limited data is available in the literature on the behavior and strength of PCTL segments subjected to punching-shear loads. The results for GFRP-RC flat slabs/plates cannot be directly applicable to GFRP-reinforced PCTL segments—given their curved geometry—particularly when investigating the effect of their numerous variables. Furthermore, the current design provisions in ACI 544.7R-16, ACI 533.5R-20, CSA S806-12, and ACI 440.11-22 do not address the design of PCTL segments reinforced internally with FRP bars. This study—which presents the first experimental results of full-scale precast RC segments reinforced with GFRP bars—contributes to understanding the general punching-shear behavior of such elements and enriches the state of the art. It also aimed to evaluate the effects of various parameters such as reinforcement type and ratio, shear stirrups, and segment length. A theoretical prediction was conducted to examine the accuracy of using the current punching equations in the code provisions (ACI-440.1R-15; CAN/CSA S806-12; fib TG-9.3 2007; JSCE 1997; BSI 1997; AASHTO 2018) for PCTL segments reinforced with GFRP bars (Hassan 2013). The findings provide valuable information for incorporating GFRP
reinforcement in PCTL segments, which can be an attractive solution to the corrosion problems and related maintenance and repair costs. Moreover, the knowledge gained in this study is valuable for designers, engineers, and members of code committees using GFRP reinforcement in infrastructure and for the development of codes and standards.

4.3 Experimental Program

4.3.1 Material Characteristics

The sand-coated GFRP reinforcement used in this study was manufactured by pultruding 78% (by weight) boron-free glass fibers impregnated with thermoset vinyl-ester resin, additives, and fillers. Newly developed curvilinear (Pultrall 2019) No.5 (15 mm [0.59 in.]) GFRP bars were employed to reinforce the GFRP-reinforced PCTL segments in the top and bottom longitudinal direction. Transverse reinforcement was provided as No. 4 (13 mm [0.51 in.]) closed GFRP ties. Number 4 (13 mm [0.51 in.]) and No. 5 (15 mm [0.59 in.]) U-shaped GFRP bars served as shear reinforcement and were used to ensure proper anchorage for the longitudinal reinforcement, respectively. All GFRP bars and ties had sand-coated surfaces to improve the bonding between the bars and the surrounding concrete. Figure 4.1(a) shows an overview of the GFRP bars and ties. Steel bars were used to reinforce the control segment specimen. Deformed 15M (16 mm [0.62 in.]) curvilinear steel bars were used as longitudinal reinforcement and No. 10M (11.3 mm [0.44 in.]) steel ties were provided as transverse reinforcement. The tensile properties of the longitudinal bars were determined according to ASTM D7205 (2021). Table 4.1 summarizes the tensile properties of the GFRP and steel reinforcement. All segmental tunnel specimens were cast with normal-weight concrete with a target 28-day compressive strength of 40 MPa (5.8 ksi). The actual compressive strength ($f'_c$) ranged from 40.7 MPa to 51.3 MPa (5.9 ksi to 7.4 ksi) based on the average test results of six concrete cylinders measuring 100 x 200 mm (3.94 x 7.89 in.).

(a)
4.3.2 Reinforcement Details and Segment Geometry

A total of six full-scale PCTL segments were constructed and tested under punching-shear loads. The test matrix was arranged to investigate the influence of the following parameters: (1) reinforcement type (GFRP or steel); (2) reinforcement ratio; (3) shear reinforcement; and (4) specimen length. Table 4.2 presents the test matrix and reinforcement details of the tested specimens. Segments were labeled with a letter denoting the reinforcement type (G for GFRP and S for steel bars and ties). The subscripts (x%) stand for the longitudinal reinforcement ratio of the top and bottom meshes, followed by the specimen arc length in meters; SR denotes shear reinforcement. The experimental program comprised five PCTL segments reinforced with GFRP bars and one reinforced with steel for comparison. Two different amounts of longitudinal
reinforcement were used to reinforce the GFRP specimens (7 and 13 No. 5 bars, with reinforcement ratios of 0.46% and 0.86%, respectively) for the top and bottom meshes. The segments were designed to have a flexural reinforcement ratio greater than the balanced reinforcement ratio (ACI 440.11-22). All GFRP segments were reinforced with No. 4 ties in the transversal direction at a spacing of 200 mm (7.87 in.). The control specimen was reinforced longitudinally with 7 No. 15M steel bars with a reinforcement ratio of 0.47% and transversely with 10M steel ties at 200 mm (7.87 in.). One of the GFRP segments had five discrete branches of No. 4 GFRP U-shaped stirrups as shear reinforcement to assess the stirrups’ contribution to punching and deformation capacity, as shown in Fig. 4.1(b). The shear reinforcement stirrups for typical slabs/plates reinforced with straight bars are arranged in a cruciform pattern (ACI 318-19, CSA A23.3-19). Nevertheless, the precast tunnel segments were fabricated with difficult geometry and complex configurations. A new technique was designed to arrange the shear reinforcement stirrups by adding the discrete stirrups symmetrically to the closed ties spaced at 0.25\(d\) and 2.25\(d\) (where \(d\) is the average effective depth of the segment) from the loading-plate face. The rhomboidal PCTL segment considered herein is one of seven different shapes of segments comprising full parallel rings with an internal diameter of 6500 mm (256 in.). Two average arc lengths of the tested segments were 2100 mm and 3100 mm (82.7 in. and 122 in.), as shown in Fig. 4.1(c), while the width and thickness were 1500 mm (59 in.) and 250 mm (9.8 in.), respectively. The clear concrete cover was kept constant at 40 mm (1.54 in.) for all segments. Figure 4.2 shows the geometry and reinforcement details of the testing specimens.

Table 4.2 Details of test specimens.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Reinf. Type</th>
<th>Arc Length (mm)</th>
<th>Longitudinal Reinforcement</th>
<th>Transverse Reinforcement</th>
<th>(f'_{c} ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G(_{(0.46)\text{-}2.1})</td>
<td>GFRP</td>
<td>2100</td>
<td>7 No. 5</td>
<td>No. 4 @200 mm</td>
<td>47.8</td>
</tr>
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<td>G(_{(0.86)\text{-}2.1})</td>
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<td>2100</td>
<td>13 No. 5</td>
<td>No. 4 @200 mm</td>
<td>41.6</td>
</tr>
<tr>
<td>G(_{(0.46)\text{-}3.1})</td>
<td>GFRP</td>
<td>2100</td>
<td>7 No. 5</td>
<td>No. 4 @200 mm</td>
<td>47.1</td>
</tr>
<tr>
<td>G(_{(0.86)\text{-}3.1})</td>
<td>Steel</td>
<td>3100</td>
<td>7 No. 5</td>
<td>No. 4 @200 mm</td>
<td>51.3</td>
</tr>
<tr>
<td>S(_{(0.47)\text{-}2.1})</td>
<td></td>
<td>3100</td>
<td>13 No. 5</td>
<td>No. 4 @200 mm</td>
<td>48.8</td>
</tr>
</tbody>
</table>

1 \(G\) denotes GFRP bars and stirrups and \(S\) denotes steel, with a subscript (x\%) indicating longitudinal reinforcement ratio, followed by specimen arc length.

2 \(SR\) denotes shear reinforcement represented by 5 additional legs for the middle fourth ties at the punching region.

3 \(\rho_b\) calculated according to ACI 440-11-22 and ACI 318-19 for GFRP and steel-reinforced concrete slabs, respectively.

4 \(f'_{c}\) based on six 100 x 200 mm cylinder tests.

Note: 1 mm = 0.0394 in.; 1 MPa = 145.04 psi.
Figure 4.2 Test specimens’ geometry and reinforcement details.

4.3.3 Specimen Fabrication Details

GFRP and steel cages were assembled for the various configurations at the University of Sherbrooke. The specimens were shipped to a precast plant (Sym-Tech precast company) in Saint-Hyacinthe (QC, Canada) for instrumentation and casting. Curved wooden formwork was designed and constructed to fabricate the PCTL segments. Concrete was cast into the specimen mold using an overhead crane bucket. The segments were compacted with an electrical vibrator to ensure adequate consistency and then leveled manually. A plastic sheet covered the entire mold to cure
the segments and eliminate steam and moisture losses. The concrete strength reached sufficient strength after 24 hours. The specimens were stripped and then placed in a storage yard. Figure 4.3 illustrates the fabrication process of the PCTL segments.

![Fabrication and preparation of PCTL segments: (a) formwork, (b) casting, (c) stripping, and (d) storage.](image)

**4.3.4 Instrumentation and Test Setup**

Each PCTL specimen was equipped with two instrumented bars (curvilinear and closed tie) in the orthogonal directions. The instrumentation consisted of five electrical strain gauges attached to the middle bars in the bottom mat (tension side) and one additional strain gauge for the longitudinal direction of the long span. To obtain the longitudinal and transverse strain profiles, the gauges were located at 0, 100, 200, 300, and 400 mm (0, 3.9, 7.9, 11.8, and 15.7 in.) from the loading plate face. Two strain gauges in each orthogonal direction were glued to the top of the reinforcement (compression side) at the plate face. Strain in the shear stirrups was measured
through two gauges mounted at mid-height of the vertical legs. In addition, five concrete strain gauges (C1 to C5) were mounted on the concrete surface. The deflection at mid- and quarter-span was measured with five linear potentiometers (LPOTs), both longitudinal and transverse. The specimens were placed on cylindrical steel supports covered with Teflon sheets to ensure free movement (Abbas et al. 2014). Two horizontal LPOTs measured this deformation, which can be induced by the development of gaps between the segments due to imperfect placement during installation in the construction of tunnel lining segments (ACI 544.7R-16). All PCTL segments were point-loaded to failure over center-to-center spans of 1400mm and 2400 mm (55.1 in. and 94.5 in.) for segments with small and large arc lengths, respectively. The load was applied to the segment using a circular loading plate with a diameter and thickness of 150 mm and 40 mm (5.9 in. and 1.57 in.), respectively, as shown in Fig. 4.4. The load was applied to the extrados face with a 1000 kN actuator at a controlled displacement rate of 0.5 mm/min. The first crack was monitored visually and measured with a gauge card. Then, the crack propagation was marked and measured with three high-accuracy LVDTs installed at the crack’s location. A data-acquisition system was used to record all measurements during the test.

Figure 4. 4 Test setup.
4.4 Test Results

4.4.1 Cracking Pattern and Load

Figure 4.5 shows similar crack pattern propagation for all tested PCTL specimens. However, the final crack pattern for segments that had a large arc span length (G_{(0.46)-3.1} and G_{(0.86)-3.1}) was different. Generally, the first crack that formed on the tension side of all segments (intrados surfaces) was flexural underneath the loaded region and oriented in the transverse direction parallel to the steel supports. The cracking load ranged between 94.8 and 141.3 kN (21.3 and 31.7 kips) [at about 33% to 44% of the ultimate loads] for shorter specimens (G_{(0.46)-2.1}, G_{(0.86)-2.1}, G_{(0.46)-2.1-SR}, and S_{(0.47)-2.1}). Sequentially, radial cracks started from free edges and progressively extended to interfere with the transversal cracks. Tangential cracks at higher loads were observed around the leading plate and crossed over the radial cracks to form the classical punching cone. The circular fan-type cracks (radial and circumferential) were more detectable in the short specimens; their number and width increased in the vicinity of the plate. Further, new longitudinal and diagonal cracks initiated in intrados surfaces near the steel supports and extended toward other cracks. At the end of the test, the loading plate sank into the concrete surface, accompanied by transversal cracks on the compression side and inclined shear cracks at the free edges. The first flexural (transverse) cracks in specimens G_{(0.46)-3.1} and G_{(0.86)-3.1} started at a load of 51.9 and 62.7 kN (11.7 and 14.1 kips), respectively, at 22% of the ultimate load. Numerous transverse cracks then developed beyond the loaded area. In addition, intermittent radial cracks spread across the bottom surface to connect the transverse cracks. At higher loads, major inclined shear cracks formed on the free edges of specimens G_{(046)-3.1} and G_{(0.86)-3.1} at angles of 47° and 17°, respectively.
4.4.2 Ultimate Punching-Shear Capacity and Failure Modes

The idealized failure mechanism for segments $G_{(0.46)}-2.1$ and $G_{(0.86)}-2.1$ was observed as a punching failure mode (P). There was a sudden drop in load capacity and the typical shape of a punching cone was formed. The radius ($R_{cone}$) from the plate face to the location of the failure envelope was measured to define the failure surface of each specimen. Table 4.3 summarizes the ultimate punching loads and radii of the failure surfaces. Specimen $G_{(0.46)}-2.1$ failed at a load level of 288 kN (64.6 kips). A typical brittle failure occurred without much warning, characterized by slight penetration of the point load (Fig. 4.6(a)) and a major transverse crack at the center of the extrados surface. On the tension surface, there was an average limit to the radius $R_{cone}$ of about 1.1$d$. Moreover, spalling of the concrete cover near the supports was observed. Specimen $G_{(0.86)}$-
2.1 failed at a load of 356 kN (79.9 kips) with failure of the intrados surface similar to specimen G_{0.46}-2.1 and a flatter inclination angle of shear cracks. As a result, the radius ($R_{cone}$) of the punching failure surface increased to $2.1d$, which was expected from existing experimental evidence (Hassan 2013). The penetration distance was adequate to separate the concrete at the level of the bottom mesh along the lateral side. The failure load and radius for specimen G_{0.46}-2.1-SR were 326 kN (73.3 kips) and $0.9d$, respectively, as the failure manner of specimen G_{0.46}-2.1. Several splitting cracks were generated near the loading plate on top of the shear stirrups. Moreover, no rupture was observed in the shear stirrups, as the corresponding strain was equal to $1320 \mu \varepsilon$ at failure. These cracks were considered premature warning signs before punching failure occurred. Shear stirrups eliminated and converted the brittle failure to a ductile mode (DP) (Marzouk et al. 1997). Beyond failure, the concrete was crushed, and the stirrups pushed out of the top surface, as presented in Fig. 4.6(b). Specimens G_{0.46}-3.1 and G_{0.86}-3.1 failed at loads of 233 and 280 kN (52.1 and 62.8 kips), respectively. Both segments experienced mixed flexural-punching failure mode (FP), which can be attributed to concrete crushing on the compression surface and an increase in deformation capacity (Dam et al. 2017). Moreover, the failure was more ductile for specimen G_{0.46}-3.1, evidenced by a gradual drop in load capacity and a considerable increase in deflection. Once the lead plate penetrated the concrete cover, the upper middle strip detached (Fig. 4.6(c)). Hosseini et al. (2022a) tested the same segment under line loading and it failed in pure flexural mode. The failure load and deformation capacity, however, were higher than in G_{0.46}-3.1, which confirms the difference between the two failures. The punching cone did not develop in either segment as the radial intermittent cracks could not form a complete circle. The failure surface, however, the radius ($R_{cone}$) was approximately $3.6d$ for G_{0.86}-3.1 and $0.8d$ for G_{0.46}-3.1. The control specimen (S_{0.47}-2.1) failed at a load of 321 kN (72.1 kips). The steel bars near the loading area yielded prior to punching failure, a sign that flexural failure (F) had occurred. Thereafter, the specimen experienced separation on the thrust side at the level of the top mesh. The plate pushed through the concrete with an instant drop in loading; the failure surface was measured at $1.6d$. Further, the arched shape of the segments affected the resistance of the punching behavior. This could be attributed to changing the applied forces and stresses around the curvature load region. Figure 4.7 shows the failure surface for all tested segments.
Table 4. 3 Summary of test results.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>$\rho$/$E/E_s$</th>
<th>Cracking</th>
<th>Ultimate</th>
<th>$R_{cone}$, $\mu$m at plate face</th>
<th>$\epsilon_{max}$, $\mu$m at plate face</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>G(0.46)-2.1</td>
<td>0.13</td>
<td>94.8</td>
<td>1.25</td>
<td>75.84</td>
<td>79.3</td>
<td>2.05 P</td>
</tr>
<tr>
<td>G(0.86)-2.1</td>
<td>0.24</td>
<td>141.3</td>
<td>3.26</td>
<td>43.34</td>
<td>102.7</td>
<td>1.55 P</td>
</tr>
<tr>
<td>G(0.46)-2.1-SR</td>
<td>0.13</td>
<td>109.7</td>
<td>1.35</td>
<td>79.78</td>
<td>90.3</td>
<td>2.36 DP</td>
</tr>
<tr>
<td>G(0.46)-3.1</td>
<td>0.13</td>
<td>51.9</td>
<td>1.22</td>
<td>42.54</td>
<td>62.7</td>
<td>5.03 FP</td>
</tr>
<tr>
<td>G(0.86)-3.1</td>
<td>0.24</td>
<td>62.7</td>
<td>2.07</td>
<td>30.29</td>
<td>76.6</td>
<td>3.50 FP</td>
</tr>
<tr>
<td>S(0.47)-2.1</td>
<td>0.47</td>
<td>141.1</td>
<td>2.10</td>
<td>67.19</td>
<td>321</td>
<td>1.23 F</td>
</tr>
</tbody>
</table>

Note: $V_u$ and $\Delta_u$ for the control specimen were calculated at the yield of steel bars; 1 mm = 0.0394 in.; 1 kN = 0.225 kip; 1 kN/mm = 5.71 kip/in.
1 Denotes the effective reinforcement ratio.
2 Denotes normalized ultimate punching-shear capacity.
3 Denotes the radius from plate face to punching failure surface.
4 Observed failure modes: P is punching; DP is ductile punching; F is flexural due to steel yielding, and FP is mixed flexure and punching.
Figure 4. 6 (a) punching concrete surface; (b) pushing of shear stirrups; (c) concrete crushing.

Figure 4. 7 Failure surface of the tested specimens.
4.4.3 Deflection Behavior

Figure 4.8 plots the applied load versus the maximum deflection at the specimen center. The load–deflection curve of all tested PCTLs has three distinct portions. The first portion demonstrates similar stiff pre-cracking behavior. All segments behaved linearly up to the appearance of the initial flexural crack. Specimens G_{(0.46)-2.1}, G_{(0.86)-2.1}, S_{(0.47)-2.1}, and G_{(0.46)-2.1-SR} had nearly the same uncracked response. This could be ascribed to the negligible effect of the reinforcement type, reinforcement ratio, or shear stirrups on the gross moment of inertia in the tunnel cross section. Consequently, the concrete primarily resisted the flexural stresses at the uncracked stage (Elgabass et al. 2016). The flexural stresses increased in specimens G_{(0.46)-3.1} and G_{(0.86)-3.1} due to their high arc length. Therefore, the first flexural cracks appeared at a low load level, causing a decrease in the pre-cracking stiffness. The post-cracking deflection until failure was provided in the second portion. With further loading, the number and width of the flexural cracks increased and changed the section properties from gross to effective. At this stage, the deflection of all segments with GFRP reinforcement increased nearly linearly due to the linear elastic behavior of the GFRP bars, with a significant reduction in the post-cracking stiffness. In contrast, the segment with steel reinforcement exhibited an initially steep linear response till yielding, followed by a short plateau of nonlinear behavior. Table 4.3 presents the deflection and stiffness corresponding to the load at the cracking and ultimate stages. Lastly, the third portion presents the post-peak behavior, which relied on the mode of failure for each specimen. All specimens exhibited a sudden decay in the load–deflection relationship, followed by an increase in deflection, except for specimen G_{(0.46)-3.1}, which had a soft descending branch. The horizontal deformations were minimal; however, these values simulated the actual deformation during the construction of PCLTs.
4.4.4 Longitudinal and Transverse Reinforcement Strains

Figures 4.9(a) and (b), respectively, depict the load–strain behavior at plate face of the longitudinal bars and transverse closed ties. In these figures, the strain of the flexural reinforcement was measured at 75 mm (2.95 in.) from the centerline of the loading plate. Prior to the onset of flexural cracking, a minimal strain was recorded for all types of reinforcement. The load–flexural strain behavior for all segments was similar to their load–deflection curves. The longitudinal bars, however, exhibited significant linear response after the cracking stage up to failure. In contrast, in the transverse direction, the load values dropped and the strain gauges of the ties continued to read more strain after failure. This might be attributed to the loading mechanism being mainly dependent on the longitudinal bars, which transfer loads to supports found only in this direction. For the GFRP-reinforced specimens, the maximum strains ($\varepsilon_{\text{max}}$) in the longitudinal bars were 8785, 6460, 10180, 16100, and 10315 $\mu$ε for specimens $G_{(0.46)-2.1}$, $G_{(0.86)-2.1}$, $G_{(0.46)-2.1}$-$\text{SR}$, $G_{(0.46)-3.1}$, and $G_{(0.86)-3.1}$, respectively, representing 43%, 32%, 50%, 80%, and 51% of the
guaranteed tensile strength. The $\varepsilon_{\text{max}}$ for the transverse ties was 3180, 2850, 3900, 4440, and 3390 $\mu$e, respectively, representing 0.36, 0.44, 0.38, 0.28, and 0.33 times those measured along the longitudinal bars, respectively. In this context, the ultimate low strain at peak load in all segments indicates that the failures were not triggered by the rupture of the GFRP reinforcement in all the specimens. Figure 4.10 provides the flexural strain distribution along the span of longitudinal and transverse directions, respectively. It was obtained for each specimen from electrical strain gauges located at 0, $d/2$, $d$, $1.5d$, and $2d$ from the plate face. The strain values were typically high beneath the applied load and decreased proportionally toward the segment supports, which is consistent with past studies (Benmokrane et al. 2007; Hassan et al. 2013; Eladawy et al. 2019). This confirms that the GFRP bars transferred load without signs of bar slippage or bonding degradation during the test. The large-span specimens, however, exhibited random increases along the strain profile in both directions. This could be attributed to the formation of intermittent radial shear cracks at the intrados surface close to the gauge location. Furthermore, the strain in the top reinforcement was negligible in comparison with the bottom tensile strains. The steel-reinforced segment $S_{(0.47)}$-2.1 showed a clear yielding of the longitudinal bars at an applied load equal to 76% of the ultimate load. A steeper increase in load–strain behavior was observed up to ultimate load and the maximum strain reached 3975 $\mu$e. The transverse ties showed no signs of yielding at the peak load. Both the longitudinal and transverse strain profiles of the steel specimen are similar to those reported by Ospina et al. (2003). Ultimately, all specimens exhibited a higher strain in the longitudinal bar than the transverse tie at the same load level due to loading in one direction.

Figure 4.9 Load-strain relationship at plate face of (a) longitudinal bar and (b) transverse tie.
Figure 4. 10 Strain profiles for longitudinal bars and transverse stirrups.
4.4.5 Stirrups and Concrete Strains

Figure 4.11 plots the load versus strain relationship for the shear reinforcement stirrups and concrete. The strains were measured at mid-height of the legs of the vertical stirrups for specimen G_{(0.46)}-2.1-SR at 0.25\(d\) and 2.25\(d\) from the plate face, whereas the concrete strains were recorded at the plate face (gauge C1). Figure 4.11(a) implies that the GFRP stirrups at \(d/2\) contributed to the punching-shear resistance before the stirrups at 2.25\(d\) because they were closer to the load region. The maximum strains at 0.25\(d\) and 2.25\(d\) were 2075 \(\mu\varepsilon\) and 1720 \(\mu\varepsilon\), respectively. In particular, the contribution of the stirrups to punching resistance was negligible before the radial shear cracks appeared. A significant jump in stirrup strain was observed as the circular fan cracks developed due to the transfer of forces across these cracks. This, in turn, delayed the widening of cracks and increased the punching capacity (Rizk et al. 2011). The concrete strains in specimens G_{(0.46)}-2.1, G_{(0.86)}-2.1, and G_{(0.46)}-2.1-SR were -2385, -2990, and -2960 \(\mu\varepsilon\), respectively, which were less than the theoretical crushing failure of 3000 \(\mu\varepsilon\) specified in ACI 440.11-22 and 3500 \(\mu\varepsilon\) as per CSA S806-12. Furthermore, no signs of concrete crushing were detected during the test. This confirmed that the final failure mode was punching shear. In contrast, specimens G_{(0.46)}-3.1, G_{(0.86)}-3.1, and S_{(0.47)}-2.1 recorded maximum concrete strains of -4480, -5650, and -5465 \(\mu\varepsilon\), respectively (Fig. 4.11(b)). Mixed flexural–punching failure was observed for segments with high length due to the high relatively strains. Specimen S_{(0.47)}-2.1, however, failed in steel yielding mode at a concrete strain of 2140 \(\mu\varepsilon\) before punching occurred.

![Figure 4.11 Load-strain relationship of (a) shear stirrup at 0.25d and 2.25d and (b) concrete surface.](image)
4.4.6 Crack Opening

Figure 4.12(a) illustrates the maximum measured crack opening (at the intrados tension surface under the loaded region) against the applied load for all tested specimens. The figure demonstrates that the crack opening for specimens reinforced with GFRP bars varied linearly till punching failure occurred. The steel specimen behaved initially linearly until the steel reinforcement yielded. Inclined shear cracks then formed and widened to punching failure, resulting in nonlinear cracking behavior. The maximum recorded crack widths were 2.05, 1.55, 2.36, 5.03, and 3.50 mm (0.08, 0.06, 0.09, 0.20, and 0.14 in.) for specimens $G_{(0.46)}$-2.1, $G_{(0.86)}$-2.1, $G_{(0.46)}$-2.1-SR, $G_{(0.46)}$-3.1, and $G_{(0.86)}$-3.1, respectively. Increasing the reinforcement ratio and decreasing the segment length significantly improved the structural integrity of the GFRP-reinforced PCTL segments when the crack numbers and width are the dominant criteria. Steel bars in specimen $S_{(0.47)}$-2.1 had a crack width equal to 0.73 mm (0.03 in.) when the steel bars yielded; the maximum crack width prior to punching was 1.23 mm (0.05 in.). Figure 4.12(b) presents the relationship between the maximum crack openings and the corresponding measured strain in the longitudinal tension bars. This figure confirms that the values of the crack opening are directly proportional to the experimental strain in all loading stages. Early loading revealed a few flexural cracks, often with small openings, which, in turn, maintained normally close relationships. Up to the reference point, the crack opening was also in close relation to the measured strain and less than 0.5 mm (0.02 in.) for all specimens except $G_{(0.46)}$-3.1. The reference point was identified at a strain equal to 2000 $\mu\varepsilon$ (strain limit specified in ISIS 2007 to keep the crack opening to less than 0.5 mm [0.02 in.]).

Figure 4.12 Crack-width relationships.
4.5 Discussions

4.5.1 Influence of the Reinforcement Type

Specimens G\textsubscript{(0.46)-2.1} and S\textsubscript{(0.47)-2.1} were designed to have the same longitudinal reinforcement ratio. Specimen G\textsubscript{(0.46)-2.1} had pre-cracking stiffness 12\% higher than S\textsubscript{(0.47)-2.1} due to its high compressive strength. The crack patterns propagated similarly regardless of the reinforcement type. The punching-shear capacity was normalized to the cubic root of the concrete strength to minimize the influence of its variation (Eladawy et al. 2019). In addition, for each type of reinforcement, the effective reinforcement ratios (\(\rho E_f / E_s\)) were used to account for the difference in modulus of elasticity. As shown in Fig. 4.13(a), the normalized punching-shear capacity for the GFRP specimen was 15\% lower than its steel counterpart. Moreover, the post-cracking stiffness of specimen S\textsubscript{(0.47)-2.1} was 3.7 times higher than that of specimen G\textsubscript{(0.46)-2.1}. This can be attributed to the GFRP reinforcement having lower moduli of elasticity than that of the steel (\(\approx 0.27\)), which is consistent with past findings (Mousa et al. 2019a). The type of reinforcement influenced the mode of failure. The steel-reinforced specimen might have failed similarly to the slab referred to by Ghali and Gayed (2018). They concluded that the slab with a low reinforcement ratio (approximately less than 0.6\% to 0.7\%) might have failed in flexure before what looked like punching failure in the end. Given the same load level, using GFRP reinforcement at the same ratio as the steel reinforcement yielded higher strain, wider final cracks, and higher deflection, which, in turn, decreased the punching capacity.

4.5.2 Influence of the Longitudinal Reinforcement Ratio

Four GFRP specimens (G\textsubscript{(0.46)-2.1}, G\textsubscript{(0.46)-3.1}) and (G\textsubscript{(0.86)-2.1}, G\textsubscript{(0.86)-3.1}) were designed with two reinforcement ratios of 0.46\% and 0.86\%, respectively. Increasing the reinforcement ratio significantly affected the cracking behavior, failure surfaces, punching capacities, deformations, and crack opening. Regardless of the reinforcement ratio, the cracks propagated in the same manner. The specimens with the lower reinforcement ratio, however, had more flexural cracks around the loading plate. In addition, the radius of the punching failure surface \(R_{cone}\) was 4.6 and 1.9 times higher than that of specimens G\textsubscript{(0.46)-2.1} and G\textsubscript{(0.46)-3.1}, respectively. This is in good agreement with the findings of Hussein et al. (2018), who reported that the reinforcement ratio was proportional to the flatter angle of inclined shear cracks and consequently increased the failure
Furthermore, the radius of failure surfaces for specimens $G_{(0.86)-2.1}$ and $G_{(0.86)-3.1}$ exceeded the radius of the critical section ($0.5d$, $1.5d$, or $2d$) specified in the available standards governing design punching shear. The uncracked concrete depth (compression region) increased with the higher reinforcement ratio, which decreased the strains and crack openings at the same load level. The aggregate interlock and dowel action were also increased, which, in turn, enhanced the concrete’s contribution. As a result, the increased reinforcement ratio considerably enhanced the punching capacity (Fig. 4.13(b)).

4.5.3 Influence of the Arc Span Length

Increasing the segment length from 2100 to 3100 mm (82.7 in. to 122 in.) had a pronounced effect on the cracking behavior, failure mode, stiffness, and normalized punching-shear and deformation capacities. The cracking loads of specimens $G_{(0.46)-2.1}$ and $G_{(0.86)-2.1}$ were 83% and 125% greater than their counterparts, respectively. Moreover, the pre-cracking stiffness was 1.8 times higher for $G_{(0.46)-2.1}$ and 1.4 times higher for $G_{(0.86)-2.1}$. Specimens with a short span exhibited punching-shear failure. In contrast, the specimens with a long span exhibited flexural-punching failure with more flexural cracks, even outside the loading area. This could be attributed to increasing the flexural stresses in specimens $G_{(0.46)-3.1}$ and $G_{(0.86)-3.1}$. The short arc span increased the post-cracking stiffness by 165% for $G_{(0.46)-2.1}$ and 150% for $G_{(0.86)-2.1}$. Figure 13(b) plots the normalized punching-shear load corresponding to the deflection for the four specimens to investigate the effectiveness of the span length. The figure indicates that $G_{(0.46)-2.1}$ had an ultimate normalized punching load 26% higher than that of $G_{(0.46)-3.1}$. Similarly, the ultimate normalized punching capacity of $G_{(0.86)-2.1}$ was 34% higher than that of $G_{(0.86)-3.1}$. Moreover, the ultimate deflections increased for specimens $G_{(0.46)-3.1}$ and $G_{(0.86)-3.1}$ by 139% and 120%, respectively, resulting in higher deformation capacity and more ductile behavior. Increasing the span length yielded higher tensile stresses, which significantly decreased the depth of the neutral axis. This, in turn, increased the strains and the crack widths at the same load level and decreased the punching-shear strength.

4.5.4 Influence of the Shear Reinforcement

Placing the GFRP stirrups within the punching-shear region of specimen $G_{(0.46)-2.1}$-SR enhanced its punching resistance. Both segments experienced similar crack-pattern propagation and
punching-shear failure. Specimen G_{(0.46)-2.1-SR} exhibited a ductile punching mode of failure with precursors in the form of splitting cracks above the legs of stirrups on the extrados surface. The use of shear stirrups decreased the radius of the punching-shear failure cone ($R_{cone}$). The ultimate normalized punching load of specimen $G_{(0.46)-2.1-SR}$ was 14% higher than that of $G_{(0.46)-2.1}$ with a 7% increase in ultimate deflection (from 19.55 mm to 20.95 mm [0.77 in. to 0.82 in.]) (Fig. 4.13(a)). The test results reveal that the crack opening and strain value were smaller for $G_{(0.46)-2.1-SR}$ at the same load level. In fact, using the GFRP stirrups around the punching zone mobilized the longitudinal reinforcement to achieve lower strains. This could be attributed to the confinement of the plate vicinity, which forced the strain of the flexural reinforcement to decrease. According to Lips et al. (2012), GFRP shear stirrups have good anchorage to contribute to punching-shear resistance, even when small in number. Further research is needed to determine the distribution, amount, and extension limit of GFRP shear stirrups in PCTL segments due to their complex curvature cages.

![Figure 4.13](image)

Figure 4.13 (a) effect of reinforcement type and shear and (b) effect of reinforcement ratio and span length on the normalized punching load–deflection relationship.

### 4.6 Theoretical Prediction

#### 4.6.1 Effect of the Curved Shape on Segment Capacity

Figure 4.14 shows the arched shape effect on the acting loads applied to PCTL segments during the test. The forces and moment equilibrium were employed to calculate the actual normal, shear, and moment at the center of specimens, as expressed in the following equations:
Normal = \( R \sin \theta = \frac{V}{2 \sin \theta} \)  
(4.1)

Shear = \( R \cos \theta = \frac{V}{2 \cos \theta} \)  
(4.2)

Moment = \( R(0.5l \cos \theta + \Delta \sin \theta) \)  
(4.3)

where \( l \) and \( \Delta \) are the distance between support reactions on the centerline and the vertical distance at mid-span (mm), \( R \) is the reaction force of support (kN), and \( \theta \) is the inclination angle of the reaction forces in the test setup (rad).

Figure 4. 14 Applied forces and effect of curved shape on segment’s capacity.

4.6.2 Predictions of Punching-Shear Capacity

The tunnel codes and design guidelines (ACI 544.7R-16 and ACI 533.5R-20) do not provide any equations that can be employed to assess the punching capacity of PCTL segments reinforced internally with FRP bars. Therefore, the calculation of the punching-shear resistance was performed according to the normative predictions with ACI-440.1R-15, CAN/CSA S806-12, fib TG-9.3 (2007), JSCE (1997), BSI (1997), and AASHTO (2018). The equations of the current provisions are based on the original steel equations with some amendments to account for the difference in the mechanical properties between the two reinforcement types. Table 4.4 summarizes the available expressions for punching capacity.
Table 4. 4 Punching-shear capacity equations.

<table>
<thead>
<tr>
<th>Code</th>
<th>Resistance of segments without shear stirrups</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI 440.11 (2022)</td>
<td>$V_c = \frac{4}{5} \sqrt{f'<em>c b</em>{0.5d}} (kd)$</td>
</tr>
<tr>
<td></td>
<td>$k = \sqrt{2\rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f$</td>
</tr>
<tr>
<td></td>
<td>Where $n_f = E_f / E_c$ ; $E_c = 4700\sqrt{f'_c}$</td>
</tr>
<tr>
<td>CAN/CSA S806-12 (2012)</td>
<td>The least of the following equations:</td>
</tr>
<tr>
<td></td>
<td>$V_r = V_c = \left(1 + \frac{2}{\beta_c} \right) \left[0.028\lambda \phi \left(E_f \rho_f f'<em>c\right)^{1/3}\right] b</em>{0.5d} d$</td>
</tr>
<tr>
<td></td>
<td>$V_r = V_c = \left[\frac{\alpha_s d}{b_o} + 0.19\right] \left[0.147\lambda \phi \left(E_f \rho_f f'<em>c\right)^{1/3}\right] b</em>{0.5d} d$</td>
</tr>
<tr>
<td></td>
<td>$V_r = V_c = 0.056\lambda \phi \left(E_f \rho_f f'<em>c\right)^{1/3} b</em>{0.5d} d$</td>
</tr>
<tr>
<td>fib TG-9.3 (2007)</td>
<td>$V_c = \left(\frac{5}{2} k\right) 0.33\sqrt{f'<em>c b</em>{0.5d} d}$</td>
</tr>
<tr>
<td>JSCE (1997)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$V_c = \beta_d \beta_p \beta_r \beta_{f,pa} b_{0.5d} / \gamma_b$</td>
</tr>
<tr>
<td></td>
<td>$\beta_d = (1000 / d)^{1/4} \leq 1.5$</td>
</tr>
<tr>
<td></td>
<td>$\beta_p = (100 \rho_f E_f / E_s)^{1/3} \leq 1.5$</td>
</tr>
<tr>
<td></td>
<td>$\beta_r = 1 + 1/(1 + 0.25u / d)$</td>
</tr>
<tr>
<td></td>
<td>$f_{f,pa} = 0.2\sqrt{f'_c} \leq 1.2$ MPa</td>
</tr>
<tr>
<td>BS 8110 (1997)</td>
<td>$V_c = 0.79 \left[100 \rho_f (E_f / E_s)^{1/3} \left(f_{cu} / 25\right)^{1/3} (400 / d)^{1/4} b_{0.5d} d \right]$</td>
</tr>
<tr>
<td>AASHTO (2018)</td>
<td>$V_c = 0.316 k\sqrt{f'<em>c b</em>{0.5d} d}$ kips</td>
</tr>
</tbody>
</table>

$\beta$, $\gamma$, $\alpha$, ksi, in.

4.6.3 PCTL Segment with Shear Stirrups

The punching-shear resistance of FRP-RC slabs within the shear-reinforced zone is computed as the sum of concrete shear resistance ($v_c$) and the shear-stirrup contribution ($v_f$). The concrete shear strength was calculated considering the effect of longitudinal flexural FRP bars, as in past provisions. None of the current codes and design guidelines for concrete slabs reinforced with FRP bars provide equations for calculating the contribution of the shear stirrups, except AASHTO.
The nominal shear resistance—as specified in AASHTO (2018)—provided by single or multiple-leg GFRP stirrups, \( V_f \) in kips, shall be taken as:

\[
V_f = \frac{A_{f_v} f_{f_v} d_v}{S_{f_v}}
\]

(4.12a)

\[
f_{f_v} = 0.004 E_f \leq f_{f_b}, \ E_f \text{ in ksi}
\]

(4.12b)

Where \( f_{f_b} \) and \( f_{f_v} \) are the design tensile strength of the bent portion of the GFRP reinforcing bar and shear stirrups, \( A_{f_v} \) is the sum of the area of all legs of reinforcement on one peripheral line that is geometrically similar to the perimeter of the circular plate (in.\(^2\)), and \( S_{f_v} \) is the spacing between the peripheral lines of shear reinforcement in the direction perpendicular to the plate face (in.). The distribution of the shear stirrups was unique according to the curvature geometry of PCTL cages and was not mentioned, even in the steel-related codes. Thus, a simplified method was used to identify \( A_{f_v} \) and \( S_{f_v} \), in which every peripheral line with the same perimeter of the circular plate passes through four legs of shear stirrups and the spacing of the peripheral lines was 250 mm (9.8 in.). The contribution of the shear stirrups for specimen G\(_{(0.46)}\)-2.1-SR (AASHTO 2018) was added to the predicted punching-shear capacities for all provisions, as reported in Table 4.5.

### 4.6.4 Comparison Between Experimental and Predicted Results

The prediction is mainly dependent on the available equations for FRP-RC codes and design guidelines to estimate the applicability of these standards for calculating the punching capacity of GFRP-reinforced PCTL segments. As shown in Table 4.5, the accuracy of the predictions of punching-shear capacity versus the experimental values obtained was assessed. The safety factors incorporated in all the punching-shear equations were considered equal to unity. In comparison, ACI 440.1R-22, the modification equation suggested by fib TG-9.3 (2007), and AASHTO (2018) yielded excessively conservative predictions with an average \( V_{exp}/V_{pred} \) of 1.72 ± 0.27, 1.67 ± 0.26, and 1.66 ± 0.26, respectively, and a COV of 15%. Moreover, each specimen separately had a highly conservative prediction regardless of the variation in the test parameters. This can be attributed to the concrete’s contribution to resistance that involves only the flexural reinforcement material and ratio in predicting the depth of the uncracked section, while the other variables are
neglected. CAN/CSA S806-12 yielded accurate predictions with an average $V_{\text{exp}}/V_{\text{pred}}$ of 0.98 ± 0.13 and a corresponding COV of 13% due to the direct introduction of axial stiffness for FRP bars into the punching equation. In contrast, JSCE (1997) and BSI (1997) yielded good predictions with an average $V_{\text{exp}}/V_{\text{pred}}$ of 1.07 ± 0.12 and 0.96 ± 0.13 with corresponding COVs of 11% and 13%, respectively. These equations are reliable because they account for the size effect as well as the reinforcement axial stiffness. Nonetheless, CAN/CSA S806-12, JSCE (1997), and BSI (1997) overestimated predictions for specimens $G_{(0.46)}$-3.1 and $G_{(0.86)}$-3.1, which confirms the influence of neglecting the span length in the prediction equations. Lastly, adding the contribution of shear stirrups to the concrete shear resistance in specimen $G_{(0.46)}$-2.1-SR yielded better predictions. According to the ACI 440.11-22 and CAN/CSA S806-12 equations, the flexural capacities for segments $G_{(0.46)}$-2.1, $G_{(0.86)}$-2.1, and $G_{(0.46)}$-2.1-SR were significantly higher than the punching capacity. Due to increasing flexural stresses for segments with longer spans, the flexural capacity decreased; however, it was higher than the punching capacity. In contrast, the steel-reinforced segment had an excessively lower flexural capacity than the punching capacity. These findings are consistent with the experimental observations regarding the failure mode of the tested specimens.

Table 4. 5 Experimental-to-predicted punching and flexural capacity.

<table>
<thead>
<tr>
<th></th>
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<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$G_{(0.46)}$-2.1</td>
<td>140 2.05 269 1.07 145 1.98</td>
<td>239 1.20 270 1.06 146 1.97</td>
<td>608 0.47 2.1</td>
<td>700 0.41</td>
<td>743 0.48 845</td>
<td>42</td>
<td>608 0.53 695</td>
<td>0.47</td>
</tr>
<tr>
<td>$G_{(0.86)}$-2.1</td>
<td>180 1.97 315 1.13 186 1.91</td>
<td>297 1.20 317 1.12 187 1.91</td>
<td>743 0.48 0.97</td>
<td>845 0.42</td>
<td>608 0.53 695</td>
<td>0.47</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$G_{(0.46)}$-2.1-SR</td>
<td>220 1.48 347 0.96 225 1.45</td>
<td>318 1.03 348 0.94 226 1.44</td>
<td>608 0.53 695</td>
<td>0.47</td>
<td>608 0.53 695</td>
<td>0.47</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$G_{(0.86)}$-3.1</td>
<td>143 1.63 275 0.85 147 1.58</td>
<td>239 0.97 277 0.84 149 1.57</td>
<td>331 0.70 395</td>
<td>0.59</td>
<td>331 0.70 395</td>
<td>0.59</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$G_{(0.46)}$-3.1</td>
<td>188 1.49 332 0.84 194 1.44</td>
<td>297 0.94 334 0.84 195 1.43</td>
<td>431 0.65 499</td>
<td>0.56</td>
<td>431 0.65 499</td>
<td>0.56</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: S.D. is standard deviation; COV is coefficient of variation; 1 kN = 0.225 kips.
1 The predicted punching capacity was the sum of concrete and shear stirrup contributions (AASHTO 2018) for specimen $G_{(0.46)}$-2.1-SR.
2 The predicted punching and flexural capacities of segment $S_{(0.47)}$-2.1 were calculated according to ACI 318-19 and CSA-A23.3-19.
4.7 Conclusions

This paper reports on experimental and theoretical investigations into the punching-shear behavior of PCTL segments to study the effects of reinforcement type and ratio, segment length, and shear stirrups. Based on the test results presented and discussed herein, the following conclusions can be drawn:

1. The short-span GFRP-reinforced PCTL segments exhibited punching–shear failure, while a mixed flexural–punching mode was observed in the segments with a longer span. The shear stirrups in specimen G\((0.46)\)-2.1-SR eliminated and converted the brittle failure to a more ductile mode. The steel-reinforced segment failed in flexure due to yielding of the steel.

2. Using GFRP reinforcement at the same ratio as the steel reinforcement in the PCTL segments satisfied code requirements in terms of crack opening at service load. Furthermore, both segments G\((0.46)\)-2.1 and S\((0.47)\)-2.1 failed at a load of 288 kN (64.6 kips) and 321 kN (72.1 kips), respectively, in a range of only an 11% difference.

3. The higher GFRP reinforcement ratio yielded higher punching capacity, lower deflection, lower strain, and narrower crack openings. Increasing the reinforcement ratio from 0.46% to 0.86% in G\((0.86)\)-2.1 and G\((0.86)\)-3.1 increased the normalized punching capacity by 30% and 22%, respectively.

4. The segment length significantly affected the punching–shear behavior. The flexural stresses for segments increased for the longer span at the same applied load. Wider and more cracks appeared, resulting in an increase in deflection.

5. A small amount of GFRP shear stirrups improved the confinement of the punching region and contributed to an enhancement in normalized punching-shear resistance by 14% and segment deformation by 7%.

6. The punching-shear capacity of PCTL segments reinforced with GFRP bars can be predicted with current FRP design provisions. CAN/CSA S806-12 yielded accurate predictions with an average \(V_{exp}/V_{pred}\) of 0.98 ± 0.13 and a corresponding COV of 13%. 
Moreover, JSCE (1997) and BSI (1997) yielded good predictions, while the predictions of ACI 440.11-22, fib TG-9.3 (2007), and AASHTO (2018) were excessively conservative.

7. For large-span specimens, CAN/CSA S806-12, JSCE (1997), and BSI (1997) overestimated predictions due to neglecting the effect of span length. Adding the contribution of shear stirrups in specimen $G_{(0.46)}$-2.1-SR enhanced the predictions of these provisions. That notwithstanding, further study is warranted.

Lastly, this research confirms the efficiency of employing GFRP as internal reinforcement for PCTLs when the durability requirements and applicability of capacity are the dominant criteria. Moreover, the use of GFRP bars and shear stirrups are considered promising solutions for PCTL segments under punching-shear loads.
CHAPTER 5
EFFECT OF TEST PARAMETERS ON THE PUNCHING-SHEAR STRENGTH OF PRECAST TUNNEL SEGMENTS REINFORCED WITH GFRP BARS

Foreword

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Journal and Status:


Reference:


Note:

The manuscript had been slightly adjusted from the original paper by remembering the figures and tables to include the chapter number. In addition, the reference list has been moved to the appropriate section in the thesis as indicated in the table of contents.
Abstract

While assessing the load cases is crucial for designing the precast concrete tunnel lining (PCTL) segments, none of the tunnel-specific standards or codes explicitly consider punching-shear loads. This study aimed at determining the structural performance of full-scale precast concrete tunnel segments reinforced with glass fiber-reinforced polymer (GFRP) bars. The segments were constructed with a rhomboidal shape measuring 2100 x 1500 x 250 mm and subjected to point loading on their extrados surfaces until failure. Such loading simulates the geotechnical exposure conditions surrounding tunnels, such as rock expansion. The effects of concrete strength and GFRP flexural reinforcement ratio on the punching behavior were analyzed and are discussed herein. The load–deflection response, failure mode, punching strength, cracking behavior, reinforcement and concrete strain, deformability, and energy absorption were all evaluated in the experiments. The results reveal that increasing both the flexural reinforcement ratio and concrete strength enhanced the punching response and decreased crack width. In addition, using HSC in tunnel segments yielded higher pre-cracking stiffness, punching-shear capacity, and energy absorption. A theoretical investigation involving a comparison between predictions of the FRP-reinforced concrete slab design provisions and experimental results was performed to assess the applicability of the current punching equations for PCTL segments. The equations in the FRP-reinforced concrete codes predicted the punching capacities conservatively, while the researchers’ proposed equations overestimated. Moreover, CAN/CSA S806-12 showed the most accurate prediction of the punching capacity even if the concrete compressive strength exceeded 60 MPa.

Keywords: punching; precast; normal- and high-strength concrete; tunnel segments; glass fiber-reinforced polymer bars; reinforcement ratio; theoretical investigation.
5.1 Introduction

Substituting the conventional in situ lining technique with precast concrete tunnel lining (PCTL) segments in infrastructure projects maintains time and emphasizes workers’ safety with superior quality (Dean et al. 2006). In precast plants, PCTL segments are fabricated in repetitive steps of batching and casting concrete to enhance fabrication control and reduce wastage. Tunnel segments are exposed to provisional and permanent loads that affect tunnel life cycle. ACI 544.7R-16 and ACI 533.5R-20 divide these loading conditions into three main stages: production and transient loads, loads for the construction stage, and service loads. The service stages of PCTLs include long-term loads imposed on the lining by earth pressure, groundwater, and surcharges. Other loads should be considered in the service stage based on the ground condition, tunnel function, and any special circumstances such as punching loading. The expansion of rocks or other geotechnical conditions above or underneath the tunnel segments can induce punching. In addition, ACI 533.5R-20 considers that the TBM backup load induced by the self-weight of the backup equipment train behind the TBM shield applies a concentrated variable load on PCTL segments. So far, the literature contains a paucity of information about the strength and response of PCTL segments subjected to punching shear.

Abbas et al. (2014) investigated experimentally the punching behavior of full-scale PCTL segments reinforced with conventional steel bars and steel fibers. The corrosion of steel reinforcement in railway and roadway tunnels due to the attack of chloride ions and stray currents affects PCTL durability. The corrosion of embedded reinforcement induces internal pressures, causing the concrete cover to spall and accelerating failure (Abbas et al. 2014). The deterioration and related catastrophic failure of segments can jeopardize the structural integrity of tunnels, requiring costly repairs and rehabilitation. Adopting glass fiber-reinforced polymer (GFRP) bars in PCTL segments as an alternative to corroding steel reinforcement is considered a realistic and cost-effective solution, particularly in harsh environmental conditions surrounding tunnels. GFRP materials present high tensile strength, high strength-to-weight ratio, long service life, and neutrality to electrical and magnetic disturbances, while improving the life-cycle cost efficiency (ACI Committee 440; Benmokrane et al. 2021). These nonmetallic properties can reduce the concrete cover and create dielectric joints in PCTLs. Consequently, segments reinforced with GFRP bars avoid concrete crushing during handling and guarantee an effective remedy to
corrosion (Caratelli et al. 2016). Using high-strength concrete (HSC)—with compressive strengths exceeding 50 MPa—can improve the strength, stiffness, and durability of tunnel segments (El-Sayed et al. 2006). HSC segments have higher tensile strength and moduli of elasticity than their NSC counterparts (Hassan et al. 2013). Incorporating GFRP reinforcement in high-strength concrete in PCTLs is a valuable solution for taking advantage of GFRP bars’ high stress and strain properties as well as their contribution to post-cracking stiffness. To date, no study has investigated the punching behavior of GFRP-reinforced PCTL segments fabricated with HSC.

Few studies, however, have been conducted recently to investigate the possibility of using GFRP bars and HSC in PCTL segments (Caratelli et al. 2017; Spagnuolo et al. 2017, 2018; Meda et al. 2019; Hosseini et al. 2022a, 2022b). All the tests were performed on full-scale PCTLs under bending and thrust loads to assess their structural behavior. The experimental results revealed the feasibility and efficiency of such materials in tunnel applications as well as the fact that they satisfied service requirements. The curved geometry of PCTLs results in the applied load and stresses around the loaded region differing from that of flat or slab elements (Hosseini et al. 2022a). The punching-shear behavior of flat slabs/plates can, however, be used to understand the response of tunnel segments. GFRP-reinforced slabs had a lower punching capacity than their counterparts reinforced with steel bars due to the lower modulus of GFRP bars (Eladawy et al. 2020; Hassan et al. 2017). The punching strength of the specimens reinforced with GFRP bars, however, were comparable to the yielding level of the steel bars (Salama et al. 2021). The tests showed that HSC slabs had higher ultimate punching capacity and uncracked stiffness than those fabricated with NSC and experienced more brittle failure (Gouda and El-Salakawy 2015). For tested slabs with a concrete strength greater than 60 MPa, Hassan et al. (2013) demonstrated that the Canadian provisions for prediction equations for punching shear were applicable. Mostafa and El-Salakawy (2018) found that increasing the flexural reinforcement ratio in GFRP-reinforced HSC slabs increased their punching capacity, adverse effects on ductility, however, were observed. The high reinforcement ratio resulted in higher cracked stiffness and reduced concrete strains. The effect of the design parameters on the punching behavior of the GFRP-reinforced slabs was similar to that of their counterparts reinforced with steel bars Inácio et al. 2020; Santos et al. 2022). The study reported on herein evaluated experimentally and theoretically the effects of the flexural reinforcement ratio and concrete strength on GFRP-reinforced PCTL segments under punching loads.
This study is a part of an ongoing extensive research project conducted on precast concrete tunnel segments at the University of Sherbrooke. Full-scale GFRP-reinforced PCTL segments were tested under different loading conditions—punching tests in addition to static flexural loading (Hosseini et al. 2022a, 2022b); quasi-static cyclic flexural loading; and settlement—to investigate the influence of different design parameters. The research program developed more efficient design and construction approaches for using curvilinear GFRP bars and stirrups in PCTLs during the production, transport, construction, and service stages. This paper provides the first experimental data on the punching behavior of GFRP-reinforced HSC PCTL segments. Moreover, the test results were used to evaluate the applicability of FRP punching equations in codes, guidelines, and the literature for segmental tunnels.

5.2 Research Objectives

North American guidelines do not address the design of PCTL segments reinforced internally with FRP bars due to the lack of information in the literature. In addition, no experimental studies on the punching strength of GFRP-reinforced HSC PCTL segments have been reported. This study aims at understanding the structural behavior of such elements under punching-shear loads. The main objectives are identified and summarized as follows:

- Assess the effect of concrete strength (normal and high-strength concrete) on the behavior of GFRP-reinforced PCTL segments subjected to punching-shear loads.
- Investigate the influence of the longitudinal GFRP reinforcement ratio on the punching strength and behavior of PCTLs.
- Examine the accuracy of available design equations for predicting the punching shear capacities of GFRP-reinforced PCTL segments with a varying range of reinforcement ratios and concrete strength.

The experimental results in this study enrich the state of the art with evidence about the structural behavior of GFRP-reinforced HSC PCTL segments under punching-shear loads. Moreover, the theoretical predictions will support the work of the North American technical committees for using GFRP reinforcement in tunnels and the development of standards and design provisions. Lastly,
the findings pave the way for integrating curvilinear GFRP bars and high-strength concrete in infrastructure applications.

5.3 Experimental Investigation

5.3.1 Test Parameters and Segment Design

In this study, four full-scale PCTL segments entirely reinforced with GFRP bars and ties were constructed and tested up to failure under punching-shear loads. The rhomboidal-shaped PCTL segment considered herein is one of seven different shapes of segments comprising full parallel rings with an internal and external diameter of 6500 mm and 7000 mm, respectively. The test specimens measured 2100 mm in length, 1500 mm in width, and 250 mm in thickness. The clear concrete cover was kept constant at 40 mm for all segments as provided for in ACI 533.5R-20. The test matrix was designed to assess the influence of the longitudinal reinforcement ratio and concrete strength (normal versus high strength). Each test parameter involved two pairs of PCTL segments reinforced with GFRP bars. Segments were labeled with the letter G, indicating the GFRP bars and stirrups. The subscripts (x%) stand for the flexural reinforcement ratio for the top and bottom mat, followed by the specimen arc length in meters, while the letter H indicates high-strength concrete. Table 5.1 presents the test matrix and reinforcement details of the tested specimens. Two different amounts of longitudinal reinforcement were used to reinforce the segments (7 and 13 No. 5 GFRP with reinforcement ratios of 0.46% and 0.86%, respectively) for the top and bottom meshes. The concrete-crushing failure mode is marginally more desirable for flexural members reinforced with FRP bars so that they exhibit some plastic behavior before failure (ACI 440.11-22). To ensure a desirable failure mode, the segments were designed to have a flexural reinforcement ratio greater than the balanced reinforcement ratio. For each reinforcement ratio, one specimen was fabricated using a concrete strength of 40 MPa (NSC), while the other was cast with a concrete strength of 70 MPa (HSC). All segments were reinforced with No. 4 ties in the transversal direction at a spacing of 200 mm and No. 5 U-shaped anchorage bars at the ends of the longitudinal bars. Figure 5.1 presents the typical dimensions and reinforcement details of the testing specimens.
Table 5. Details of the test specimens.

<table>
<thead>
<tr>
<th>Specimen ID*</th>
<th>Concrete Type</th>
<th>Arc Length</th>
<th>Longitudinal Reinforcement</th>
<th>$\rho$ (%)</th>
<th>$\rho_b$ (%)</th>
<th>Transverse Reinforcement</th>
<th>$f'_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$G_{(0.46)-2.1}$</td>
<td>NSC</td>
<td>2100</td>
<td>7 No. 5</td>
<td>0.46</td>
<td>0.33</td>
<td>No. 4 @200 mm</td>
<td>47.8</td>
</tr>
<tr>
<td>$G_{(0.86)-2.1}$</td>
<td></td>
<td>2100</td>
<td>13 No. 5</td>
<td>0.86</td>
<td>0.31</td>
<td>No. 4 @200 mm</td>
<td>41.6</td>
</tr>
<tr>
<td>$G_{(0.46)-2.1-H}$</td>
<td>HSC</td>
<td>2100</td>
<td>7 No. 5</td>
<td>0.46</td>
<td>0.45</td>
<td>No. 4 @200 mm</td>
<td>70.7</td>
</tr>
<tr>
<td>$G_{(0.86)-2.1-H}$</td>
<td></td>
<td>2100</td>
<td>13 No. 5</td>
<td>0.86</td>
<td>0.42</td>
<td>No. 4 @200 mm</td>
<td>66.9</td>
</tr>
</tbody>
</table>

* The letter H stands for high-strength concrete.

**Figure 5.** Typical dimensions and reinforcement details of the segments.

**5.3.2 Materials and Specimen Production**

Figure 5.2(a) shows the newly developed curvilinear (Pultrall 2019) No. 5 (15 mm) GFRP bars used to reinforce the PCTL segments in the top and bottom longitudinal directions. Number 4 (13...
mm) closed GFRP ties served as transverse reinforcement. Number 5 (15 mm) U-shaped GFRP bars were used to ensure proper anchorage for the longitudinal reinforcement. The tensile strength and elastic modulus of the GFRP bars and ties were determined according to ASTM D7205-21, as summarized in Table 5.2. All the GFRP reinforcement had sand-coated surfaces to improve the bonding between the bars and surrounding concrete. The sand-coated GFRP bars and ties were manufactured by pultruding 78% (by weight) boron-free glass fibers impregnated with thermoset vinyl-ester resin, additives, and fillers (ASTM D3171-15).

Table 5.2 Mechanical properties of the GFRP reinforcement.

<table>
<thead>
<tr>
<th>Reinforcement Type</th>
<th>Reinforcement Shape</th>
<th>Bar Size</th>
<th>$d_b$ (mm)</th>
<th>$A_f$ (mm$^2$)</th>
<th>$E_f$ (GPa)</th>
<th>$f_{fu}$ (MPa)</th>
<th>$\varepsilon_{fu}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GFRP</td>
<td>Curvilinear bars</td>
<td>#5</td>
<td>15.0</td>
<td>199</td>
<td>55.1 ± 1.25</td>
<td>1115 ± 60</td>
<td>2.0 ± 0.1</td>
</tr>
<tr>
<td></td>
<td>Closed stirrups</td>
<td>#4</td>
<td>13.0</td>
<td>129</td>
<td>55.6 ± 1.6</td>
<td>1248 ± 74</td>
<td>2.2 ± 0.1</td>
</tr>
<tr>
<td></td>
<td>U-shaped closing bars</td>
<td>#5</td>
<td>15.0</td>
<td>199</td>
<td>53.5 ± 1.1</td>
<td>1283 ± 42</td>
<td>2.4 ± 0.1</td>
</tr>
</tbody>
</table>

Two configurations of cages were assembled at the University of Sherbrooke, as shown in Fig. 5.2(b). The segments were shipped to a precast plant (Sym-Tech) in Saint-Hyacinthe, QC (Canada) for instrumentation and casting. The target 28-day compressive strength was 40 MPa for the NSC segments and 70 MPa for the HSC segments. The concrete compressive strength for each specimen was determined based on the average test results of six 100 × 200 mm concrete cylinders tested on the same day of testing. The actual compressive strengths for specimens $G_{(0.46)}$-2.1, $G_{(0.86)}$-2.1, $G_{(0.46)}$-2.1-H, and $G_{(0.86)}$-2.1-H were 47.8, 41.6, 70.7, and 66.9 MPa, respectively. Curved wooden formwork was designed and constructed to fabricate the PCTL segments. An overhead crane bucket was used to pour the concrete into the specimen molds. The segments were then compacted with an electrical vibrator to ensure adequate consistency and then leveled manually. A plastic sheet was placed over the entire mold to cure the segments and eliminate steam and moisture losses. The forms were stripped after 24 hours and the segments then stored in a storage area. Figure 5.2(c) illustrates the fabrication stages of the PCTL segments.
Figure 5. Overview of (a) GFRP reinforcement, (b) typical assembled GFRP cages for the specimens, and (c) fabrication and preparation of the PCTL segments.
5.3.3 Test Setup and Instrumentation

All the PCTL segments were tested on cylindrical supports for easy movement and rotation over a 1400 mm center-to-center span, as shown in Fig. 5.3. The concentrated load was applied to the extrados surface of the segment with an MTS 1000 kN actuator at a displacement control rate of 0.5 mm/min. To distribute the point load radially, a circular steel plate was attached to the actuator with a diameter and thickness of 150 mm and 40 mm, respectively. The plate dimensions were determined to ensure such punching-shear failure would occur (ACI 440.11-22; CSA S806-12). During the test, the first crack was monitored visually and measured with a gauge card. Subsequently, the crack propagation was marked and measured with three high-accuracy LVDTs installed at the first crack locations. Each segment was equipped with two instrumented middle bars (curvilinear and closed ties) in the bottom mesh (tension side). Five electrical strain gauges were located at 0, 100, 200, 300, and 400 mm from the loading plate face to obtain the strain distribution in the longitudinal and transverse directions. At the plate face, two strain gauges were attached to the top of the reinforcement (compression side) in each orthogonal direction. Strain in the concrete was measured with five gauges mounted on the concrete surface before testing (Fig. 5.1). The deflection at the mid- and quarter-span was measured with five linear potentiometers (LPOTs), both longitudinally and transversely. The actuator, strain gauges, LVDTs, and LPOTs were connected to a data-acquisition system to record all measurements.
5.4 Test Results and Discussion

5.4.1 Load-Deflection Response

Figure 5.4 plots the applied load versus the maximum deflection at the center of specimens. The load–deflection curve of all the tested PCTL segments showed typical bilinear behavior until a sudden drop occurred due to punching-shear failure. The first portion implies similar stiff pre-cracking behavior, while the second portion shows a deviation in post-cracking stiffness. The slopes of these portions were used to calculate pre-cracking stiffness ($k_{pre}$), post-cracking stiffness ($k_{post}$), and stiffness degradation ($k_{post}/k_{pre}$), as provided in Table 5.3.

All the segments behaved linearly up to the appearance of the initial flexural crack. Using HSC, however, significantly improved the uncracked response for the tested specimens compared to the reinforcement ratio. This could be attributed to the negligible effect of the reinforcement ratio on
the gross moment of inertia in the segments’ cross section (Elgabbas et al. 2016; Mousa et al. 2019). Concrete compressive strength was considered the main parameter in resisting flexural stresses at the uncracked stage. Thus, the initial stiffness of the segments was significantly affected by its increase. For each pair of segments with the same reinforcement ratio, the HSC specimen had a higher pre-cracking stiffness than its NSC counterpart. The pre-cracking stiffness of G(0.86)-2.1-H was higher than that of G(0.86)-2.1 by 83%.

After cracking occurred, the load–deflection curve of all the segments increased nearly linearly due to the linear elastic characteristics of the GFRP bars. As the point load increased, the number and width of the flexural cracks increased, with a detectable reduction in the post-cracking stiffness up to failure. At this stage, the section properties changed from gross to effective, which, in turn, yielded larger deflection. Increasing the reinforcement ratio in the pairs of specimens (G(0.46)-2.1, G(0.86)-2.1) and (G(0.46)-2.1-H, G(0.86)-2.1-H) from 0.46% to 0.86% reduced ultimate deflections by 17% and 19%, respectively. Conversely, the ultimate deflection of the HSC segment (G(0.46)-2.1-H) was 9% higher than that of the NSC segment (G(0.46)-2.1). Similarly, G(0.86)-2.1-H had 10% higher ultimate deflection than G(0.86)-2.1. This confirms that using the HSC in the GFRP-reinforced PCTL segments resulted in higher deflections at failure, which increased the deformability of the test specimens (Hassan et al. 2013). The post-cracking stiffness depended more on the reinforcement ratio than the concrete strength. Specimens G(0.86)-2.1 and G(0.86)-2.1-H had post-cracking stiffness 57% and 55% higher than to their counterparts. Ultimately, the post-peak behavior revealed the incidence of punching-shear failure for all segments. A sudden decay in the load–deflection relationships was observed, accompanied by the steel plate sinking into the extrados surface. The deflection continued to increase, however, while the point load decreased. The segments behaved nonlinearly beyond failure due to widespread transversal cracks at the top surface around the loading plate. The ratio between the post and pre-cracking stiffness indicated stiffness degradation. Table 5.3 shows that increasing the reinforcement ratio decreased the stiffness degradation. Conversely, HSC segments G(0.46)-2.1-H and G(0.86)-2.1-H had higher degradation than their NSC counterparts.
Figure 5.4 Load-deflection responses.

Table 5.3 Summary of test results.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Stiffness</th>
<th>( V_{cr} ), kN</th>
<th>( A_{cr} ), mm</th>
<th>( V_{ur} ), kN</th>
<th>( A_{ur} ), mm</th>
<th>( R_{cone} ), mm</th>
<th>( v_{cr} )^2/2</th>
<th>( f_{c}' )</th>
<th>( \varepsilon_{max} ), µs at plate face</th>
<th>( \varepsilon_{cmax} ), µs</th>
<th>( W_{max} ), mm</th>
<th>Def. Factor, ( J_d )</th>
<th>Energy Absorp., ( U_d ), kN.m</th>
<th>Theoretical Capacity, ( V_{pred} ), kN</th>
<th>V_{punch}</th>
<th>V_{flex}</th>
<th>ACI 806</th>
<th>ACI S806</th>
<th>ACI S806</th>
</tr>
</thead>
<tbody>
<tr>
<td>G_{(0.46)}-2.1</td>
<td>75.84</td>
<td>10.53</td>
<td>0.14</td>
<td>94.8</td>
<td>1.25</td>
<td>288</td>
<td>19.55</td>
<td>1.1d</td>
<td>0.37</td>
<td>8785</td>
<td>3180</td>
<td>-2385</td>
<td>2.05</td>
<td>7.19</td>
<td>8.13</td>
<td>140</td>
<td>269</td>
<td>608</td>
<td>700</td>
</tr>
<tr>
<td>G_{(0.86)}-2.1</td>
<td>43.34</td>
<td>16.56</td>
<td>0.38</td>
<td>141.3</td>
<td>3.26</td>
<td>356</td>
<td>16.21</td>
<td>2.1d</td>
<td>0.48</td>
<td>6460</td>
<td>2850</td>
<td>-2990</td>
<td>1.55</td>
<td>4.01</td>
<td>11.17</td>
<td>180</td>
<td>315</td>
<td>743</td>
<td>845</td>
</tr>
<tr>
<td>G_{(0.46)}-2.1-H</td>
<td>84.61</td>
<td>11.71</td>
<td>0.13</td>
<td>141.8</td>
<td>1.69</td>
<td>299</td>
<td>21.37</td>
<td>1.5d</td>
<td>0.34</td>
<td>11450</td>
<td>5020</td>
<td>-2605</td>
<td>1.81</td>
<td>8.02</td>
<td>9.64</td>
<td>156</td>
<td>313</td>
<td>727</td>
<td>797</td>
</tr>
<tr>
<td>G_{(0.86)}-2.1-H</td>
<td>79.43</td>
<td>18.21</td>
<td>0.23</td>
<td>154.9</td>
<td>1.95</td>
<td>388</td>
<td>17.89</td>
<td>2.2d</td>
<td>0.44</td>
<td>8505</td>
<td>3590</td>
<td>-2998</td>
<td>1.12</td>
<td>6.07</td>
<td>12.03</td>
<td>205</td>
<td>384</td>
<td>924</td>
<td>1044</td>
</tr>
</tbody>
</table>

1 Indicates stiffness degradation.
2 Identifies the distance from plate the face to the punching-failure surface.
3 Represents the normalized ultimate punching-shear stress at \( d/2 \) from the plate face.
4 Denotes theoretical punching and flexural capacities based on FRP design provisions.

Figure 5.5 plots the deformation of segments along the longitudinal direction, measured by LPOTs (Fig. 5.1) at different load levels. Note that similar deflection profiles were recorded for all the specimens during the testing history. A straight line was observed for all profiles up to 25% of the ultimate punching capacity. The initial deformation was constant regardless of the loaded area and test parameters. Then, cracks formed and propagated as the load increased. This, in turn, increased deflections and concentrated the profile trough directly under the point load.
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5.4.2 Cracking Behavior

During the test, the first crack formed on the intrados surface of all the specimens (tension side) was a flexural crack beneath the point load and oriented transversely parallel to the steel supports. The flexural cracks initiated at loads of 94.8, 141.3, 141.8, and 154.9 kN (at about 33%, 39%, 47%, and 40% of the ultimate loads) in specimens G\(_{(0.46)}\)-2.1, G\(_{(0.86)}\)-2.1, G\(_{(0.46)}\)-2.1-H, and G\(_{(0.86)}\)-2.1-H, respectively. The second type of crack was radial, which started sequentially from the free edges and propagated to interfere with the transversal cracks. Tangential cracks at higher loads were observed, connecting the radial cracks around the loaded region to form the typical punching cone. All segments exhibited detectable fan-shaped cracks, which comprised radial and circumferential types. With further loading, the number and width of these cracks increased in the plate vicinity. In addition, new longitudinal and diagonal cracks on intrados surfaces appeared near steel supports and were intercepted by the other cracks. Inclined shear cracks also developed at the
lateral edges and extended from the tension toward the compression side until failure occurred. Figure 5.6 shows similar patterns of crack propagation for all the tested PCTL segments regardless of the reinforcement ratio and concrete compressive strength. Specimens G\(_{(0.46)}\)-2.1 and G\(_{(0.46)}\)-2.1-H (with the low reinforcement ratio) experienced more flexural cracks around the steel plate. Specimens G\(_{(0.86)}\)-2.1 and G\(_{(0.86)}\)-2.1-H (with the higher reinforcement ratio) cracked at 49% and 10% higher loads than specimens G\(_{(0.46)}\)-2.1 and G\(_{(0.46)}\)-2.1-H, respectively. Moreover, the HSC segments evidenced the same higher cracking load than their NSC counterparts. Increasing concrete compressive strength considerably influenced the first cracking load due to higher concrete tensile strength. Moreover, narrower and fewer cracks were observed in the HSC specimens.

![Figure 5.6 Schematic of the crack-pattern propagation of (a) G\(_{(0.46)}\)-2.1, (b) G\(_{(0.86)}\)-2.1, (c) G\(_{(0.46)}\)-2.1-H, and (d) G\(_{(0.86)}\)-2.1-H at different load stages.](image)
5.4.3 Failure Mode

Regardless of the design parameters, the idealized failure mechanism for all the PCTL segments was observed as a punching-shear mode characterized by an instant drop in load capacity. In addition, the steel plate penetrated the concrete surface and a classical punching cone appeared. While the same failure occurred in the steel-reinforced PCTL segment (Abbas et al. 2014), the lower steel reinforcement ratio yielded flexural failure due to the yielding of the steel bars before punching failure (Ghali and Gayed 2018). The failure surface was composed of intercepting inclined shear cracks by wide evident cracks that appeared on the intrados surface around the loading plate. Table 5.3 presents the theoretical two-way punching ($V_{\text{punch}}$) and flexural ($V_{\text{flex.}}$) capacities of the tested segments according to ACI 440.11-22 and CSA S806-12. The experimental results were higher than the predicted punching-shear capacity and significantly lower than the flexural capacity for all the specimens. This finding is consistent with the experimental observations, which, in turn, confirm the occurrence of the punching-failure mode. It was defined for each specimen by measuring the radius ($R_{\text{cone}}$) from the plate face to the failure-envelope location. Figure 5.7 shows the marked punching-failure surfaces for the tested specimens. The average limit to the radius $R_{\text{cone}}$ was about $1.1d$, $2.1d$, $1.5d$, and $2.2d$ for segments $G_{(0.46)-2.1}$, $G_{(0.86)-2.1}$, $G_{(0.46)-2.1-H}$, and $G_{(0.86)-2.1-H}$, respectively (where $d$ is the average effective depth of segments). Indeed, the flexural reinforcement ratio considerably affected the radii of surface failure more than the concrete compressive strength. The radius ($R_{\text{cone}}$) of $G_{(0.86)-2.1}$ and $G_{(0.86)-2.1-H}$ were 1.9 and 1.5 times higher than that of $G_{(0.46)-2.1}$ and $G_{(0.46)-2.1-H}$, respectively. This is consistent with the findings of Hussein and El-Salakawy (2018), who reported that the flexural reinforcement ratio was proportional to the flatter angle of the inclined shear cracks. As a result, increasing the amount of longitudinal reinforcement increased the failure surfaces. This could be explained by the higher reinforcement ratio improving constraint in the slab’s plan for the failure mechanism involving vertical displacement at the shear failure surface (Dulude, et al. 2013). The available standards (ACI 440.11-22; BS8110-97; EN 1992-1-1:2004) for design punching shear compute the perimeter of the failure surface at $0.5d$, $1.5d$, or $2d$; the $R_{\text{cone}}$ of specimens $G_{(0.86)-2.1}$ and $G_{(0.86)-2.1-H}$ were, however, exceeded. Using the HSC yielded a slight increase in the punching-shear cone. Furthermore, significant spalling of the concrete cover accompanied the failure surface on the tension side due to the brittleness of the HSC.
Figure 5.7 Punching-shear failure surface of the tested specimens.

5.4.4 Punching-Shear Capacity

Figure 5.8(a) depicts the normalized punching-shear stress calculated at \( d/2 \) from the plate face versus the flexural reinforcement ratio for the tested segments. To minimize the influence of variations in concrete strength, the punching-shear stresses were normalized to their cubic root (Eladawy et al. 2020). The figure implies that the normalized punching stress and the flexural reinforcement ratio are not linearly proportional. Normalized stress is proportional to the power of 0.41 with the reinforcement ratio and it is close to the 1/3 indicated in the design equations for punching (CSA S806-12; JSCE 1997). Increasing the reinforcement ratio from 0.46% to 0.86% increased the normalized punching stress by 30% for \( G_{0.46}-2.1 \) and 29% for \( G_{0.86}-2.1-H \). The increase in segments with a high reinforcement ratio was ascribed to their exhibiting an increase in dowel action. This, in turn, reduced the flexural stiffness losses after cracking. In addition, the uncracked concrete depth (compression region) and the aggregate interlock increased, which
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decreased crack number and width. Consequently, the increased longitudinal reinforcement ratio enhanced the concrete’s contribution to specimens and yielded higher punching capacity.

Figure 5.8 (b) shows the experimental punching-shear capacities of the tested specimens against the concrete compressive strengths. The results indicate that the punching capacity increased slightly linearly with the increase of concrete strength for each reinforcement ratio with a linear average trend. Increasing the concrete strength from 47.8 to 70.7 MPa increased the ultimate punching capacity of specimen G(0.46)-2.1-H by 4%. The corresponding value was 9% when the concrete strength increased from 41.6 to 66.9 MPa for G(0.86)-2.1-H. Using HSC in the segments enhanced the compressive block’s contribution above the neutral axis after crack initiation and increased the punching strength (Salama et al. 2021). Conversely, it decreased the normalized punching-shear stress due to aggregate interlock. The smoother crack surfaces in the HSC segments were induced by the cracks passing through the aggregate, reducing the compression concrete area (El-Sayed et al. 2006). The normalized punching stress for both HSC segments was 8% lower than that of G(0.46)-2.1 and G(0.86)-2.1. Table 5.3 reports the ultimate punching-shear capacity and the corresponding normalized punching stress for all segments.

![Figure 5.8](image1.png)

(a) Relationship between reinforcement ratio and normalized punching shear stress.

(b) Effect of concrete compressive strength on punching capacity.

5.4.5 Reinforcement Strain

Figure 5.9 shows the load–strain behavior in the orthogonal directions for each longitudinal bar and transverse closed tie. In the figure, the strain of these middle bottom reinforcements was
gauged at 75 mm from the centerline of the steel plate. Prior to the onset of flexural cracking, a minimal strain was recorded for the reinforcement of all the segments in both directions. After the cracking stage, the longitudinal bars exhibited a linear response until failure. The load–strain curve of transverse reinforcement then maintained behavior similar to its load–deflection response. In this context, the load values dropped suddenly, accompanied by an increase in the strain reading even after punching failure. This might be attributed to the segments being supported only on two sides (one way) in the longitudinal direction and the loading manner being mainly dependent on the longitudinal bars. Once failure occurred, the point load dropped and the strain gauges stopped registering. For the longitudinal bars, the maximum strains ($\varepsilon_{\text{max}}$) in the longitudinal bars were 8785, 6460, 11450, and 8505 $\mu\varepsilon$ for specimens G$_{(0.46)2.1}$, G$_{(0.86)2.1}$, G$_{(0.46)2.1}$-H, and G$_{(0.86)2.1}$-H, respectively, representing 43%, 32%, 57%, and 42% of the characteristic tensile strength. The $\varepsilon_{\text{max}}$ for the transverse ties was recorded at 3180, 2850, 5020, and 3590 $\mu\varepsilon$, respectively, representing 0.36, 0.44, 0.43, and 0.42 times those measured along the longitudinal bars, respectively. In both directions, the ultimate flexural strain was lower than the guaranteed tensile strength for all the specimens, which implies that the punching failures were not induced by the rupture of GFRP reinforcement. Furthermore, the compressive strains measured in the top reinforcement were very low in comparison to the strains in the tension ties and bars.

Increasing the reinforcement ratio significantly reduced the measured strain at all load levels for the tested segments. In the case of the longitudinal bars, the ultimate strain of specimens G$_{(0.86)2.1}$ and G$_{(0.86)2.1}$-H was 26% lower than that of their counterparts with a lower reinforcement ratio. Moreover, Figure 5.9 shows that segments G$_{(0.46)2.1}$ and G$_{(0.46)2.1}$-H experienced a considerable jump and a sharp increase in tensile strain, consistent with the appearance of the initial flexural cracks. This could be ascribed to the high ratio of FRP flexural bars in the segments, which, in turn, dissipated large amounts of energy during cracking, which is consistent with past studies (Elgabbas et al. 2016; El-Nemr et al. 2013). Similarly, the maximum strains in the transverse ties at peak load in segments G$_{(0.86)2.1}$ and G$_{(0.86)2.1}$-H were 10% and 28% lower, respectively, than G$_{(0.46)2.1}$ and G$_{(0.46)2.1}$-H. Moreover, the figure demonstrates the direct effects of employing HSC on the reinforcement strains, both longitudinal and transversal. At earlier loading stages, the strain in the longitudinal bars of the HSC segments was lower than that of G$_{(0.46)2.1}$ and G$_{(0.86)2.1}$. Nonetheless, specimens G$_{(0.46)2.1}$-H and G$_{(0.86)2.1}$-H exhibited higher strains in the longitudinal bars and transverse ties at high loading stages. In particular, the strain
increased suddenly in specimen $G_{(0.46)}$-2.1-H (with the highest concrete compressive strength) to transfer the stresses between the tensioned concrete and the longitudinal bars due to crack formation (Inácio et al. 2020).

![Graph showing load-strain relationships at the plate face of the longitudinal bars and transverse ties.](image)

Figure 5.9 Load-strain relationships at the plate face of the longitudinal bars and transverse ties. Figures 5.10 and 5.11 provide the profiles of the strain distribution along the mid-bottom span of the longitudinal bar and transverse tie, respectively. Electrical strain gauges located at 0, $d/2$, $d$, $1.5d$, and $2d$ from the plate face were used to obtain the profiles for each segment. The readings were typically high beneath the applied load and decreased proportionally toward the segment supports as for the same profiles in the literature (Eladawy et al. 2020; El-Gendy and El-Salakawy 2016). This confirms that the GFRP bars in all the segments transferred load without signs of bar slippage or bond degradation during the test. All the segments provided identical strain decay at higher loads in both directions, coinciding with the increased distance from the plate face. Nevertheless, the transverse strain distribution of specimen $G_{(0.86)}$-2.1-H exhibited a high strain value at 275 mm ($d$) from the center of the steel plate at different loading stages. This could imply the influence of gauge placement on crack location. Due to loading being distributed mainly in one direction, all the segments had higher strain in the longitudinal bars than the transverse ties.
Figure 5. 10 Strain distribution along the span of the longitudinal bars.
5.4.6 Concrete Strain

Figure 5.12(a) provides the recorded concrete strains at the plate face (gauge C1) on the extrados surfaces of the segments. The figure indicates that, before the initial flexural cracks appeared, the measured concrete strains were negligible in all the segments and ranged from -90 to 150 µε. There was an instant drop in load-carrying capacity accompanied by a slow and steady increase in the concrete strain after cracking occurred. Before punching failure occurred, the maximum measured strains ($\varepsilon_{c_{max}}$) were -2385, -2990, -2605, and 2998 µε for specimens $G_{(0.46)}-2.1$, $G_{(0.86)}-2.1$, $G_{(0.46)}-2.1-H$, and $G_{(0.86)}-2.1-H$, respectively. The theoretical crushing failure specified in ACI 440.11-22 was 3000 µε and 3500 µε as per CSA S806-12, which is higher than the recorded strains. Therefore, no signs of concrete crushing were detected during the test in any of the specimens. This implies that the final failure mode was punching shear rather than flexure. As expected from the
experimental evidence, increasing the reinforcement ratio in specimens $G_{(0.86)2.1}$ and $G_{(0.86)2.1-H}$ decreased the measured strains in the concrete at the same load levels. In addition, the HSC segments evidenced lower concrete strains than their NSC counterparts.

### 5.4.7 Crack Width

Figure 5.12(b) shows the relationship between the measured crack opening (on the intrados tension surface under the loading area) and the applied load for the tested specimens. This figure indicates that, in all the segments, early loading produced a few flexural cracks with small openings, which behaved linearly until punching failure occurred. The maximum recorded crack widths ($W_{max}$) were 2.05, 1.55, 1.81, and 1.12 mm for specimens $G_{(0.46)2.1}$, $G_{(0.86)2.1}$, $G_{(0.46)2.1-H}$, and $G_{(0.86)2.1-H}$, respectively. The flexural and shear crack widths were reduced by increasing the longitudinal reinforcement ratio. Thus, specimens $G_{(0.86)2.1}$ and $G_{(0.86)2.1-H}$ had narrower cracks than $G_{(0.46)2.1}$ and $G_{(0.46)2.1-H}$ at the same load levels. The higher concrete strength in segments $G_{(0.46)2.1-H}$ and $G_{(0.86)2.1-H}$ produced narrower cracks. This is ascribed to the higher concrete strength, which, in turn, resulted in higher tensile strength. The cracks in specimen $G_{(0.46)2.1-H}$ (highest compressive concrete strength) initiated at a strain of 991 $\mu$e. This, in turn, points to the strong bond between the concrete and GFRP bars, which delayed cracking. For the NSC segments, the crack opening at a longitudinal strain ($\varepsilon_L$) equal to 2000 $\mu$e was less than 0.5 mm and remarkably lower for the HSC specimens (ISIS 2007). Indeed, increasing the reinforcement ratio and using HSC for PCTL segments are considered appropriate solutions when the crack numbers and width are the dominant criteria during the initial handling and installation process.

### 5.4.8 Deformability and Energy Absorption

Preventing catastrophic failure in PCTL segments is deemed the prime design objective in all tunnel provisions. Therefore, the segments should be designed with adequate ductility to deform considerably before experiencing punching failure. Ductility in structural RC elements reinforced with steel provides warning signs of failure due to the elements being able to undergo plastic deformations. This is quantified by considering the steel yielding as a reference point. Unlike steel bars, FRP reinforcement does not exhibit a yielding plateau. The linear elastic behavior of FRP-reinforced concrete members allows them to undergo substantial deformations up to the ultimate due to FRP bars having a lower modulus of elasticity. Thus, deformability and energy absorption
have been proposed to assess the deformable behavior of FRP-reinforced concrete structures (Salama et al. 2019). The deformability factor (J) was employed to compute the deformation characteristics of GFRP-PCTL segments using the modification of Salama et al. (2019) on the CSA S6-19 approach, as shown in Eq. 5.1. \( P \) and \( \Delta \) refer to the applied load and deflection, whereas the subscripts \( u \) and \( s \) denote the ultimate and service limits, respectively. Several definitions are introduced to quantify the service limit states. These definitions determined the service state corresponding to a concrete compressive strength of 1000 \( \mu \varepsilon \) according to CSA S6-19 or a tensile strain in the flexural bars of 2000 \( \mu \varepsilon \) as recommended by ISIS Canada 2007. The service limit considered herein to calculate the deformability factor used the state corresponding to a tensile strain of 2000 \( \mu \varepsilon \) (El-Gendy and El-Salakawy 2016). The deformability factor (J) for all GFRP-reinforced segments is provided in Table 5.3.

\[
J_\Delta = \frac{P_u \Delta_u}{P_s \Delta_s}
\]  

(5.1)

Table 5.3 demonstrated that all specimens achieved appropriate deformability in comparison with the CSA S6-19 code limit of 4 (for rectangular sections). The deformability factors (J) were 7.19, 4.01, 8.02, and 6.07 for specimens \( G_{(0.46)}-2.1 \), \( G_{(0.86)}-2.1 \), \( G_{(0.46)}-2.1-H \), and \( G_{(0.86)}-2.1-H \), respectively. The higher J-factor contributes to increasing the safety of tunnel segments. This is attributed to the ample warning signs of impending failure for GFRP-reinforced concrete members. Specimens \( G_{(0.46)}-2.1 \) and \( G_{(0.46)}-2.1-H \) (with the low reinforcement ratio) exhibited J-factors 1.7 and 1.3 times higher than \( G_{(0.46)}-2.1 \) and \( G_{(0.46)}-2.1-H \), respectively.

Energy absorption is a considerable indicator for GFRP-reinforced PCTL segments to maintain punching capacity, particularly in earthquake-prone areas. It was calculated by integrating the area under the load–deflection curve (Fig. 5.4), as listed in Table 5.3. The reported values imply that the higher reinforcement ratio in specimen \( G_{(0.86)}-2.1 \) increased the energy absorption by 38% compared to \( G_{(0.46)}-2.1 \). Similarly, specimen \( G_{(0.86)}-2.1-H \) had an energy absorption 25% greater than that of \( G_{(0.46)}-2.1-H \). In contrast, HSC segments \( G_{(0.46)}-2.1-H \) and \( G_{(0.86)}-2.1-H \) recorded increases in energy absorption of 18% and 7% compared to their NSC counterparts, respectively. Figure 5.12(c) depicts the influence of concrete strength and longitudinal reinforcement ratio on the cumulative energy absorption of the PCTL segments.
Figure 5.12 Relationships of (a) load versus concrete strain, (b) load versus crack width, and (c) cumulative energy absorption versus deflection.

5.5 Theoretical Investigation

5.5.1 Punching-Shear Capacity of Segments

The punching-shear equations for PCTL segments reinforced internally with FRP bars are not included in the specific provisions for tunnels (ACI 544.7R-16; ACI 533.5R-20). In this study, the punching-capacity predictions of GFRP-reinforced PCTL segments were performed according to the equations available for FRP-reinforced concrete slabs (ACI 544.11-22; CSA S806-12; JSCE 1997; AASHTO 2018) to assess their accuracy. Moreover, the equations proposed by researchers based on their experimental or analytical studies (Hassan et al. 2017; Matthys and Taerwe 2000; El-Ghandour et al. 2000; Theodorakopoulos and Swamy 2008) were evaluated. The current FRP
provisions contain expressions that rely on the original steel equations with some modifications to the mechanical characteristics, as summarized in the following sections.

### 5.5.1.1 ACI 440.11 (2022)

The punching capacity of FRP-reinforced concrete slabs was provided based on the mechanistic beam shear model, which accounts for reinforcement stiffness with other modifications to involve the effect of shear transfer, as shown in Eq. (5.2)

$$V_c = \frac{4}{5} \sqrt{f'_{c}} b_{o,0.5d} (kd)$$

(5.2a)

$$k = \sqrt{2\rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f$$

(5.2b)

where $b_o$ is the perimeter of critical section computed at $d/2$ away from plate face with the same shape (mm); $f'_{c}$ is the specified compressive strength of the concrete (MPa); $k$ is the ratio of the neutral-axis depth to reinforcement depth; $d$ is the effective depth of the bottom flexural bars; $\rho_f$ is the fiber flexural reinforcement ratio; and $n_f$ is the ratio between the modulus of elasticity of the FRP bars and concrete.

### 5.5.1.2 CAN/CSA S806 (2012)

The factored shear–stress resistance of concrete due to punching is the smallest of Eqns. (5.3) to (5.5)

$$V_r = V_c = \left(1 + \frac{2}{\beta_c}\right) \left[0.028\lambda\phi_c(\rho_f f'_{c})^{1/3}\right] b_{o,0.5d} d$$

(5.3)

$$V_r = V_c = \left[\frac{\alpha_s d}{b_o} + 0.19\right] 0.147\lambda\phi_c(\rho_f f'_{c})^{1/3} b_{o,0.5d} d$$

(5.4)

$$V_r = V_c = 0.056\lambda\phi_c(\rho_f f'_{c})^{1/3} b_{o,0.5d} d$$

(5.5)

where the value of $f'_{c}$ used in these equations shall not exceed 60 MPa; $\beta_c$ is the rectangularity of the plate; and $\alpha_s$ is the coefficient equal to 4 for interior columns.
5.5.1.3 JSCE (1997)

When the eccentricity of the load is negligible, the punching-shear capacity of planner members is determined by Eq. (5.6)

\[
V_c = \beta_d \beta_p \beta_r f_{pcd} b_{o,0.5d} d / \gamma_h \tag{5.6a}
\]

\[
\beta_d = (1000 / d)^{1/4} \leq 1.5 \tag{5.6b}
\]

\[
\beta_p = (100 \rho_f E_f / E_s)^{1/3} \leq 1.5 \tag{5.6c}
\]

\[
\beta_r = 1 + 1 / (1 + 0.25u / d) \tag{5.6d}
\]

\[
f_{pcd} = 0.2 \sqrt{f_c} \leq 1.2 \text{ Mpa} \tag{5.6e}
\]

where \(u\) is the peripheral length of loaded area (mm) and \(\gamma_h\) is the member factor equal to 1.3.

5.5.1.4 AASHTO (2018)

For sections without shear reinforcement, the nominal punching resistance of concrete \(V_c\) in kips shall be taken as Eq. (5.7)

\[
V_c = 0.316k \sqrt{f_c} b_{o,0.5d} d \tag{5.7}
\]

where \(b_{o,0.5d}\) and \(d\) are in inches and \(f_c\) are in ksi.

5.5.1.5 Other Proposed Punching Equations

Mattys and Taerw (2000) the punching equation for FRP-reinforced concrete flat slabs based on the empirical models (from different codes) and the modification of available mechanical and analytical models

\[
V_c = 1.36 \frac{(100 \rho_f E_f / E_s f_c)^{1/3}}{d^{1/4}} b_{o,1.5d} d \tag{5.8}
\]

where \(b_o\) computed at 1.5\(d\) from the plate face with the same shape of the plate.

El-Ghandour et al. (2000) modified the equivalent reinforcement ratio of the steel equation by replacing \(\rho_s\) with \(\rho_f (E_f/E_s)\). In addition, the factor of strain correction was modified with a strain limit of 0.0045 for FRP bars, resulting in this equation for FRP-reinforced concrete slabs:
\[ V_c = 0.79 \left[ 100 \rho_f \left( \frac{E_f}{E_s} \right) \left( \frac{0.0045}{\varepsilon_y} \right) \right]^{1/3} \left( \frac{f_{cu}}{25} \right)^{1/3} \left( \frac{400}{d} \right)^{1/4} b_{o,1.5d} d \]  

(5.9)

Based on the authors’ theoretical analysis, Theodorakopoulos and Swamy (2008) developed the following equation to predict the punching capacity of FRP-reinforced concrete slabs

\[ V_c = 0.117 f_{cu}^{2/3} \left( \frac{100}{d} \right)^{1/6} b_{o,1.5d} \frac{2\alpha_f \lambda_f}{1 + \alpha_f \lambda_f} d \]  

(5.10a)

\[ \alpha_f = \rho_f \frac{E_f}{0.145 f_{cu}} > 0.33 \]  

(5.10b)

\[ \lambda_f = \left( k_f / 6 \right) \left( -1 + \sqrt{1 + 48 / \alpha_f} \right) < 1 \]  

(5.10c)

where \( k_f \) is the coefficient reflecting the bond characteristics of the FRP bars equal to 0.55.

Hassan et al. (2017) suggested a new design model for estimating the punching-shear capacity of FRP-reinforced concrete slabs depending on the statistical analysis of their experimental results and other databases in the literature, yielding:

\[ V_c = 0.065 \phi_c \left( \frac{4d}{b_{o,0.5d}} + 0.65 \right) \left( E_f \rho f_{cu} \right)^{1/3} \left( \frac{125}{d} \right)^{1/6} b_{o,0.5d} d \]  

(5.11)

### 5.5.2 Comparison of Predictions and Experimental Results

The punching-shear capacities of all GFRP-reinforced PCTL segments were computed with the aforementioned equations to estimate the applicability of the available FRP-reinforced concrete provisions. For comparison, the integrated factors—such as material safety, loads, and resistance reduction factors—were set equal to unity. As shown in Table 5.4 and Fig. 5.13, the accuracy of the punching-shear equations was evaluated herein by comparing their predictions to the obtained experimental results. Generally, the available punching equations in design codes for FRP-reinforced concrete slabs yielded conservative predictions for GFRP-reinforced PCTL segments. In contrast, the researchers’ proposed equations overestimated punching capacities. ACI 440.11-22 and AASHTO (2018) yielded similar highly conservative predictions with an average \( V_{exp}/V_{pred} \) of 1.96 and 1.88, respectively, and a COV of 4%. The excessive conservativeness can be attributed to the neutral-axis depth, which is computed only from the flexural reinforcement and modular ratios. Ignoring the axial stiffness of reinforcement reflects on the compression area of the segment cross-section. Consequently, this absence affects the contribution of the concrete, which, in turn,
reduces the punching strength. The concrete compressive strength limit in CSA S806-12 is 60 MPa. In contrast, using high-strength concrete of 70.7 and 66.9 MPa for specimens G\((0.46)\)-2.1-H and G\((0.86)\)-2.1-H, respectively, yielded good predictions. The \(V_{\text{exp}}/V_{\text{pred}}\) ratios for those segments were 0.95 and 1.04, respectively. Moreover, employing the maximum concrete strength of 60 MPa in G\((0.46)\)-2.1-H and G\((0.86)\)-2.1-H yielded an accurate prediction of 1.00 and 1.05, respectively. Therefore, the punching equation in CSA S806-12 might be applicable even if the concrete strength is higher than the maximum limit. Furthermore, substituting the square root of the concrete strength with the cubic root in punching-equation provisions yielded better predictions, particularly in the HSC segments. JSCE (1997) reasonably predicted the actual capacities of the segments with an average \(V_{\text{exp}}/V_{\text{pred}}\) of 1.23 ± 0.05 and a corresponding COV of 4% due to the direct implementation of axial stiffness for the FRP bars in the punching equation in addition to the size effect. In constrast, Mattys and Taerwe’s (2000) equation provided a slightly overestimated prediction with an average \(V_{\text{exp}}/V_{\text{pred}}\) of 0.94± 0.05. Nevertheless, El-Ghandour et al. (2000), Theodorakopoulos and Swamy (2008), and Hassan et al. (2017) showed exceedingly overestimated predictions. Moreover, the equations proposed by the researchers yielded better punching predictions for the NSC specimens than their HSC counterparts.

### Table 5.4 Experimental-to-predicted punching capacity.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>G((0.46))-2.1</td>
<td>2.05</td>
<td>1.97</td>
<td>1.07</td>
<td>1.23</td>
<td>0.95</td>
<td>0.91</td>
<td>0.63</td>
<td>0.70</td>
</tr>
<tr>
<td>G((0.86))-2.1</td>
<td>1.97</td>
<td>1.91</td>
<td>1.13</td>
<td>1.30</td>
<td>1.00</td>
<td>0.85</td>
<td>0.79</td>
<td>0.74</td>
</tr>
<tr>
<td>G((0.46))-2.1-H</td>
<td>1.91</td>
<td>1.84</td>
<td><strong>1.00</strong></td>
<td>1.20</td>
<td>0.87</td>
<td>0.78</td>
<td>0.79</td>
<td>0.65</td>
</tr>
<tr>
<td>G((0.86))-2.1-H</td>
<td>1.89</td>
<td>1.82</td>
<td><strong>1.05</strong></td>
<td>1.20</td>
<td>0.93</td>
<td>0.84</td>
<td>0.85</td>
<td>0.69</td>
</tr>
<tr>
<td>Mean</td>
<td>1.96</td>
<td>1.88</td>
<td>1.06</td>
<td>1.23</td>
<td>0.94</td>
<td>0.84</td>
<td>0.76</td>
<td>0.70</td>
</tr>
<tr>
<td>S.D.</td>
<td>0.07</td>
<td>0.07</td>
<td>0.05</td>
<td>0.05</td>
<td>0.05</td>
<td>0.05</td>
<td>0.08</td>
<td>0.04</td>
</tr>
<tr>
<td>COV (%)</td>
<td>4.00</td>
<td>4.00</td>
<td>3.40</td>
<td>4.00</td>
<td>6.00</td>
<td>6.00</td>
<td>11.00</td>
<td>10.00</td>
</tr>
</tbody>
</table>

Note: S.D. = standard deviation; COV = coefficient of variation.
* Calculated using a compressive strength of 60 MPa (CSA S806-12).
5.6 CONCLUSIONS

This study explored the performance of precast normal and high-strength concrete tunnel segments reinforced with variable GFRP reinforcement ratios. Based on the experimental and theoretical findings presented herein, the following conclusions can be drawn.

1. All tested segments exhibited the idealized mechanism of the punching-failure mode triggered by an instant decay in load-carrying capacity and a classical punching cone. Moreover, the steel plate sank into the concrete with no signs of flexural failure.

2. Increasing the GFRP flexural reinforcement ratio reduced reinforcement strain, deflection, and crack width, and flattened the angle of the inclination crack. The high reinforcement ratio yielded a greater failure surface, energy absorption, and normalized punching stress 30% and 29% higher for G(0.86)-2.1 and G(0.86)-2.1-H, respectively.

3. Employing HSC in the GFRP-reinforced PCTL segments significantly enhanced the pre-cracking behavior (increased the cracking load and pre-cracking stiffness). While all the PCTL segments satisfied the service limit of crack width, the HSC specimens had narrower and fewer cracks.

4. The ultimate normalized punching-shear stress was calculated by incorporating the cubic root of the concrete strength. Consequently, CSA S806-12 produced more accurate predictions—especially for HSC segments—than JSCE (1997), which used the square root of the concrete strength. The average $V_{exp}/V_{pred}$ for CSA S806-12 was $1.06 \pm 0.05$ and a COV of 3%.
5. CSA S806-12 specified a maximum limit of concrete compressive strength of 60 MPa. Despite that, using high-strength concrete of 70.7 and 66.9 MPa in specimens G_{0.46}-2.1-H and G_{0.86}-2.1-H, respectively, yielded good predictions. The $V_{\text{exp.}}/V_{\text{pred.}}$ ratios for those segments were 0.95 and 1.04, respectively.

Lastly, the test results show the effectiveness of using HSC in GFRP-reinforced PCTL segments under punching loads. Further research using a wide range of concrete strength, however, is warranted.
CHAPTER 6
CONTRIBUTION OF CLOSED TIES TO THE SHEAR STRENGTH OF GFRP-REINFORCED PRECAST CONCRETE TUNNEL LINING (PCTL) SEGMENTS: EXPERIMENTAL AND ANALYTICAL INVESTIGATIONS

Foreword

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Journal and Status:

Journal of Composites for Construction, under review.

Reference:


Note:

The manuscript had been slightly adjusted from the original paper by remembering the figures and tables to include the chapter number. In addition, the reference list has been moved to the appropriate section in the thesis as indicated in the table of contents.
Abstract

Current tunnel design provisions do not consider the shear resistance of transverse reinforcement in precast concrete tunnel lining (PCTL) segments and are often designed as a minimum reinforcement for shrinkage and temperature resistance. This study evaluated the behavior and shear strength of PCTL segments reinforced with glass fiber-reinforced polymer (GFRP) bars with and without closed shear ties. A total of five full-scale rhomboidal PCTL specimens with a 1500 x 250 mm rectangular cross section and an arched length of 2100 mm were constructed and tested under three-point loading up to failure. The testing parameters included the configuration of the transverse reinforcement (bars versus closed ties), the spacing of closed shear ties, the longitudinal reinforcement ratio, and concrete strength. The load–deflection behavior, cracking, failure mechanisms, shear capacities, and strain in the reinforcement and concrete are discussed herein. The results indicate that all specimens experienced shear failure due to diagonal tension failure or shear compression mode. The presence of shear stirrups enhanced the structural performance of the GFRP-reinforced PCTL segments. In addition, increasing the closed-tie ratio, longitudinal reinforcement ratio, and concrete strength increased the shear strength of the segments. A comparison between the test results and predictions made with the Critical Shear Crack Theory (CSCT) was introduced in the theoretical approach to evaluate the applicability of existing FRP design code provisions. The predictions produced according to FRP design codes fell between conservative and nonconservative. The modified CSCT yielded predictions closest to the experimental values with an accuracy of 98%.

Keywords: precast concrete tunnel lining (PCTL) segments; glass fiber-reinforced polymer (GFRP) bars; shear strength; closed ties; concrete strength; Critical Shear Crack Theory (CSCT); diagonal tension failure; rupture; design codes.
6.1 Introduction

Precast concrete tunnel lining (PCTL) segments are currently widely used in infrastructure projects (i.e., roadways, subways, gas pipelines, and wastewater tunnels) because they save time, satisfy the safety requirements, and offer high quality. Although steel-reinforced concrete tunnels are designed for a service life of more than 100 years, the corrosion of embedded steel reinforcement can accelerate failure and threaten structural integrity (Caratelli et al. 2016). The durability of PCTL segments is affected by the aggressive environmental conditions around tunnels involving exposure to deicing salts, seawater, stray currents, or the chloride-ion attack of steel bars in contaminated water (Spagnuolo et al. 2018). Deterioration occurs most frequently in stirrups due to their location, especially in honeycombs or segregation. Stirrup corrosion can then spread to the reinforcement cages, resulting in spalling of the concrete cover and catastrophic tunnel failure (Abbas et al. 2014). Annual reports on tunnel corrosion problems have revealed substantial costs for repairing and rehabilitating deteriorated segments.

In the case of PCTL segments, replacing the steel reinforcement with glass fiber-reinforced polymer (GFRP) reinforcement has proven to be a realistic, cost-effective solution to mitigate the corrosion problem (Caratelli et al. 2016, 2017; Spagnuolo et al. 2017, 2018; Meda et al. 2019; Hosseini et al. 2022a, 2022b). GFRP materials are gaining popularity in tunnel applications due to their high resistance to environmental attack, high tensile strength, light weight, long service life, and neutrality to electrical and magnetic disturbances (ACI 440.11-22; Benmokrane et al. 2021). These desirable nonmetallic characteristics create dielectric joints in PCTLs and prevent stray currents and the related corrosion (Caratelli et al. 2016). GFRP reinforcement eliminates the problem of crushing segments during the fabrication process by reducing the concrete cover. Considering all aspects, GFRP-reinforced PCTL segments seem to be an adequate alternative in harsh soil conditions.

Shear force is one of the straining actions that can be induced by the provisional and permanent loads on tunnels. According to tunnel provisions (ACI 544.7R-16; ACI 533.5R-20; ITA WG2-19), production and transient loads as well as grouting pressure in the construction stage are all provisional loads. Permanent loads include earth/groundwater pressure and surcharges in the service stage that affect the tunnel’s life span. The literature contains limited research on the flexural and thrust behavior of GFRP-reinforced PCTL segments. Moreover, there is a lack of
information on the shear strength of such elements. Hosseini et al. (2022) studied the effects of concrete strength and longitudinal GFRP reinforcement ratio on the structural response of PCTLs with a shear span-to-depth ratio \((a/d)\) of 6.3. Due to the high \(a/d\) of their specimens, most of the segments experienced flexural failure rather than shear failure because the flexural stresses increased. The authors concluded that the use of high-strength concrete (HSC) in tunnel segments increased the cracking and ultimate loads and enhanced the post-cracking stiffness. Increasing the longitudinal reinforcement ratio affected the failure behavior of the segments, which shifted from flexural mode to a combination of flexure and shear. Indeed, the shear behavior of GFRP-reinforced PCTL segments with or without closed ties has not been investigated.

Fundamentals of the shear transfer mechanism for GFRP-reinforced PCTLs can be similar to FRP-reinforced concrete beams and one-way slabs, despite the curved geometry of tunnels. Generally, concrete shear resistance represents the strength of the uncracked depth and dowel action, whereas stirrup shear strength is determined by stirrup limitations (ACI 440.11-22; JSCE 1997). There is a consensus in the literature that high longitudinal reinforcement ratios and the use of HSC increase the shear capacity of beams (Mahmoud and El-Salakawy 2014; Gurutzeaga et al. 2015; Al-Hamrani et al. 2021). The shear strength of stirrups in FRP-reinforced concrete beams has been examined in several studies, mainly focusing on investigating the type, configuration, and ratio of stirrups (Issa et al. 2016; Ali et al. 2017; Fan et al. 2021). The experimental results of these studies indicate that using stirrups in beams improved concrete resistance after the initiation of a shear crack. Furthermore, decreasing the spacing between the stirrups increased the confinement and controlled crack propagation, resulting in an increase in shear strength. The current study presents the first experimental evidence to evaluate the contribution of closed GFRP ties to the shear strength of PCTLs and the effect of different parameters on their shear behavior.

Despite precast tunnel segments with transverse reinforcement being extensively used in practice, international reports on tunnel design and construction do not address the contribution of stirrups to shear resistance. While Hosseini et al. (2022) determined theoretically the shear capacity of GFRP-reinforced PCTL segments, they neglected the shear resistance of closed ties. Moreover, current shear equations in FRP-RC standards (ACI 440.11-22; CSA S806-12; AASHTO 2018; JSCE 1997) do not consider the arch-shape effect of tunnels. Basically, more research is needed to study the effect of closed ties on GFRP-reinforced PCTLs and verify the mechanical model of
the CSCT. In addition, it is imperative to examine the applicability of the available equations in FRP provisions to predict the shear capacity of PCTLs reinforced with GFRP bars and ties. This paper is a part of an ongoing extensive research project conducted on PCTLs under different loading conditions at the University of Sherbrooke. In this study, the effects of transverse reinforcement (configuration and spacing), flexural reinforcement ratio, and concrete strength on the behavior of GFRP-reinforced PCTL segments under shear loads were evaluated experimentally and analytically.

6.2 Experiments

6.2.1 GFRP Bars and Ties

Two sizes of Grade II sand-coated GFRP reinforcement (No. 5 (15 mm) and No. 4 (13 mm)) were used to reinforce the PCTL segments in the longitudinal and transverse directions, respectively (Fig. 6.1). Pultrusion and an inline sand-coating process for the bar surface were used in manufacturing the GFRP reinforcement in both directions. For the longitudinal direction, newly developed curvilinear GFRP bars were fabricated at Pultrall Inc. (Thetford Mines, Quebec, Canada) with a radius of 3305 mm and 3405 mm for the bottom and top mats, respectively. The tensile strength, modulus of elasticity, and ultimate tensile strain of the flexural curvilinear bars were calculated according to ASTM D7205-21. These bars were anchored using U-shaped GFRP closing bars to ensure proper end anchorage. The transverse direction of specimens was reinforced with GFRP straight bars and closed ties. The bent tensile strength of the closed ties was determined according to AASHTO (2018) and CSA S6-19. Table 6.1 provides the mechanical properties of the GFRP reinforcing bars and ties based on their nominal cross-sectional area.

<table>
<thead>
<tr>
<th>Reinforcement type</th>
<th>Bar size</th>
<th>Bar diameter (mm)</th>
<th>Nominal area (mm²)</th>
<th>Elastic tensile modulus (GPa)</th>
<th>Tensile strength (MPa)</th>
<th>Tensile strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Curvilinear bars</td>
<td>#5</td>
<td>15</td>
<td>199</td>
<td>55.1 ± 1.251</td>
<td>1115 ± 601</td>
<td>2.0 ± 0.11</td>
</tr>
<tr>
<td>U-shaped closing bars</td>
<td>#5</td>
<td>15</td>
<td>199</td>
<td>53.5 ± 1.10</td>
<td>1283 ± 42</td>
<td>2.4 ± 0.1</td>
</tr>
<tr>
<td>Transverse reinforcement</td>
<td>#4</td>
<td>13</td>
<td>129</td>
<td>55.6 ± 1.60</td>
<td>1248 ± 74</td>
<td>2.2 ± 0.1</td>
</tr>
<tr>
<td>(closed ties/bars)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( f_{tu, bent} = 518^2 )</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( f_{tu, bent} = 345^3 )</td>
<td></td>
</tr>
</tbody>
</table>

1 The reported properties were calculated according to ASTM D7205-21.
2,3 The bent tensile strength was determined using AASHTO (2018) and CSA S6-19, respectively.
Chapter 6: Contribution of Closed Ties to the Shear Strength of GFRP-PCTL Segments

Figure 6.1 GFRP reinforcement: (a) curvilinear bars, (b) U-shaped closing bars, and (c) transverse reinforcement (closed ties or bars).

6.2.2 Concrete

Four PCTL segments were cast with normal-strength concrete (NSC), while the fifth one was cast with high-strength concrete (HSC). The two mixes had a target 28-day compressive strength of 40 MPa for the NSC and 70 MPa for the HSC. The concrete compressive strength for all specimens was determined based on the average test results of six concrete cylinders measuring 100 x 200 mm tested on the same day of testing. The NSC’s compressive strengths ranged from 50.2 MPa to 51.8 MPa. The actual compressive strength of the HSC segment was 70.1 MPa. Table 6.2 shows the mix designs of the two types of concrete.

<table>
<thead>
<tr>
<th>Concrete Type</th>
<th>Cement (kg/m³)</th>
<th>Sand (kg/m³)</th>
<th>Limestone (kg/m³)</th>
<th>Superplasticizer (mL/m³)</th>
<th>Air-entrainment (mL/m³)</th>
<th>Water (L/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NSC</td>
<td>450</td>
<td>615</td>
<td>1015</td>
<td>4500</td>
<td>140</td>
<td>170</td>
</tr>
<tr>
<td>HSC</td>
<td>475</td>
<td>778</td>
<td>800</td>
<td>7000</td>
<td>170</td>
<td>135</td>
</tr>
</tbody>
</table>

6.2.3 Segment Design and Fabrication

The five full-scale GFRP-reinforced PCTL segments were designed, constructed, and tested under an increasing monotonically shear load. Figure 6.2 shows segment configuration, dimensions, and
reinforcement details. Each specimen was labeled with the letter G, indicating GFRP reinforcement, both longitudinally and transversely. The subscript in parenthesis refers to the flexural reinforcement ratio. T(x) or C(x) denote the spacing of the transverse bars or closed stirrups, respectively, whereas the letter H stands for high-strength concrete. The test matrix was arranged to assess their contribution to the shear strength of tunnel segments (Table 6.3). The effect of shear stirrups was investigated with No. 4 closed GFRP ties and transverse bars at a spacing of 200 mm. In addition, their ratio was investigated with No. 4 closed GFRP ties spaced at 100 mm and 200 mm. Specimen $G_{(0.86)/C100}$ had a tie spacing less than the maximum spacing limit specified in CAN/CSA S806-12 (CSA 2012), which represents the minimum of $0.6d_V \cot \theta$ and 400 mm. The closed-tie spacing in $G_{(0.46)/C200}$, $G_{(0.86)/C200}$, and $G_{(0.86)/C200-H}$ was as per that recommended in tunnel standards (ACI 2020 and ITA 2019). Furthermore, a control specimen ($G_{(0.46)/T200}$) with transverse bars was included in the test variables for comparison. Two longitudinal reinforcement ratios of 0.46% and 0.86% (7 and 13 No. 5 GFRP bars, respectively) were used for the top and bottom meshes. To ensure that the desirable failure mode occurred, the segments were designed with a flexural reinforcement ratio greater than balanced (ACI 440.11-22). Segments with a high longitudinal reinforcement ratio were fabricated with NSC and HSC. The GFRP cages were assembled for different segment configurations and reinforcement details (Fig. 6.3(a)).

The cages were placed into curved wooden formwork and cast with an overhanging crane bucket (Figs. 6.3(b) and 6.3(c)) at the Sym-Tech precast concrete facility in Saint-Hyacinthe, Quebec, Canada. The clear concrete cover was kept constant at 40 mm for the tested segments. The forms were stripped and the specimens stored after curing in a storage area (Figs. 6.3(d) and 6.3(e)). The typical subway tunnel comprises full parallel rings with seven different geometries of segments for each ring. In this study, the segments were rhomboid in shape with internal and external diameters of 6500 mm and 7000 mm, respectively. The dimensions for the segments were identical, measuring 2100 mm in arc length, 1500 mm in width, and 250 mm in thickness, with a constant shear span-to-effective depth ratio of 3.6.
Figure 6.2 Segment configuration and reinforcement details (all dimensions in mm).
Table 6. 3 Test matrix and segment details.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Dimensions (mm)</th>
<th>Longitudinal reinforcement</th>
<th>Concrete type</th>
<th>$f'_c$ (MPa)</th>
<th>Transverse reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>$G_{(0.46)/T200}$</td>
<td>2100 x 1500 x 250</td>
<td>7 No. 5</td>
<td>0.46</td>
<td>1.3</td>
<td>NSC</td>
</tr>
<tr>
<td>$G_{(0.46)/C200}$</td>
<td>7 No. 5</td>
<td>0.46</td>
<td>1.3</td>
<td>NSC</td>
<td>50.9</td>
</tr>
<tr>
<td>$G_{(0.86)/C100}$</td>
<td>13 No. 5</td>
<td>0.86</td>
<td>2.5</td>
<td>NSC</td>
<td>51.8</td>
</tr>
<tr>
<td>$G_{(0.86)/C200}$</td>
<td>13 No. 5</td>
<td>0.86</td>
<td>2.5</td>
<td>NSC</td>
<td>50.2</td>
</tr>
<tr>
<td>$G_{(0.86)/C200-H}$</td>
<td>13 No. 5</td>
<td>0.86</td>
<td>1.9</td>
<td>HSC</td>
<td>70.1</td>
</tr>
</tbody>
</table>

1 “G” denotes GFRP reinforcement, with a subscript (x%) representing flexural reinforcement ratio; (Tx) or (Cx) refers to the spacing of transverse bars or closed stirrups, and the suffix H stands for high-strength concrete.

2 $\rho_b$ calculated according to ACI 440.11-22 for FRP-reinforced concrete structures.

![Figure 6](image_url)

Figure 6. 3 Overview of (a) assembled cages, (b) cage in formwork ($G_{(0.86)/C100}$), (c) casting, (d) stripping, and (e) storage.
6.2.4 Instrumentation and Test Setup

Strain in the flexural bars and closed ties was measured with an electric strain gauge with a gauge length of 10 mm. The two central curvilinear bars in each PCTL segment were instrumented with a single resistance gauge at mid-span. Four strain gauges were placed at the mid-height of the stirrups’ vertical legs (Fig. 6.2). Two concrete strain gauges (C1 and C2) with a gauge length of 60 mm were mounted directly under loading on the extrados and lateral surfaces to measure compressive strains. In addition, four strain gauges (C3 to C6) were glued to the lateral side to measure concrete diagonal strains along the shear span. Segment deflection was monitored by five linear potentiometers (LPOTs): three distributed along the central width and two fastened at quarter-span. Two fixed high-accuracy LVDTs were installed diagonally on the lateral surface of the segments to measure shear-crack width. Flexural cracks were inspected visually, and their width was first obtained with an Elcometer crack-width ruler. Then, the cracks were highlighted and measured with three LVDTs.

The segments were loaded under three-point bending over a simply supported center-to-center span of 1400 mm, as shown in Fig 6.4. The line load was applied monotonically with an MTS 1000 kN actuator attached to a spreader beam at a displacement-controlled rate of 0.5 mm/min. The specimens were placed on cylindrical steel supports covered with Teflon sheets to ensure free movement and rotation. The experiment was video recorded, and measurements were collected with a data-acquisition system.
6.3 Test Results and Discussion

This section provides a summary of the experimental results obtained for all the GFRP-reinforced PCTL segments, as listed in Table 6.4. More details about the effect of test parameters on shear behavior are discussed subsequently.
Table 6.4 Test results.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Cracking load, $V_c$ (kN)</th>
<th>Ultimate shear load, $V_{exp}$ (kN)</th>
<th>Deflection (mm)</th>
<th>Stiffness (kN/mm)</th>
<th>Normalized shear strength, $V_{nor}$ (MPa)</th>
<th>Maximum strain, $\varepsilon_{max}$ ($\mu$s)</th>
<th>Major shear crack at failure</th>
<th>Mode of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>G(0.46)/T200</td>
<td>108 125</td>
<td>334</td>
<td>167</td>
<td>0.94 17.2</td>
<td>104 13.4</td>
<td>0.15</td>
<td>8590 — — — —</td>
<td>DT</td>
</tr>
<tr>
<td>G(0.46)/C200</td>
<td>113 140</td>
<td>366</td>
<td>183</td>
<td>0.81 19.1</td>
<td>102 16.3</td>
<td>0.17</td>
<td>8790 9160 -1580</td>
<td>DT</td>
</tr>
<tr>
<td>G(0.86)/C100</td>
<td>126 275</td>
<td>467</td>
<td>233</td>
<td>0.73 21.1</td>
<td>125 34.6</td>
<td>0.22</td>
<td>7310 5475 -2900</td>
<td>SC+TR</td>
</tr>
<tr>
<td>G(0.86)/C200</td>
<td>101 196</td>
<td>407</td>
<td>207</td>
<td>1.09 15.8</td>
<td>100 31.1</td>
<td>0.19</td>
<td>4450 3160 -2600</td>
<td>DT</td>
</tr>
<tr>
<td>G(0.86)/C200-H</td>
<td>145 290</td>
<td>556</td>
<td>278</td>
<td>0.87 18.4</td>
<td>135 36.8</td>
<td>0.24</td>
<td>5595 8040 -1270</td>
<td>DT</td>
</tr>
</tbody>
</table>

1 Obtained due to initial flexural and shear cracks.
2 Indicated the ultimate stiffness before the initiation of a major shear crack.
3 Calculated according to CSA S806-12 for FRP-reinforced building structures.
4 DT is diagonal tension failure, SC is shear compression failure, and TR is closed GFRP-tie rupture.

6.3.1 Effect of Test Parameters on Load–Deflection Behavior

The load–deflection curves at the center of the tested PCTL segments are presented in four groups to show the influence of individual parameters on shear behavior, as depicted in Fig. 6.5. The three LPOTs at mid-span measured an almost coincident deflection, indicating no torsion occurred. Generally, all the PCTLs had three distinct regions (pre-cracking, post-cracking, and post-peak), representing the different loading stages. The first region is prior to the onset of the flexural crack; segments revealed linear and steep behavior. In the second region, the segments behaved linearly due to the linear elastic characteristics of the GFRP bars, with reduced stiffness and increased deflection until failure. Lastly, post-peak behavior is described in the third region and was affected by the test matrix.

6.3.1.1 Effect of Shear Closed Ties

Figures 6.5(a) and (b) show the effect of shear reinforcement and its ratios on load–deflection behavior, respectively. Specimens G(0.46)/C200 and G(0.86)/C200 had a closed-tie ratio (0.09%) lower than the minimum, while G(0.86)/C100 had one that was slightly higher (0.17%). The minimum shear reinforcement ratio for segments was computed according to ACI 440.11-22, as in the following equation:

$$\rho_{fr, min} = \frac{A_{fr, min}}{b_u s} = \frac{0.75 \sqrt{f_c}}{f_b}$$  \hspace{1cm} (6.1a)
where \( f_{R} = 0.005E_{f} \leq f_{R} \text{ (psi)} \) \hspace{1cm} (6.1b)

The figures indicate that, regardless of shear stirrups, the pre-cracking region of the load–deflection curves for all segments follow the same trend. In contrast, segment \( G_{(0.86)/C100} \) had 1.2 times the initial stiffness of segment \( G_{(0.86)/C200} \) due to a slight increase in concrete strength. While the ratio of closed ties in \( G_{(0.46)/C200} \) was less than the maximum limit, it increased the post-cracking stiffness and ultimate deflection by 22% and 12%, respectively (Fig. 6.5(a)). This could be attributed to the closed ties enhancing the confinement of the concrete and GFRP bars (Jumaa and Yousif 2019). Decreasing the stirrup spacing from 200 mm to 100 mm significantly improved the behavior after cracking. Segment \( G_{(0.86)/C100} \) had ultimate stiffness 12% higher than \( G_{(0.86)/C200} \) (Fig. 6.5(b)). The major diagonal shear crack was initiated in \( G_{(0.86)/C100} \) at a high load level of 445 kN. Then, the high closed-tie ratio exhibited efficient control of crack width and propagation. Consequently, the ratio maintained confinement and increased stiffness at high load levels, which is consistent with past findings (Ali et al. 2017). Segment \( G_{(0.46)/T200} \) did not show any post-peak carrying capacity after the formation of the major shear crack due to the absence of shear stirrups. In contrast, the crack propagated between the ties in specimens \( G_{(0.46)/C200} \) and \( G_{(0.86)/C200} \) resulting in the load decreasing steadily. The shear reinforcement permitted stress redistribution, but the increased spacing between the stirrups prevented the load recovery from exceeding the ultimate. This is apparent in the post-peak behavior of specimen \( G_{(0.86)/C100} \): each increase in stiffness corresponds to crack propagation crossing over the tie legs. The results are contrary to the assumptions in the ACI 533.5R-20 tunnel provisions, which state that transverse reinforcement is used for shrinkage and temperature purposes only.

### 6.3.1.2 Effect of Concrete Strength

Figure 6.5(c) shows the effect of concrete compressive strength on the load–deflection curves of segments \( G_{(0.86)/C200} \) and \( G_{(0.86)/C200-H} \). In the first portion, the use of HSC significantly enhanced the uncracked behavior of specimen \( G_{(0.86)/C200-H} \). High-strength concrete enabled this specimen to produce 35% higher initial stiffness than its NSC counterpart segment \( G_{(0.86)/C200} \) due to higher flexural strength at the uncracked stage. According to the figure, segment \( G_{(0.86)/C200-H} \) had steeper load–deflection behavior in the second portion than segment \( G_{(0.86)/C200} \). Stiffness is a function of the modulus of elasticity of concrete, which is equal to \( 4700 \sqrt{f_c} \) (ACI 318-19).
Therefore, the HSC segment had 15% higher ultimate stiffness, which is consistent with the increase in the square root of the concrete compressive strength between the two segments. Upon initiation of the major shear crack, both segments experienced some increase in load-carrying capacity until failure. The ultimate deflections corresponding to the failure loads were 15.8 mm for segment G(0.86)/C200 and 18.4 mm for segment G(0.86)/C200-H. Both segments had soft descending branches in the third portion due to the presence of shear ties.

6.3.1.3 Effect of Longitudinal Reinforcement Ratio

Figure 5(d) depicts the influence of the GFRP longitudinal reinforcement ratio on the shear strength of the PCTL segments. Segments G(0.46)/C200 and G(0.86)/C200 had almost pre-cracking stiffness in the first region. This could be attributed to the negligible effect of the flexural reinforcement ratio on the gross moment of inertia in the tunnel cross section (Mousa et al. 2019b). When the load increased, the section changed from uncracked to cracked in the second region. Therefore, the characteristics of the segment’s section depended on the effective moment of inertia. The post-cracking stiffness of the specimens relied mainly on the flexural reinforcement ratio. Increasing the reinforcement ratio from 0.46% to 0.86% in G(0.86)/C200 increased the ultimate stiffness by 90%. Moreover, the steep slope of the load–deflection curve reduced deformation by 17%. After failure occurred, the load-carrying capacity of the segments gradually decreased as they tried to recover.
Figure 6. 5 Effect of test parameters: (a) closed ties, (b) spacing of shear reinforcement, (c) concrete strength, and (d) longitudinal reinforcement ratio.

6.3.2 Cracking Load and Pattern

Figure 6.6 provides schematic drawings of crack propagation and the corresponding loads (kN) during the test history. Similar features in the cracking pattern were observed in all the segments tested. The first flexural cracks initiated at the intrados surface of segments and developed vertically underneath the loading area, where the flexural stress was highest. The cracks occurred at loads of 108, 113, 126, 101, and 145 kN for specimens $G_{0.46}/T200$, $G_{0.46}/C200$, $G_{0.86}/C100$, $G_{0.86}/C200$, and $G_{0.86}/C200$-H, respectively. With further loading, additional flexural cracks—perpendicular to the intrados surface—appeared inside/outside the maximum bending region, while the existing ones propagated toward the compression zone and widened. Thereafter, shear stresses dominated segment behavior, resulting in flexural-crack stabilization. These cracks, however, grew as flexural-shear cracks and inclined progressively toward the line load. Inclined shear cracks were also formed from the bottom surface with flatter angles and extended to intersect
Chapter 6: Contribution of Closed Ties to the Shear Strength of GFRP-PCTL Segments

with the flexural-shear cracks. At a load level of 37%–58% of the peak load, segments developed initial shear cracks. The intensity of shear cracks increased gradually with load increments, leading finally to one or two critical shear cracks. Segments $G_{(0.46)/T200}$, $G_{(0.46)/C200}$, $G_{(0.86)/C100}$, $G_{(0.86)/C200}$, and $G_{(0.86)/C200-H}$ had major shear cracks at loads of 310, 350, 445, 365, and 495 kN, respectively, representing 89%–95% of the failure load.

Replacing transverse bars with closed stirrups increased the load of the initial shear crack by 12% in $G_{(0.46)/C200}$. Furthermore, the load was 40% higher in segment $G_{(0.86)/C100}$ with a higher stirrup ratio than segment $G_{(0.86)/C200}$. The closed ties significantly affected the number and spacing of the final crack patterns of the tested specimens. Segment $G_{(0.86)/C100}$ showed more flexural cracks due to the closer spacing between shear ties. This could be attributed to ties acting as crack initiators that induced 50% lower spacing between the flexural cracks than $G_{(0.86)/C200}$ (Dawood and Marzouk 2012). The presence of closed ties in $G_{(0.46)/C200}$ decreased the flexural crack spacing by 17% compared to $G_{(0.46)/T200}$. The closed ties contributed to enhancing the cracking behavior of the tunnel segments; decreasing the crack spacing reduced the crack width, and the segments could thus resist higher loads. The use of HSC in specimen $G_{(0.86)/C200-H}$ showed more cracks compared to its counterpart made with NSC due to its lower ability to dissipate energy (Mahmoud and El-Salakawy 2014). The high flexural reinforcement ratio in $G_{(0.86)/C200}$ compared to $G_{(0.46)/C200}$ decreased the number and depth of cracks.
6.3.3 Modes of Failure

Figure 6.7 consists of photographs of the final state of the tested PCTL segments. All specimens experienced shear failure (Table 6.4); no bonding deterioration or flexural failure was observed. When one or two major shear cracks extended to the loading point on the extrados surface, a diagonal tension failure occurred for almost all segments. In contrast, segment \( G_{c.86}/C100 \) failed in shear compression mode, followed by tie rupture. The major shear-crack angles for the tested segments varied from 24° to 36° at failure. The theoretical inclination of critical shear cracks was calculated based on strain in the longitudinal bars of the segments (CSA S806-12). The results were in the range of 55°–71°, which is not consistent with the experimental observations. Experimentally and theoretically, the crack inclination angle of the NSC segment \( G_{c.86}/C200 \) was steeper than that of the HSC segment. The complicated shear transfer in arched members—in
addition to several reasons found elsewhere (Jumaa and Yousif 2019)—can exhibit this uneven and highly separated variation.

Segment G\(_{(0.46)}\)/T\(_{200}\) (with transverse bars) experienced brittle failure, characterized by an instant drop in load-carrying capacity and explosive sounds. Specimens G\(_{(0.46)}\)/C\(_{200}\), G\(_{(0.86)}\)/C\(_{200}\), and G\(_{(0.86)}\)/C\(_{200}\)-H (with wide stirrup spacing) had less brittle failure. Regardless of concrete strength or the flexural reinforcement ratio, the segments evidenced a smooth decrease in load after failure, and the deformation continued to increase. The major shear crack widened in specimen G\(_{(0.86)}\)/C\(_{100}\) at a high load level, resulting in the loss of aggregate interlock. Then, the segment redistributed the shear transfer mechanism along the shear span. Closer spacing between the ties enhanced the resistance and carried additional loads until the upper concrete strain reached 2900 µε. The GFRP stirrups continued to maintain segment resistance and exhibited higher deformation until the segment ruptured. This could be attributed to the potential of closed ties to resist the tensile stress across the major shear crack, as previously described (Mohamed et al. 2017). The ductile shear failure achieved in our study supports the idea that closed ties rather than transverse bars could have a valuable effect on the failure mechanism of PCTL segments. Closed ties provide advanced warning signs of failure, which meet the safety requirements for tunnels. As a result, tunnel codes need to consider the transverse reinforcement configuration to assess its impact on the principles adopted for stirrup function.
6.3.4 Shear Strength of PCTL Segments

The contribution of the concrete and closed ties accounts for the shear capacity of all the PCTL segments tested, except $G_{(0.46)/T200}$, in which only the concrete contributed. The shear strength was normalized to contain its variation with respect to the cubic root of concrete strength ($b_n d \sqrt{f_c}$) according to shear design equations (CSA S806-12 and JSCE 1997). Figure 6.8(a) depicts the normalized shear strength versus the ratios of closed ties. The figure confirms that either the presence of closed ties or an increase in their ratios increased the normalized shear strength of the tunnel segments linearly with a nearly linear average trend. The closed ties in specimen $G_{(0.46)/C200}$ increased the normalized shear strength by 13%. Herein, stirrups restrained the propagation of the critical shear crack to a certain extent and continued to resist more loads. Ties and flexural bars served as hoops to restrict the concrete, which contributed to increasing the shear strength (Fan et al. 2021). The corresponding value was 16% when the closed-tie ratio increased from 0.09% to
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0.17% for $G_{(0.86)/C100}$. This increase can be attributed to the narrower spacing between stirrups and the related increase in numbers that resisted shear loads. A large number of ties intersected the major shear crack, resulting in distributed shear force and reduced strain in the ties.

Figure 6.8(b) shows the nonlinear relationship between the longitudinal reinforcement ratio and normalized shear strength for the PCTL segments. The figure implies that normalized strength is proportional to the flexural reinforcement ratio to the power of 0.4, which is close to the 1/3 specified in shear design equations (CSA S806-12 and JSCE 1997). The increase in normalized strength was 12% for segment $G_{(0.86)/C200}$ with approximately a 48% increase in the amount of flexural reinforcement. As expected, the segment with a high reinforcement ratio had increased dowel action and a lower loss of flexural stiffness. In addition, the flexural reinforcement improved the shear transfer mechanism by decreasing the width and penetration depth of the shear cracks. Consequently, the aggregate interlock, residual strength, and concrete contribution increased, yielding a higher shear capacity. It is worth noting that using stirrups had the same effect as increasing the longitudinal reinforcement ratio on the normalized shear strength of the PCTL segments. Thus, replacing the increase in this ratio with closed GFRP ties in the manufacture of segments is considered cost-effective in tunnel projects.

Figure 6.8(c) shows the experimental shear capacities of the tested specimens to the concrete compressive strengths. Segments $G_{(0.46)/T200}$, $G_{(0.46)/C200}$, $G_{(0.86)/C200}$, and $G_{(0.86)/100}$ had almost the same concrete strengths, which is consistent with the maximum limits of ACI 440.11-22, AASHTO (2018) (69 MPa) and CSA S806-12 (60 MPa) for predicting shear strength. There was, however, a variation in shear capacity governed by the contribution of stirrups and longitudinal bars. For specimens with constant flexure and shear ratios, increasing the concrete strength by 40% (from 50.2 to 70.1 MPa) increased the shear capacity by 52% for segment $G_{(0.86)/C200-H}$. This result is reliable because concrete shear resistance is a function of compressive strength, which is considered one of the main parameters, even if it exceeds limits.
Figure 6.8 (a) relationship between normalized shear strength and closed-tie ratio, (b) normalized shear strength versus reinforcement ratio, and (c) shear capacity versus concrete strength.

6.3.5 Load-Shear-Crack Width

The FRP design provisions limit the maximum allowable flexural crack opening to 0.5 and 0.7 mm for exterior and interior exposure, respectively, but only for esthetic considerations (CSA S806-12). At early loading stages, the flexural cracks for all the PCTL segments were narrower than the serviceability requirement before transitioning to shear cracks. There are no specific limits on the shear crack opening, but the strain values of FRP stirrups can be used as a method to control it. Figure 6.9 provides the maximum width of the critical shear crack formed on the lateral sides of the tested segments and was measured with two attached LVDTs. The figure indicates that the major shear crack was observed at high load levels and had widths of 1.54, 3.39, 3.71, 3.69, and 1.11 mm at failure for specimens G(0.46)/T200, G(0.46)/C200, G(0.86)/C100, G(0.86)/C200, and G(0.86)/C200-H, respectively. At the same load level, segment G(0.46)/T200 (with transverse bars) evidenced a wider
crack opening than \( G_{(0.46)/C200} \) (with closed ties). Furthermore, due to the inclined cracks passing through the LVDT, both segments recorded values for crack width prior to the initiation of a major shear crack at lower ratios of the applied loads. It was found that the shear crack width decreased with increasing longitudinal and shear reinforcement ratios. The effect of flexural reinforcement was, however, larger and more pronounced on the shear crack width of the PCTL segments. Using HSC in segment \( G_{(0.86)/C200-H} \) produced a large number of narrower cracks. The results reveal that increasing the stirrups, longitudinal bars, and concrete strength enhanced the esthetic shape while meeting the safety requirements during all practical tunnel stages.

Figure 6.9 Load-critical shear-crack width relationship.

6.3.6 Flexural Reinforcement and Concrete strains

Figure 6.10 shows the relationships between the applied load and flexural strain in the longitudinal bars and concrete for tested PCTL segments. Tensile and compressive strains were recorded at the center of specimens (maximum flexural stresses) for the bottom curvilinear GFRP bars and concrete extrados surfaces, respectively. The figure reveals negligible flexural strain before the initial cracks occurred in both the bars and concrete, with corresponding values close to 100 \( \mu \varepsilon \). Once the section had cracked, instant jumps in the flexural strains were observed, coinciding with fluctuations in the applied load for all the segments.

After cracking occurred, the strain reading in the longitudinal reinforcement increased, and the segments behaved more linearly until failure. The maximum recorded values were 8590, 8790, 7310, 4450, and 5590 \( \mu \varepsilon \) for segments \( G_{(0.46)/T200} \), \( G_{(0.46)/C200} \), \( G_{(0.86)/C100} \), \( G_{(0.86)/C200} \), and \( G_{(0.86)/C200-H} \), respectively, representing 22%–44% of the rupture strain of the curvilinear GFRP bars. The
The tested specimens had maximum concrete strains in the range of 1270 to 2900 $\mu$e, which is less than the theoretical crushing strains of 3000 $\mu$e as specified in ACI 440.11-22 and 3500 $\mu$e as in S806-12. Accordingly, the load–flexural strain response for the segments is consistent with crack propagation and shear failure modes. There were no signs of flexural failure—such as bar rupture and/or concrete crushing—during segment testing. Segment $G_{0.86}/C_{100}$ had a horizontal crack under line load and high concrete strain close to the maximum limit. This implies that the high closed-tie ratio confined the concrete core, which, in turn, converted the failure from diagonal tension to shear compression, followed by tie rupture. The presence of closed ties in segment $G_{0.46}/C_{200}$ had no effect on the tensile strain of the bars. This could be attributed to the location of the bar gauges, which were mounted at the center of the bars between two widely spaced stirrups. In contrast, flexural strain is mainly dependent on the amount of longitudinal reinforcement. Compared to $G_{0.46}/C_{200}$, increasing the reinforcement ratio in segment $G_{0.86}/C_{200}$ decreased the flexural strain in the bars and concrete at the same load level.

6.3.7 Strain in Closed GFRP Ties

Tensile strain in GFRP closed ties was measured at the shear span with four strain gauges attached to the vertical legs. The strain readings for ties in the left shear span were not consistent with the right ties located at the same distance. In addition, some of the measured strains were unreliable. This could be attributed to the location of the critical shear cracks with respect to the closed ties and gauges. Figure 6.11(a) shows the applied load versus maximum stirrup strain of the first left and right ties (S2 and S3). Generally, all segments showed the same response before cracking with...
negligible tensile strains. Then, a steady increase in strain was compatible with the formation of the initial shear cracks. The critical shear crack appeared at high load levels. It intersected with one of the first left and right ties or both at mid-height of the straight portion. The strain values increased rapidly only in the stirrups intersected by the major shear crack where the gauges were located. This might be attributed to the transition of shear resistance from the concrete to the ties.

Decreasing the spacing of the ties in segment G\(_{(0.86)/C100}\) decreased the recorded strains due to the participation of more stirrups to carry the applied load, which in turn, decreased the stirrups’ strain. The HSC segment (G\(_{(0.86)/C200-H}\)) had a higher ultimate stirrup strain than segment G\(_{(0.86)/C200}\) with NSC. In the HSC tunnel segments, the crack passed through the aggregate, resulting in smooth surfaces that reduced aggregate interlock. This increased the contribution of stirrups in resisting the load, which led to higher strain values in the ties. Despite the maximum measured strain of \(5475 \, \mu \varepsilon\) in segment G\(_{(0.86)/C100}\)—representing 25% of failure strain—it finally failed due to tie rupture. This is consistent with the findings of Jumaa and Yousif (2019), who reported that the crack intersected with ties at the weak bent portions, resulting in tie rupture at these locations.

Table 6.4 lists the maximum measured strains at mid-height of the straight portion, representing 1.3–3.7 times the limits of 2500 \(\mu \varepsilon\) (BISE 1999). These values are also much higher than the limits of 4000 \(\mu \varepsilon\) and 5000 \(\mu \varepsilon\) specified in CAN/CSA S6-19 and ACI 440.11-22, respectively. In contrast, segment G\(_{(0.86)/C200}\) had an ultimate strain lower than the latter limits for the reasons given above. Figure 6.11(b) depicts the relationship between the applied load ratio and the maximum strain ratio for each segment at different limits. The applied shear forces corresponding to the limit of 2500 \(\mu \varepsilon\) were 145, 230, 190, and 250 kN for segments G\(_{(0.46)/C200}\), G\(_{(0.86)/C100}\), G\(_{(0.86)/C200}\), and G\(_{(0.86)/C200-H}\), respectively, representing 79%–97% of the ultimate shear loads. Moreover, the ratios of the applied load at all limits are close to unity. This implies that the stirrups cannot undergo large strains before failure, which is clearly apparent from the observed failure of segment G\(_{(0.86)/C100}\).
Figure 6.11 (a) applied load versus maximum mid-height strain of closed GFRP ties and (b) relationship between the ratio of the applied load and the ratio of the maximum strain.

6.4 Analytical Shear Capacity

6.4.1 Critical Shear Crack Theory (CSCT) Failure Criterion

The Critical Shear Crack Theory was first introduced by Muttoni and Schwartz (1991) as a generalization of the diagonal shear failure envelope, in which the major shear crack widens and develops strains at adjacent locations. This hypothesis has been refined and applied to the shear-strength calculations for beams and slabs without shear reinforcement (Ruiz et al. 2015; Al-Hamaydeh and Orabi 2021). Theoretical modifications to the shear strength of two-way slabs according to the CSCT have been reported elsewhere (Ruiz and Muttoni 2009) and compared with experimental results available in the literature to include the contribution of the stirrups. In this case, once a major shear crack develops breaking the concrete teeth, the concrete carries a portion of the applied shear force, while the shear reinforcement carries the remaining portion. On that basis, the shear strength of beams or one-way slabs can be redefined as shown in Fig. 6.12(a). The following expression is proposed:

\[ V_{r,\text{in}} = V_{r,c} + V_{r,s} \]  \hspace{1cm} (6.2)

6.4.1.1 Shear Resistance of GFRP-Reinforced PCTL Segments

Based on the CSCT, an inclined compressive strut related directly to the opening and roughness of the critical shear crack provides the concrete contribution. Muttoni and Ruiz (2008) proposed the following equation depending on the resistance of this strut that only carries the shear load:
\[ V_{r,c} = b_w d \sqrt{f_c'}, f(w, d_g') } \]  
\[ (6.3) \]

The failure criterion provided in Eq. (6.3) links the function of width and roughness of the critical shear crack \( f(w, d_g') \) (assessed through the maximum aggregate size \( d_g \)) to the concrete resistance \( V_{R,c} \) (kN) of steel structures. The CSCT theory states—consistent with the considerations proposed in this study—that the critical crack opening is assumed to be proportional to a reference flexural strain times the effective depth \( W \propto \varepsilon_{f,c} d \). Based on this correlation, concrete shear resistance can be determined as in Eq. (6.4):

\[ V_{r,c} = \frac{b_w d \sqrt{f_c'}}{3(1+120 \frac{\varepsilon_{f,c} d}{d_g + d_g})} \]  
\[ (6.4) \]

where \( d_{go} \) is set to 16 mm. The control section is defined as 0.5\( d \) from the point load and assumes that the concrete’s compression region has linear elastic behavior, while neglecting the tensile strength of cracked concrete. For PCTL segments reinforced with GFRP bars, the reference flexural strain in the control section is computed at a depth of 0.6\( d \) (Campana et al. 2014) from the top surface (Fig. 6.12(b)) and can be rewritten as:

\[ \varepsilon_{f,c} = \frac{M}{A_f E_f z} 0.6d - x = \frac{V(a - 0.5d)}{A_f E_f (d - \frac{x}{3})} 0.6d - x \]  
\[ (6.5) \]

where \( M \) is the external moment at the control section; \( A_f \) and \( E_f \) are the area and modulus of elasticity of the longitudinal reinforcement; and \( z \) is the flexural lever arm between the compression force in the concrete and the tensile force of the reinforcement. The depth of the compression zone \( x \) can be calculated considering the factor of the neutral-axis depth for the cracked transformed section \((k)\) as follows (Fig. 6.12(c)):

\[ x = kd \]  
\[ (6.6a) \]

\[ k = \sqrt{2\rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f \]  
\[ (6.6b) \]

The arched shape of the tunnel segments deviates in the applied forces and stresses, which, in turn, affects the shear-force transfer through the critical shear crack. Thus, an additional shear force should be estimated due to the change in the tension force of curvilinear GFRP bars in PCTLs. Figures 6.12(d) and (e) show the free bodies and the corresponding equilibrium forces of shear
transfer by a critical shear crack. Equations (6.7) and (6.8) can be estimated from the equilibrium and geometry of free bodies:

\[ V_a = z_I F_c \cos \theta \]  
\[ \tan \alpha = \frac{\Delta V}{F_c \cos \theta} = \frac{\lambda d}{R_f} \]  
\[ \Delta V = \frac{V_a \lambda d}{z_I R_f} = \frac{V_a \lambda d}{z - \frac{(\lambda d)^2}{2R_f}} \]  

Hence, the additional force induced by the deviation forces is calculated as:

where \( \theta \) and \( \alpha \) are the slope angles of the resultant forces of the concrete and reinforcement, respectively; \( R_f \) is the curvature of the GFRP longitudinal bars; \( z_I \) is the length of the flexural lever arm at mid-span; \( a \) is the length of the shear span; and \( \lambda \) is a factor that stands for the length along which the deviation force is quantified. The literature introduced the factor \( \lambda \) to estimate the concrete contribution based on segment geometry, control-section location, critical shear-crack location and shape, and loading conditions (concentrated or distributed loads). According to Campana et al. (2014), the value of \( \lambda \) was assumed to be 0.50 for steel-reinforced arch-shaped members. In this study, however, the value adopted to consider the length of deviation forces for GFRP-reinforced PCTL segments using regression analysis was determined to be 0.65. During failure, the sum of the concrete contribution and deviation forces equals the total shear force through the CSC (for concrete only). The predicted concrete strength of segments can be calculated with Eq. (6.10):

\[ V_{cf} = V_{c,\infty} - \Delta V = V_{r,\infty} - \Delta V \]
6.4.1.2 Modification of CSCT for PCTL Segments with Closed Ties

As mentioned previously, the Critical Shear Crack Theory considers that the shear force applied to arched members is carried by the concrete alone as an inclined strut action. In such members, however, the experimental results demonstrated the effectiveness of using stirrups to increase the shear strength of tunnel segments. According to the main hypothesis of Ruiz and Muttoni (2009), shear reinforcement can contribute to the resistance to shear force in the CSC. Consequently, in this study, the CSCT was used at the preceding control section, in which the crack width and the effective strain in the closed ties of the segment were proportional:

\[ W \propto \varepsilon_{fr} \]  

(6.11)
Stress in stirrups is a function of their strain limit at the control section of PCTLs. Thus, the contribution of closed GFRP ties can be calculated based on the width of the CSC (which intersects the stirrups) by:

\[ V_{sf} = \sum_{i=1}^{n} \sigma_{fvi} (\varepsilon_{fvi}) A_{fvi} \]  

(6.12)

Based on the average strain limit of the nearest ties to the control section, Eq. (6.12) can be rewritten as:

\[ V_{sf} = \frac{A_{fv} E_{fv} \varepsilon_{fvr}}{S} d_v \]  

(6.13)

where \( A_{fv} \) is the total area of the closed ties; \( E_{fv} \) is the modulus of elasticity of the shear reinforcement; \( S \) is the spacing between stirrups; and \( d_v \) stands for the effective shear depth, which is assumed to be the maximum of 0.72\( h \) or 0.9\( d \). The strain value in the GFRP ties should be limited in order to control shear-crack width, maintain concrete shear integrity, and avoid failure at the bent portion of the ties. The current design codes and standards specify these limits with constant strain values (BISE 1999; CSA S6-19; ACI 2022) or proposed predicted equations (JSCE 1997; ISIS 2007). It should be noted that the strain limit \( \varepsilon_{fvr} \) of 0.004 was adopted in the control section and is suitable for calculating the effective stress of GFRP ties.

Lastly, once the contributions of the concrete and stirrups are known using the proposed values, the total predicted shear strength of the segments can then be obtained:

\[ V_{pred} = V_{cf} + V_{sf} \]  

(6.14)

The number of full-scale specimens in this study was limited. Indeed, future experimental work is needed to assess the accuracy of \( \lambda \) and \( \varepsilon_{fvr} \) in calculating the shear strength of PCTL segments with GFRP bars and closed ties.

### 6.4.2 Comparisons between the Predictions with the CSCT and Design Provisions

The specific tunnel codes and standards do not include equations related to the shear capacity of GFRP-reinforced PCTL segments. Based on the experimental results, a comparison of the predicted values using the simplified approaches in current FRP provisions (ACI 440.11-22; CSA
S806-12; AASHTO 2018; CSA S6-19; JSCE 1997) and those predicted by the Critical Shear Crack Theory for PCTLs was conducted. Calculations were performed using the equations reported in Table 6.5 and Eqns. (6.4)–(6.14) to evaluate the applicability of these methods in predicting the shear strength of PCTL segments with GFRP bars and ties. Both the modified CSCT and FRP design guidelines consider the shear capacity carried by the concrete shear resistance $V_{cf}$ and the stirrup’s contribution $V_{sf}$. The FRP provisions, however, neglect the effect of the curved shape of PCTLs in concrete contributions. The general form of the transverse reinforcement component is similar for all provision equations based on the truss mechanism assumption as is followed in steel-reinforced concrete design codes. The main difference between the equations presented, however, is the strain limit of the stirrups, which controls the opening of the shear crack. CAN/CSA S6-19 and AASHTO (2018) recommend an effective strain of 0.004 as the upper strain limit for FRP stirrups, with the design strength not exceeding the stirrups’ strength in the bent portion. The maximum tensile stress in stirrups should not exceed 0.005 $E_f$, 40% of the ultimate strength of the straight portion of the FRP ties or 1200 MPa according to CAN/CSA S806-12.
Table 6.5 Codified shear capacity for the concrete and stirrups.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Concrete contribution, $V_{cf}$ (kN)</th>
<th>Stirrup resistance, $V_{sf}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI 440.11 (2022)</td>
<td>$V_{cf} = \frac{2}{5} \lambda_s \sqrt{f_c b_u (kd)}$</td>
<td>$V_{sf} = A_k f_p d / S$</td>
</tr>
<tr>
<td></td>
<td>$\lambda_s = \sqrt{2 / (1 + d / 10)} \leq 1$ if $A_{fc} &lt; A_{fc,min}$</td>
<td>$f_p = 0.005 E_{fc} \leq f_{u,bent}$</td>
</tr>
<tr>
<td></td>
<td>$\lambda_s = 1$ if $A_{fc} \geq A_{fc,min}$</td>
<td>$V_{sf} = \phi f_{fc} d_v \cot \theta / S$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$f_{fc} = 0.005 E_{fc} \leq 0.4 f_u$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$d_v = \text{the maximum value of 0.72h or 0.9d}$</td>
</tr>
<tr>
<td>CSA S806-12 (2012)</td>
<td>$V_{cf} = 0.05 \lambda \phi k m k_s \sqrt{f_c b_u d_v}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$k_m = (V_f d / M_f)^{1/2} \leq 1$</td>
<td>$V_{sf} = \phi f_{fc} d_v \cot \theta / S$</td>
</tr>
<tr>
<td></td>
<td>$k_s = 1 + (E_f / \rho_f)^{1/3}$</td>
<td>$f_{fc} = 0.004 E_{fc} \leq f_{u,bent}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$f_{u,bent} = (0.05 f_b / d_b + 0.3) f_{fa} / 1.5$</td>
</tr>
<tr>
<td>AASHTO (2018)</td>
<td>$V_{cf} = 0.0316 \beta \sqrt{f_c b_v d_v}$ (kips)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>where $\beta = 5k$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CSA S6-19 (2019)</td>
<td>$V_{cf} = 2.5 \beta \phi f_{fc} c d_v$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\beta = 0.4 / (1 + 1500 \epsilon_s) . 1300 / (1000 + s_{zc})$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>JSCE (1997)</td>
<td>$V_{cf} = \beta_p \beta_p \beta_n f_{pcd} c d / \gamma_b$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\beta_d = (1000 / d)^{1/4} \leq 1.5$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\beta_p = (100 \rho_f E_f / E_c)^{1/3} \leq 1.5$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\beta_n = 1 + M_0 / M_d \geq 2$ if $N_d \geq 0$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\beta_n = 1 + 2M_0 / M_d \geq 0$ if $N_d \leq 0$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$f_{pcd} = 0.2 \sqrt{f_c} \leq 0.72 \text{MPa}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 6.6 provides the calculated shear resistance for the concrete and the closed GFRP ties for the PCTL segments tested. In addition, Figure 6.13 provides the ratios of the experimental results to the predicted shear strength to evaluate the accuracy of the prediction equations according to the CSCT and FRP standards. The comparison demonstrates that the high scatter in the normative predictions is mainly attributed to the predicted concrete strengths rather than the contribution of the GFRP ties. The CSCT yielded better predictions for PCTL segments reinforced with GFRP bars and stirrups than the current FRP design provisions. Based on the redefined CSCT (2014), the predicted shear strengths were reasonable compared to the actual capacities of the segments with an average $V_{exp} / V_{pred}$ of 1.23 ± 0.12. The modified CSCT in this study, however, yielded the most accurate predictions with an average $V_{exp} / V_{pred}$ of 0.98 and a corresponding COV of 14%. This indicates the feasibility of calculating the shear strength of GFRP ties for PCTLs, which provided the best predictions of shear capacity with ratios ranging between 0.82 and 1.17 for the
Chapter 6: Contribution of Closed Ties to the Shear Strength of GFRP-PCTL Segments

tested segments. CAN/CSA S806-12 overestimated predictions with an average $V_{\text{exp}}/V_{\text{pred}}$ of 0.75± 0.10 due to the high estimated concrete resistance. Conversely, CAN/CSA S6-19 and JSCE (1997) significantly underestimated predictions with an average $V_{\text{exp}}/V_{\text{pred}}$ of 1.49 and 1.45, respectively. This high level of deviation can be attributed to the lower predicted shear strengths of concrete in CAN/CSA S6-19 and stirrups in JSCE (1997). The JSCE equations provided lower strain values for stirrups, hence the lower estimated shear strength. ACI 440.11-22 and AASHTO (2018) yielded conservative predictions with an average $V_{\text{exp}}/V_{\text{pred}}$ of 1.25 ± 0.30 and 1.26 ± 0.23, with COVs of 24% and 18%, respectively.

Table 6. 6 Comparison of the experimental to the predicted results.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_{\text{cf}}^\text{(kN)}$</td>
<td>$V_{\text{exp}}/V_{\text{pred}}$</td>
<td>$V_{\text{cf}}^\text{(kN)}$</td>
<td>$V_{\text{exp}}/V_{\text{pred}}$</td>
<td>$V_{\text{cf}}^\text{(kN)}$</td>
<td>$V_{\text{exp}}/V_{\text{pred}}$</td>
<td>$V_{\text{cf}}^\text{(kN)}$</td>
<td>$V_{\text{exp}}/V_{\text{pred}}$</td>
</tr>
<tr>
<td>G(0.46)/T200</td>
<td>152</td>
<td>1.10</td>
<td>152</td>
<td>1.10</td>
<td>95</td>
<td>1.76</td>
<td>213</td>
</tr>
<tr>
<td>G(0.46)/C200</td>
<td>140</td>
<td>1.31</td>
<td>140</td>
<td>0.96</td>
<td>95</td>
<td>1.12</td>
<td>212</td>
</tr>
<tr>
<td>G(0.86)/C100</td>
<td>182</td>
<td>1.28</td>
<td>182</td>
<td>0.82</td>
<td>127</td>
<td>0.89</td>
<td>225</td>
</tr>
<tr>
<td>G(0.86)/C200</td>
<td>193</td>
<td>1.07</td>
<td>193</td>
<td>0.85</td>
<td>126</td>
<td>1.07</td>
<td>223</td>
</tr>
<tr>
<td>G(0.86)/C200-H</td>
<td>201</td>
<td>1.38</td>
<td>201</td>
<td>1.17</td>
<td>137</td>
<td>1.36</td>
<td>237</td>
</tr>
<tr>
<td>Mean</td>
<td>1.23</td>
<td>0.98</td>
<td>1.25</td>
<td>0.75</td>
<td>1.26</td>
<td>1.49</td>
<td>1.45</td>
</tr>
<tr>
<td>S.D.</td>
<td>0.12</td>
<td>0.14</td>
<td>0.30</td>
<td>0.10</td>
<td>0.23</td>
<td>0.32</td>
<td>0.14</td>
</tr>
<tr>
<td>COV (%)</td>
<td>10</td>
<td>14</td>
<td>24</td>
<td>13</td>
<td>18</td>
<td>21</td>
<td>10</td>
</tr>
</tbody>
</table>

1 Obtained due to the concrete compressive strength of 60 MPa as the maximum limit according to CSA S806-12 and 69 MPa as per ACI 440.11-22 and AASHTO 2018.
Conclusions and Recommendations

The behavior and shear strength of PCTL segments reinforced with GFRP bars and stirrups were investigated in this study. The main findings can be summarized as follows:

1. The observed failure mode of almost all the GFRP-reinforced PCTL segments tested was diagonal tension failure. The closer tie spacing in specimen G_{(0.86)/C100} led to shear compression failure mode, followed by rupture of closed ties. The average theoretical inclination angle of the critical shear crack is 63°, a variation that is uneven and very different that the experimentally observed value (30°).

2. Replacing the transverse bars with closed ties significantly improved PCTL shear behavior (post-peak) and strength, despite the low reinforcement ratio. The presence of stirrups increased the ultimate deflection and normalized shear strength, while decreasing shear-crack width and achieving ductile failure.
3. Compared to segment \( G_{(0.86)/C200} \), the higher shear reinforcement ratio in specimen \( G_{(0.86)/C100} \) increased the normalized shear strength and ultimate stiffness by 16% and 12%, respectively. This is evidence of effective control of the widening and propagation of shear cracks as well as maintaining the confinement of the concrete core in the PCTLs.

4. Increasing the longitudinal reinforcement ratio increased the ultimate stiffness by 90%. The high number of flexural bars, however, had the same effect as using stirrups on the normalized shear strength of the PCTL segment, which increased it by 12%. Based on the shear strength, using stirrups would be a cost-effective alternative solution for tunnels.

5. The increase in concrete strength from 50 to 70 MPa increased the cracking load and initial stiffness, which enhanced the pre-cracking behavior of the segment. In addition, the HSC in segment \( G_{(0.86)/C200-H} \) produced large numbers of narrower cracks.

6. The shear strengths of the tested segments predicted with JSCE (1997) and CSA S6-19 were significantly lower than those measured experimentally, whereas CSA S806-12 overestimated the predicted strengths. The average \( V_{exp}/V_{pred} \) for ACI 440.11-22 and AASHTO (2018) was approximately 1.25, indicating conservative predictions for shear capacity.

7. By ignoring the shear resistance of closed ties, the redefined CSCT (2014) for GFRP-reinforced PCTL segments reasonably predicted actual capacities. The modified CSCT—which considers stirrup strength—yielded the most accurate agreement with experimental results, with an average \( V_{exp}/V_{pred} \) of 0.98 and a COV of 14%.

8. Using a 0.65 length coefficient for deviation forces and an effective stirrup strain limit of 0.004 provided appropriate results with the equations in the modified CSCT. Further experimental research, however, is needed to verify the proposed values. Furthermore, future research could confirm the main results of this study concerning the effectiveness of closed ties for shear resistance in PCTL segments and pave the way to incorporate these findings into tunnel design provisions.
CHAPTER 7
CONCAVE PERFORMANCE EVALUATION OF GFRP-REINFORCED PRECAST CONCRETE TUNNEL LINING SEGMENTS

Foreword

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Journal and Status:


Reference:


Note:

The manuscript had been slightly adjusted from the original paper by remembering the figures and tables to include the chapter number. In addition, the reference list has been moved to the appropriate section in the thesis as indicated in the table of contents.
Abstract

This paper reports on an investigation into the behavior of concave GFRP-reinforced precast concrete tunnel lining (PCTL) segments subjected to transportation, storage, and settlement loads induced by soil settlement underneath tunnels and/or internal vehicular accidents. Shear tests were conducted on four full-scale PCTL segments with a rhomboidal shape measuring 1500 x 250 mm in rectangular cross section and an arched length of 2100 mm. Three main parameters—namely, reinforcement type, concrete strength, and longitudinal reinforcement ratio—were studied under three-point loading until failure. The results reveal that all specimens experienced shear failure due to the diagonal tension mode, even if initiated by the yielding of flexural bars in the steel-reinforced segment. PCTLs reinforced with GFRP or steel bars at the same ratio demonstrated comparable shear strengths and satisfied serviceability limits. The use of both a high reinforcement ratio and high-strength concrete (HSC) increased the shear strength of the GFRP-reinforced PCTL segments. Experimental results were employed to review and verify North American code provisions and existing models with some amendments to meet the requirements of designing tunnel segments reinforced with GFRP bars in terms of deflection at the service state and checking shear strength at the ultimate limit state. Measured deflection was used to conduct a comparison of the experimental values of the effective moment of inertia to predictions with current models to evaluate their capability. Based on the analysis of the results, Bischoff’s equation for the effective moment of inertia of FRP-RC structures was modified, and an equation was developed to predict the deflection of GFRP-reinforced PCTL segments with 98% accuracy. Based on a comparison of experimental and predicted shear capacities, the combination of concrete contributions (including the arch-shape effect of PCTLs and the tie resistance in the modified compression field theory (MCFT), plasticity theory (PT), and modified critical shear crack theory (CSCT)) provided better predictions.

Keywords: tunnel segments; diagonal tension failure; deflection; effective moment of inertia; modified compression field theory; modified critical shear crack theory.
7.1 Introduction

While tunnels have been designed for over 100 years, steel reinforcement corrosion has a significant impact on tunnel service life. Exposure to harsh environmental conditions—such as deicing salts, seawater, stray currents, and chloride ions in contaminated water—accelerate the corrosion of embedded steel bars (Spagnuolo et al. 2018). Tunnel degradation is commonly observed in locations with corroded reinforcement cages, causing spalling of the concrete cover and accelerating catastrophic failure (Zhang et al. 2022). The use of glass fiber-reinforced polymer (GFRP) materials in tunnel-related applications is gaining acceptance, which can be attributed to their high resistance to harsh environmental attacks in addition to their high tensile strength, lightweight, long service life, and resistance to electrical and magnetic currents (ACI 440.11-22; Benmokrane et al. 2021). When all aspects have been taken into consideration, replacing steel bars with GFRP bars is a feasible and cost-effective solution to mitigate corrosion (Caratelli et al. 2016). GFRP reinforcement’s properties are desirable in tunnels due to its ability to create dielectric joints, which, in turn, nullifies stray currents and avoids the resulting corrosion (Hosseini et al. 2022a).

Precast concrete tunnel lining (PCTL) segments are exposed to provisional and permanent loads during tunnel life cycle (ACI 544.7R-16; ACI 533.5R-20; ITA WG2-19). Production, transient, and construction loads induced by tunnel boring machines (TBM) and grouting pressure are represented as provisional loads. Permanent loads include earth or groundwater pressure and surcharges at the service stage. Other loads should be considered based on ground conditions and tunnel circumstances. Such loading is caused by soil settlement underneath tunnels and/or internal vehicular accidents. Shear force imposed on tunnel segments is considered one of the straining actions induced by fabrication loads or settlement loads at the service stage. A few studies in the literature have investigated the flexural and thrust behaviors of GFRP-reinforced PCTL segments with positive curvature (convex) (Caratelli et al. 2016; Spagnuolo et al. 2017; Meda et al. 2019; Hosseini et al. 2022a, 2022b; Ibrahim et al. 2023). All of them revealed that employing GFRP bars in PCTLs enhanced durability and satisfied safety and serviceability requirements. As well, the experimental results demonstrated the efficiency of GFRP reinforcement on the structural behavior of tunnel segments in terms of cracking behavior, deflection, and ultimate strength. Accordingly, this eliminates PCTL segment crushing during handling, stripping, transportation, and storage.
stages by reducing the concrete cover. Abbas et al. (2014) tested full-scale PCTL segments reinforced with steel bars and steel fibers under settlement loads. The authors deduced, however, that both segments failed in flexural mode and exhibited a significantly higher flexural capacity. Until recently, the literature has contained very limited information about the shear strength of GFRP-reinforced PCTL segments.

Hosseini et al. (2022b) focused solely on studying the shear strength of GFRP-reinforced PCTL segments, but the loading differed from that induced by fabrication and settlement loads. The test was also different. They investigated the effect of reinforcement type, longitudinal reinforcement ratio, and concrete strength on shear capacity. They concluded that using GFRP bars in PCTLs increased the ultimate capacity compared to a steel-reinforced segment with the same number of bars and decreased deflection at service load. The use of HSC in tunnel segments increased cracking, ultimate strength, and flexural stiffness. The high longitudinal reinforcement ratio changed the failure mode from flexural to a combination of flexure and shear; it also enhanced the shear capacity. The authors further determined the theoretical shear strength, although they neglected the arch-shape effect and the tie contribution. The shear equations in North American standards for FRP-RC structures (ACI 440.11-22; CSA S806-12) and the modified compression field theory (MCFT) do not consider the effect of the curved geometry of PCTL segments in the concrete’s contribution to shear strength (CSA S806-12). Based on the critical shear crack theory (CSCT), Campana (Campana et al. 2014) proposed an approach to address the deviation forces along the curved length of steel-reinforced tunnels. Based on plasticity theory (PT), Kragh-Poulsen (2020) proposed a new model to predict the shear strength of curved-shape steel-reinforced beams. Furthermore, some current shear models (Nehdi et al. 2020; Ali et al. 2021) do not account for the predicted resistance of shear reinforcement. All models still need some modifications to be adequate for predicting the shear strength of arched tunnel segments reinforced with GFRP bars and ties. Comparing the actual capacities and predictions of modified models is imperative to verifying model accuracy. Indeed, the shear strength of concave PCTLs reinforced with GFRP bars and the effects of different parameters have not yet been investigated. The current study presents the first experimental data on the shear behavior of negative arch-shaped GFRP-reinforced PCTL segments and evaluates their capacity through theoretical predictions.
Deflection control is one of the primary considerations in designing FRP-RC structures based on fulfilling serviceability limitations. Under field loads, doing so is more critical than the ultimate limit state because the magnitude of deflections in precast tunnel segments reinforced with GFRP bars is greater compared to steel reinforcement due to their lower elastic modulus (Gouda et al. 2022; Mousa et al. 2019). An elastic–deflection equation is often used to calculate the deflections of flexural members, along with an effective moment of inertia ($I_e$) as introduced in Branson’s equation (Branson 1965) in steel design codes. Several authors have proposed (Benmokrane et al. 1996; Yost et al. 2003; Adam et al. 2015) coefficients to modify this equation to make it applicable to FRP-RC structures. Moreover, the design manual published by ISIS Canada provides a new equation (ISIS 2007). Other researchers have proposed a new expression to account for the tension stiffening effect on curvatures (Bischoff 2007, 2011). The models with different approaches have been adopted in current FRP design guidelines (ACI 440.11-22; CSA S806-12). These models were developed based on experimental research on non-curvature beams, yet they failed to address the effect of an arch-shaped member on the applied bending moment. An alternative equation is required to consider the geometry of GFRP-reinforced PCTL segments in calculating the effective moment of inertia, and simultaneously, an equation is required to predict deflection with reasonable accuracy. This paper reports on a study aimed at reviewing and verifying the accuracy of deflection equations using the modified equation and those available in North American standards (ACI 440.11-22; CSA S806-12). The work described herein is part of an ongoing extensive research project conducted on PCTL segments under different loading conditions such as flexural static (Hosseini et al 2022a, 2022b), quasi-static cyclic flexural (Ibrahim et al. 2023), punching, shear, settlement, and thrust loads at the University of Sherbrooke. Effects of reinforcement type, flexural reinforcement ratio, and concrete strength on the deflection behavior and shear strength of concave GFRP-reinforced PCTL segments were investigated.
7.2 Laboratory Test Program

7.2.1 Test Specimens and Reinforcement Details

This study involved a laboratory experiment to assess the shear strength of precast tunnel segments in field applications. Four real-scale PCTL segments with closed ties were fabricated, cast, and tested under a monotonically increasing line load. A typical metro tunnel comprises full parallel rings with seven different segment geometries for each ring. Herein, the segments were rhomboid in shape with internal and external diameters of 6500 mm and 7000 mm, respectively. All the segments measured 2100 mm in length, 1500 mm in width, and 250 mm in thickness. Segments had a constant shear span-to-effective depth ratio of 3.6 and a clear concrete cover of 40 mm. Reinforcement type (GFRP or steel), flexural reinforcement ratio, and concrete strength have a considerable effect on shear resistance (Mahmoud and El-Salakawy, 2014; Kaszubska et al. 2017; Mehany et al. 2023). Therefore, the experimental program comprised three PCTL segments reinforced with GFRP bars and ties, while the fourth segment was reinforced with steel for comparison. Both the top and bottom meshes of two of the GFRP-reinforced PCLTs had a flexural reinforcement ratio of 0.46% (7 No. 5 at 250 mm spacing). Another segment had a reinforcement ratio of 0.86% (13 No. 5 at 125 mm spacing). Two of the three segments with the low reinforcement ratio were fabricated with normal-strength concrete; one was made with high-strength concrete. The GFRP-reinforced specimens were had No. 4 closed ties at a pitch of 200 mm. The control segment was reinforced longitudinally with 7 M15 steel bars (0.47%) and transversely with M10 steel ties at a pitch of 200 mm. To ensure that desirable shear failure occurred with a safety margin against flexural failure, all the GFRP segments were designed with a flexural reinforcement ratio greater than the balanced ratio, while it was lower for the steel-reinforced segment (ACI 318-19; ACI 440.11-22). Table 7.1 provides the details of the segments and test parameters. Each segment was identified with a label consisting of a letter and a number. The letter indicates the type of reinforcement (G for GFRP and S for steel longitudinal bars and stirrups). The number denotes the percentage of the flexural reinforcement ratio. The high-strength concrete segment bears the letter H. Figure 7.1 shows the geometric configurations and reinforcement details of the tested specimens.
### Table 7. Details of test specimens.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Reinf. Type</th>
<th>Dimensions (mm)</th>
<th>Longitudinal Reinforcement</th>
<th>Concrete Type</th>
<th>Transverse Reinforcement</th>
<th>$f_{c}'$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G-0.46</td>
<td>GFRP</td>
<td>2500 x 1500 x 250</td>
<td>7 No. 5, 0.46, 0.33</td>
<td>NSC</td>
<td>No. 4 @200 mm</td>
<td>51.4</td>
</tr>
<tr>
<td>G-0.86</td>
<td>GFRP</td>
<td>7 No. 5, 0.46, 0.34</td>
<td>NSC</td>
<td>No. 4 @200 mm</td>
<td>48.8</td>
<td></td>
</tr>
<tr>
<td>G-0.46-H</td>
<td>Steel</td>
<td>7 No. 5, 0.46, 0.43</td>
<td>NSC</td>
<td>No. 4 @200 mm</td>
<td>70.3</td>
<td></td>
</tr>
<tr>
<td>S-0.46</td>
<td>Steel</td>
<td>7-15M, 0.47, 3.32</td>
<td>NSC</td>
<td>No. 4 @200 mm</td>
<td>48.3</td>
<td></td>
</tr>
</tbody>
</table>

1 $\rho_b$ calculated according to ACI 440.11-22 and ACI 318-19 for GFRP and steel-reinforced concrete slabs, respectively.

2 $f_{c}'$ based on six $100 \times 200$ mm cylinder tests.

### 7.2.2 Material

#### 7.2.2.1 Reinforcement

Newly developed curvilinear bars (Pultrall 2019) and closed ties were used to reinforce the GFRP-reinforced PCTL segments in the longitudinal and transverse directions, respectively (Fig. 7.2(a)). The GFRP reinforcement was manufactured using pultrusion and an inline sand-coating process to improve the bonding between the bars and the surrounding concrete. Number 5 (15 mm) curvilinear GFRP bars were fabricated with an arc length of 2044 mm for the extrados mesh and...
1956 mm for the intrados. The tensile strength, modulus of elasticity, and ultimate tensile strain of the longitudinal bars were calculated according to ASTM D7205-21. These bars were anchored with No. 5 (15 mm) U-shaped closing bars to ensure proper end anchorage. Number 4 (13 mm) closed GFRP ties served as transverse reinforcement, with a bent tensile strength of 518 MPa (AASHTO 2018). Steel bars were used to reinforce the control segment; deformed 15M (16 mm) curvilinear steel bars and 10M (11.3 mm) closed steel ties were provided as longitudinal and transverse reinforcement, respectively. Table 7.2 provides the mechanical properties of the GFRP and steel reinforcement based on nominal cross-sectional area.

Table 7.2 Properties of GFRP and steel reinforcement.

<table>
<thead>
<tr>
<th>Reinforcement Type</th>
<th>Reinforcement Shape</th>
<th>Bar Size</th>
<th>$d_b$ (mm)</th>
<th>$A_f$ (mm$^2$)</th>
<th>$E_f^2$ (GPa)</th>
<th>$f_y^2$ (MPa)</th>
<th>$\varepsilon_f^2$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GFRP</td>
<td>Curvilinear bars¹</td>
<td>#5</td>
<td>15.0</td>
<td>199</td>
<td>55.1 ± 1.25</td>
<td>1115 ± 60</td>
<td>2.0 ± 0.1</td>
</tr>
<tr>
<td></td>
<td>Closed stirrups</td>
<td>#4</td>
<td>13.0</td>
<td>129</td>
<td>55.6 ± 1.6</td>
<td>1248 ± 74</td>
<td>2.2 ± 0.1</td>
</tr>
<tr>
<td></td>
<td>U-shaped closing bars</td>
<td>#5</td>
<td>15.0</td>
<td>199</td>
<td>53.5 ± 1.1</td>
<td>1283 ± 42</td>
<td>2.4 ± 0.1</td>
</tr>
<tr>
<td>Steel</td>
<td>Curvilinear bars</td>
<td>15 M</td>
<td>16.0</td>
<td>200</td>
<td>200.0</td>
<td>$f_y^2$ = 480 ± 15</td>
<td>$\varepsilon_y^2$ = 0.24</td>
</tr>
<tr>
<td></td>
<td>Closed stirrups</td>
<td>10 M</td>
<td>11.3</td>
<td>100</td>
<td>200.0</td>
<td>$f_y^2$ = 480 ± 10</td>
<td>$\varepsilon_y^2$ = 0.24</td>
</tr>
<tr>
<td></td>
<td>U-shaped closing bars</td>
<td>15 M</td>
<td>16.0</td>
<td>200</td>
<td>200.0</td>
<td>$f_y^2$ = 480 ± 15</td>
<td>$\varepsilon_y^2$ = 0.24</td>
</tr>
</tbody>
</table>

¹ Straightened through the test fixture with a novel method and then directly subjected to tensile load (Hosseini et al. 2022).
² The reported properties were calculated according to ASTM D7205 (ASTM 2021).
³ $f_y$ and $\varepsilon_y$ are the yield strength and strain of the steel bars, respectively.

7.2.2.2 Concrete

Three PCTL segments were cast with normal-strength concrete (NSC), while the fourth segment was cast with high-strength concrete (HSC). Table 7.3 provides the concrete mix proportions. The target 28-day compressive strength was 40 MPa for the NSC segments and 70 MPa for the HSC segment. The concrete compressive strength for each specimen was determined based on the average test results of six concrete cylinders measuring 100 x 200 mm tested on the same day of testing. The actual compressive strengths ($f'_c$) for G-0.46, G-0.86, and S-0.47 (NSC specimens) were 51.4, 48.8, and 48.3 MPa, respectively. Segment G-0.46-H (HSC) had an actual compressive strength of 70.3 MPa.
Table 7.3 Concrete mix design.

<table>
<thead>
<tr>
<th>Concrete Type</th>
<th>Cement (kg/m³)</th>
<th>Sand (kg/m³)</th>
<th>Limestone (kg/m³)</th>
<th>Superplasticizer (mL/m³)</th>
<th>Air-entrainment (mL/m³)</th>
<th>Water (L/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NSC</td>
<td>450</td>
<td>615</td>
<td>1015</td>
<td>4500</td>
<td>140</td>
<td>170</td>
</tr>
<tr>
<td>HSC</td>
<td>475</td>
<td>778</td>
<td>800</td>
<td>7000</td>
<td>170</td>
<td>135</td>
</tr>
</tbody>
</table>

7.2.3 Segment Production

The GFRP and steel cages were assembled in various configurations at the University of Sherbrooke (Fig. 7.2(a)). The segments were shipped to a precast plant (Sym-Tech) in Saint-Hyacinthe (QC, Canada) for fabrication. The cages were instrumented and placed into curved wooden formwork, as shown in Fig. 7.2(b). The concrete was cast into the specimen mold from an overhead crane bucket (Fig. 7.2(c)). To ensure consistency, the segment was compacted with an electrical vibrator and manually leveled. The formwork was covered with a plastic sheet to eliminate the loss of vapor and moisture. The segments were stored in the storage yard after curing for seven days (Fig. 7.2(d)).

Figure 7.2 (a) typical assembled GFRP cage, (b) cage of 7 bars inside formwork, (c) casting concrete, and (d) segments.
7.2.4 Test Procedure and Instrumentation

The PCTL segments were tested upward under three-point loading over a simply supported center-to-center span of 1400 mm. The load was applied monotonically with an MTS 1000 kN actuator attached to a spreader beam at a stroke-controlled rate of 0.5 mm/min, as shown in Fig. 7.3. Mid-span loads on the interior surfaces of PCTL segments simulate fabrication and settlement loads in tunnels (Abbas et al. 2014). Rubber sheets were placed between the beam and the tested segment to ensure smooth and uniform load distribution. Additionally, Teflon-covered steel cylinder supports were used to ease rotation and movement. Strain in the reinforcement was measured with an electric strain gauge with a gauge length of 10 mm. Nine gauges were mounted on the external (LE1 to LE5) and internal (LI1 to LI3) longitudinal bars at 0, 200, and 400 mm from the segment’s centerline (Fig. 7.1). Five concrete strain gauges (C1 to C5) with a gauge length of 60 mm were installed at mid- and quarter-spans to measure compressive strains. Segment deflection was recorded by five linear potentiometers (LPOTs), three distributed along the central width and two at the quarter-span. Three high-accuracy LVDTs were installed after crack widths were measured with an Elcometer crack-width ruler. The experiment was video recorded, and measurements were collected with a data-acquisition system.
7.3 Results and Observations

Table 7.4 provides a summary of the general behavior of the GFRP- and steel-reinforced PCTL segments in terms of flexural and shear cracks, ultimate capacity, deflection, maximum strain in the concrete and reinforcement, and the failure mode under shear loads.

7.3.1 Cracking Behavior

The test history shows that all the tested PCTL segments exhibited similar crack propagation features. Figure 7.4 shows cracking patterns and loads (kN) at different stages. Generally, the flexural cracks occurred first at the extreme tension fiber on the extrados surface directly under the loading zone (maximum bending). These cracks developed continuously along the segment width and propagated vertically upward. In the GFRP-reinforced PCTLs, the cracks initiated at loads of 124, 165, and 170 kN for segments G-0.46, G-0.86, and G-0.46-H, respectively. Flexural cracks represented 23% to 29% of the peak load, while the depth ranged from 120 to 180 mm. Conversely,
the first flexural crack in the control segment (S-0.47) occurred at a load level of 110 kN (20% of the peak load) with a depth of 75 mm. With further loading, flexural stresses were dominant, and cracks formed inside and outside the maximum bending area. All cracks developed perpendicular to the extrados surface and extended toward the neutral axis, while the existing ones widened beneath the segments. These cracks initiated at lower loads—in addition to the number of flexural cracks that increased and developed near supports—due to the negative arch-shape effect of the PCTLs (Kragh-Poulsen 2020). Average flexural crack spacing at the mid-height of segments was 215, 155, 210, and 150 mm for G-0.46, G-0.86, G-0.46-H, and S-0.47, respectively. The flexural cracks stabilized at up to 58% to 68% of the failure load. The presence of shear stresses in the GFRP-reinforced PCTL segments caused vertical cracks to incline toward the line load as flexural–shear cracks. Specimens G-0.46, G-0.86, and G-0.46-H had shear cracks at loads of 370, 361, and 340 kN, respectively. Segment S-0.47, however, experienced inclined shear cracks at a load of 210 kN, representing 38% of the peak load. The shear crack intensity increased gradually with load increments, whereas the crack spacing decreased. As a result, major shear cracks developed at a load level of 95% to 98% of the segments’ failure load. At failure loads, crack widths increased, and critical shear cracks reached line loading.

Tunnels can undergo structural and esthetic damage due to crack widths during their life cycle (ITA WG2-19). Thus, the crack opening was recorded at the loading phases for the tested segments. The GFRP-reinforced PCTL segments had maximum crack widths recorded at a corresponding peak load equal to 2.31, 1.75, and 1.97 mm for G-0.46, G-0.86, and G-0.46-H, respectively. In the literature, the reference load has been determined at a strain equal to 2000 \(\mu\varepsilon\) in the flexural bars (Mousa et al. 2018). FRP design provisions limit crack openings to 0.5 and 0.7 mm for exterior and interior exposures, respectively (CSA S806-12). Considering that the design service limit for steel is 0.6 \(f_y\), the reference load for the control segment (S-0.47) was identified at a strain equal to 1200 \(\mu\varepsilon\) with a corresponding crack width of 0.3 mm. At 237 kN (the reference load in S-0.47), the recorded crack width was 0.27 mm. The crack opening was, however, 0.82 mm when the steel bars yielded, and it progressively increased until failure (Fig. 7.4). At reference loads, the crack widths for the segments reinforced with GFRP bars were in the range of 0.2 to 0.49 mm, which is narrower than the maximum limits. Based on the monitored cracks, the steel-reinforced segment showed catastrophic damage compared to PCTLs reinforced with GFRP bars.
under shear loads. Consequently, using GFRP bars in tunnel segments is considered a practical solution with respect to serviceability and esthetic considerations.
Figure 7.4 Observed crack pattern propagation at different load stages.
7.3.2 Shear Capacity and Failure Mechanism

Typical failure mechanisms for the tested specimens were shear, as listed in Table 7.4. All the GFRP-reinforced PCTL segments had idealized diagonal tension failure (DT) at different shear loads. A sudden drop in load capacity occurred when the major inclined cracks propagated to the concrete fibers on the intrados surface. The concrete cover spalled along the major diagonal crack, particularly observable in segment G-0.46-H. This could be attributed to the crack passing through the aggregate in the HSC segment, resulting in smooth surfaces and reduced interlock. The specimens failed at ultimate forces equal to 541, 560, and 591 kN for G-0.46, G-0.86, and G-0.46-H, respectively. Despite the yielding of the flexural steel bars in the control segment, S-0.47 failed in diagonal tension mode (Y-DT) with an ultimate load of 559 kN. The negative curvature increased the total shear capacity due to the additional shear force induced by the inclined tension force in the curved reinforcement (Campana et al. 2014). It is worth noting that, for all specimens, the critical shear cracks extended, avoided intersection with the transverse reinforcement, and grew horizontally through the upper concrete cover. This could be attributed to the closed ties enhancing the confinement of the concrete and the longitudinal curvilinear bars (Jumaa and Yousif 2019). Figure 7.5 depicts photographs of the thrust and ring sides of the tested PCTL segments in the final state. The figure also shows the shear failure plane and highlights its inclination angles.

According to Canadian standards for FRP and steel structures (CSA S6-19; CSA-A.23.3-19), the theoretical inclination of critical shear cracks was calculated based on strain in the longitudinal bars in the segments as follows:

\[
\theta = 29 + 7000\varepsilon_x
\]  

(7.1)

Experimentally, the major shear crack angles at failure were in the range of 35° to 48°. Specimen G-0.46 had a flatter inclination angle than the control specimen, which is consistent with past findings (Alam and Hussein 2013; Ali et al. 2017). The theoretical angle of the critical shear crack was 46° for S-0.47, whereas it was significantly steeper for the GFRP-reinforced segments. Indeed, there were uneven and highly separated variations between the experimental and predicted results. This was due to shear transfer mechanisms in the arched members, bond strength, and concrete and reinforcement properties. Due to the negative curvature, the major shear cracks (yield lines)
started closer to the supports (Kragh-Poulsen 2020). The effective shear-to-depth ratio for the tested segments was 3.6; however, the shear force was mainly resisted by arch action.

Table 7.4 Summary of test results.

| Specimen ID | $\rho(E_f/E_s)$ (%) | Flexural Cracking Load, $P_{cr}$ (kN) | Critical Shear Crack Load, $2V_{exp}$ (kN) | Angle, $\theta$ (deg.) | Ultimate Load, $P_u$ (kN) | Ultimate Deflection, $\Delta_u$ (mm) | Normalized Shear Strength, $V_{nor}$ (MPa) | Maximum Strain at failure, $\varepsilon_{max}$ (µs) | External Bars | Internal Bars | Concrete | Failure mode $^2$ |
|-------------|---------------------|-------------------------------------|------------------------------------------|----------------------|------------------------|----------------------------|-------------------------------------|------------------|----------------|-----------|-----------------|
| G-0.46      | 0.13                | 124                                 | 520                                      | 42                   | 541                    | 17.46                     | 0.25                  | 9300             | 4255          | -2265              | DT                 |
| G-0.86      | 0.24                | 165                                 | 530                                      | 39                   | 560                    | 12.21                     | 0.27                  | 7230             | 2295          | -1330              | DT                 |
| G-0.46-H    | 0.24                | 170                                 | 579                                      | 35                   | 591                    | 15.76                     | 0.25                  | 9730             | 4765          | -1005              | DT                 |
| S-0.46      | 0.47                | 110                                 | 532                                      | 48                   | 559                    | 43.32                     | 0.27                  | 8975             | 4450          | -2065              | Y-DT               |

1 Denotes the effective reinforcement ratio.
2 DT is diagonal tension failure and Y-DT is steel yielding followed by diagonal tension failure.

Figure 7.5 Failure modes of tested segments.
7.3.3 Deflection

Figure 7.6 plots the shear load versus the maximum deflection at both the center of the segments and shear span. LPOTs at quarter-spans measured coincident deflection, indicating that the deformation was uniformly distributed. Additionally, there was no evidence of torsion based on other LPOTs located along the mid-span. Segments had almost similar load–deflection relationships at the two locations. The GFRP-reinforced specimens showed a slight increase in central deformation capacity. Specimen S-0.47, however, had 2.2 times the ultimate deflection of the LPOTs at quarter-span due to concentrated settlement under point load. This could explain the reason for widened cracks in the segment reinforced with steel bars in the load vicinity. Consequently, the influence of a negative arch shape on crack width and associated deflection was greater in the PCTL segment reinforced with steel bars than in those reinforced with GFRP bars. Prior to the onset of the flexural crack, all segments displayed similar stiff and linear behavior, while deflection values were minimal. Using HSC in segment G-0.46-H enhanced the pre-cracking behavior and increased stiffness due to the properties of the uncracked section (Al-Hamrani and Alnahhal 2022). The section characteristics were changed from gross to effective with load increment due to the increased number, width, and depth of cracks. Segments G-0.46, G-0.86, and G-0.46-H behaved linearly until failure due to the linear elastic behavior of the GFRP bars, with a significant reduction in flexural stiffness. The corresponding ultimate deflections were 17.46, 12.21, and 15.76 mm, respectively. Segment S-0.47 exhibited a steep increase in post-cracking stiffness until the steel bars yielded at a deflection of 6.06 mm. When failure occurred, a long plateau of nonlinear behavior was observed as the deformation capacity increased. All segments experienced an instant drop in load due to brittle failure, despite the deflection continuing to increase.

At reference loads (2000 µε), the measured deflections were 3.64, 2.54, 3.54, and 4.04 mm for G-0.46, G-0.86, G-0.46-H, and S-0.47, respectively. Tunnel segments must be designed with appropriate stiffness to prevent large deflections that adversely affect strength and cause damage at service loads. Design standards and guidelines limit deflection from $L/180$ to $L/480$ ($L$ is the span), depending on the type and purpose of the structure (ACI 318-19; ACI 440.11-22). The allowable deflection is 2.91 mm, as supporting elements are likely to be damaged by large deflections. Conforti et al. (2019) assumed the bending moment of 16.8 kN.m (corresponding to a
load of 48 kN) was the service load to guarantee worker safety in exceptional conditions. The measured deflections were in the range of 0.34–0.54 mm, which is significantly lower than the allowable limit. Furthermore, Abbas (Abbas et al. 2014) considered that the point of initial cracking (110 kN) was the ultimate design load of PCTL segments. Fabrication and installation processes can induce this load, which affects structural integrity. At 110 kN, the reported deflections were 64%, 72%, 78%, and 32% smaller than the allowable deflections for G-0.46, G-0.86, G-0.46-H, and S-0.47, respectively.

Figure 7.6 Load-deflection relationships at mid and quarter-span.

7.3.4 Strain in Longitudinal Bars

Figure 7.7(a) depicts the relationship between shear load and strain in the internal and external longitudinal bars. Flexural strains were recorded in the load vicinity (maximum bending) for both the GFRP- and steel-reinforced PCTL segments. The graph shows that either strain in the internal bar or tensile strain in the external bar did not exceed 90 $\mu$e up to the service and design loads because no cracks were formed. Once segment cracking initiated, the strain in the internal bars converted to tension. This implies that the neutral-axis depth was lower than that of the top bar. The segments reinforced with GFRP bars had linear load–strain responses after flexural cracking until failure. Gauges on the internal bar of G-0.46 recorded readings at a high load level due to crack locations. The maximum measured strains in the external bars were 9300, 7230, and 9730 $\mu$e for segments G-0.46, G-0.86, and G-0.46-H, respectively, representing 46%, 37%, and 49% of the ultimate strain. The corresponding ultimate strains in the internal bars were 4225, 2295, and 4765 $\mu$e, respectively. Due to significantly widening cracks on the bottom of the control segment, the external bars produced a steeper behavior and yielded at a load equal to 62% of the peak load.
Therefore, the strain had increased rapidly and reached 8975 \( \mu \varepsilon \), coinciding with a constant load level close to failure until the gauge malfunctioned.

Figure 7.7(b) provides strain profiles for the longitudinal reinforcement along the tested span at the ultimate loads. The internal bars exhibited tensile strains along the span. This confirms that the neutral axis was above the internal mesh level in almost all cross sections due to the negative curvature of segments. In addition, the closed ties slightly affected the confinement of the longitudinal bars due to their large spacing. The recorded strains in the GFRP-reinforced PCTL segments were typically high under the applied load and decreased proportionally toward the segment supports until they reached zero. This implies that the GFRP bars enhanced load transfer without signs of bar slippage or bonding degradation. In contrast, segment S-0.47 had fluctuations in bar-strain distribution along the span. A large increase in crack opening after steel yielding could result in localized strains at cracks.
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(a) Load versus strain

(b) Strain distribution along longitudinal bars at failure.

Figure 7.7 Reinforcement strain relationships.
7.3.5 Concrete Strain

Figure 7.8 gives the concrete compressive strains measured at mid- and quarter-spans on the intrados surface of the specimens. The figure implies that the strain values at the quarter span were negligible during the test history, with a range of -250 to -350 \( \mu e \) at failure. The tested PCTL segments had almost bilinear behavior at mid-span until the maximum load was reached. The concrete strains in segments G-0.46, G-0.86, G-0.46-H, and S-0.47 were -2265, -1300, -1005, and -2065 \( \mu e \), respectively. These strains were significantly lower than the theoretical crushing failure of 3000 \( \mu e \) specified in ACI 440.11-22 and ACI 318-19. No specimens showed signs of concrete crushing. The strain values are therefore consistent with crack propagation and the related diagonal shear failure mode. The ratios between the experimental and ACI 440.11-12 flexural capacity, \( M_{\text{exp}}/M_{\text{theo}} \) ranged from 0.6 to 0.8 for the GFRP-reinforced PCTLs. As expected, these results are consistent with the maximum measured strains, which, in turn, led to shear failure. This is in good agreement with the work of (Al-Hamrani et al. 2021; Al-Hamrani and Alnahhal 2022) on the shear strength of FRP-reinforced beams and one-way slabs.

Figure 7. 8 Applied load versus concrete strain at mid- and quarter span.
7.4 Discussion

7.4.1 Effect of the Reinforcement Type

Specimens G-0.46 and S-0.47 were designed to have the same number of longitudinal bars to study the influence of reinforcement type on PCTL segment behavior under shear loads. Regardless of the axial stiffness, both segments had similar crack pattern propagations, and their widths satisfied the serviceability requirements. Segment G-0.46 initiated flexure at loads 13% greater than that of S-0.47 due to its higher concrete compressive strength. In S-0.47, however, the cracks width increased significantly at yielding and peak loads. This, in turn, affected the failure mechanism, even though the segments failed in shear mode due to diagonal shear cracks. The shear capacity was normalized to the cubic root of the concrete strength \( \left( V_{\text{exp}} / \left( h_n d f_c \right) \right) \) to minimize the influence of its variation and maintain consistency with the shear design equations in FRP standards (CSA S806-12; JSCE 1997). For comparison, effective reinforcement ratios \( (\rho E_f / E_s) \) were used to account for the difference in elasticity modulus (Table 7.4). The normalized shear strength of G-0.46 was 7% lower than that of its steel counterpart, as shown in Fig. 7.9(a). This result indicates that substituting GFRP bars for steel bars in PCTL segments while maintaining a 27% effective reinforcement ratio could yield very close shear capacities. The ultimate stiffness for segment S-0.47 was 61 kN/mm, which is greater than its GFRP counterpart. This can be attributed to steel reinforcement having elasticity moduli 3.6 times higher than the GFRP bars. Lastly, the measured deflections for both segments were lower than the maximum limit at the service and design stages.

7.4.2 Effect of the Longitudinal Reinforcement Ratio

Increasing the longitudinal reinforcement ratio from 0.46% to 0.86% (7 to 13 No. 5 bars) in segment G-0.86 considerably affected the shear capacity and failure angle, deformation, and cracking behavior. The flexural cracks in G-0.86 initiated at a load 25% higher than in segment G-0.46, while their spacing at mid-height decreased by 46%. The high reinforcement ratio had a negligible effect on the first and major shear crack loads. Both segments failed with the same diagonal tension mechanism, but G-0.86 had a flatter shear failure plane, consistent with past studies (Ali et al. 2017; El Refai and Abed 2016). This could be attributed to the angle of shear failure being a function of longitudinal reinforcement strain. Figure 7.9(b) confirms that increasing
the reinforcement ratio to 86% from G-0.46 to G-0.86 increased the normalized shear strength by 7%. A higher reinforcement ratio led to an increase in uncracked concrete depth, which subsequently reduced strains and crack openings while increasing the shear strength.

### 7.4.3 Effect of the Concrete Strength

Figure 7.9(c) shows the experimental shear capacity ($P_u/2$) versus deflection for segments G-0.46 and G-0.46-H to investigate the effect of concrete compressive strength. Using HSC enhanced the initial stiffness of G-0.46-H and increased the cracking load by 37%. This could be attributed to its superior resistance to flexural strength at the uncracked stage. Specimen G-0.46-H, analytically (ACI 318-19), had a 15% higher ultimate stiffness, which is comparable to the experimental result. Compared to the NSC segment, increasing the concrete strength from 51.5 to 70.3 MPa for G-0.46-H increased its ultimate shear capacity by 9% with steeper post-cracking stiffness. The use of HSC increased the ratio between the critical shear crack load and ultimate load. The ultimate concrete strain in NSC exceeded G-0.46-H by 70% because the ultimate strain of concrete is inversely proportional to its compressive strength (ACI 363-10).
Figure 7.9 Effect of test parameters: (a) longitudinal reinforcement type, (b) reinforcement ratio, and (c) concrete strength.

### 7.5 Analytical Deflection

#### 7.5.1 Deflection Prediction Models for FRP-RC Members

For a simply supported beam or one-way slab subjected to three-point loads with a concentrated load \( P \) placed at the center of the span \( L \), the maximum flexural deflection \( \Delta_{\text{max}} \) can be expressed based on linear elastic analysis as follows:

\[
\Delta_{\text{max}} = \frac{PL^3}{48EI_c}
\]  

(7.2)
The effective moment of inertia \( (I_e) \) was derived from several models in the literature (Branson 1965; Benmokrane et al. 1996; Yost et al. 2003; Adam et al. 2015; ISIS 2007; Bischoff 2007, 2011) to predict the deflection of FRP-reinforced concrete structures. Concrete sections have two stages: uncracked and cracked. The applied moment \( (M_a) \) is lower than the cracking moment \( (M_{cr}) \) before cracking \( (M_a \leq M_{cr}) \), resulting in an upper limit of inertia at which the non-cracked section has elastic behavior due to gross inertia \( (I_g) \). The lower limit of inertia refers to the fully cracked transformed section \( (M_a \geq M_{cr}) \), which is also known as the cracked moment of inertia \( (I_{cr}) \). The effective moment of inertia \( (I_e) \) is introduced as a transition between the two limits to determine the deflection of flexural members after cracking.

Two approaches have been used to calculate flexural deflection in most deflection models: Branson’s equation and interpolation between a fully cracked and an uncracked state of a deformation parameter (deflection or curvature). The methodology of these approaches has been modified and adopted in the predictive equations of the FRP design guidelines (ACI 440.11-22; CSA S806-12). Table 7.5 provides a concise summary of these models. Experimentally, applied loads \( (P_{exp}) \) and the corresponding deflection measured \( (\Delta_{exp}) \) at mid-span were introduced in Eq. (7.2) to obtain the effective moment of inertia \( (I_{exp}) \) as follows:

\[
I_{exp} = \frac{P_{exp}L^2}{48E_{exp}\Delta_{exp}}
\]  

Figure 7.10 shows comparisons between the experimental moment of inertia, which was calculated using the measured deflection data of GFRP-reinforced PCTL segments, and those predicted using the existing models. The figure reveals a noticeable discrepancy between the experimental results and Branson’s equation (Branson 1965) for the three PCTL segments, along with inaccurate predictions of inertia of fully cracked transformed sections \( (I_{cr}) \). Incorporating empirical correction factors of Benmokrane et al. (1996) into Branson’s equation, as well as the effect of reinforcement ratio to balanced reinforcement ratio by Yost et al. (2003), yielded a reasonable estimate of computed \( I_e \). In addition, the new equation developed by ISIS (2007) provided better predictions. ACI 440.11-22 yielded the most accurate prediction by considering the change in stiffness along the length and recommending \( 0.8M_{cr} \) as a safety factor in Bischoff’s model (Bischoff et al. 2011). The experimental \( I_{exp} \) was lower than that produced by all the prediction models at the post-cracking stage and corresponded to the experimental observations.
<table>
<thead>
<tr>
<th>Equation</th>
<th>Remarks</th>
</tr>
</thead>
</table>
| Branson (1965)  
\[ I_e = \left[ \frac{M_{cr}}{M_a} \right]^3 I_a + \left[ 1 - \left( \frac{M_{cr}}{M_a} \right)^3 \right] I_{cr} \]  
Basic equation to predict the deflection of steel reinforced concrete. |
| Benmokrane et al. (1996)  
\[ I_e = \left[ \frac{M_{cr}}{M_a} \right]^3 I_a / \beta + \left[ 1 - \left( \frac{M_{cr}}{M_a} \right)^3 \right] \alpha I_{cr} \]  
Where, \( \beta = 0.84 \) and \( \alpha = 7 \)  
First integrated constant correction factors in Branson’s equation, through a comprehensive experiment on GFRP-RC beams to reduce tension stiffening. |
| Yost et al. (2003)  
\[ I_e = \left[ \frac{M_{cr}}{M_a} \right]^3 I_a \beta_a + \left[ 1 - \left( \frac{M_{cr}}{M_a} \right)^3 \right] I_{cr} \]  
\[ \beta_a = \alpha_b \left( \frac{E_f}{E_s} + 1 \right) \]  
\[ \alpha_b = 0.064(\rho_f / \rho_s) + 0.13 \]  
Proposed factor to account for the ratio between flexural reinforcement ratio to balanced reinforcement ratio. |
| ISIS (2007)  
\[ I_e = I_g I_{cr} / \left[ I_{cr} + (1 - 0.5(M_{cr} / M_a)^2) I_g - I_{cr} \right] \]  
Developed new equation based on CEB-FIP Model code 90. |
| Bischoff (2007)  
\[ I_e = I_g I_{cr} / \left[ 1 - (I_{cr} / I_g)(M_{cr} / M_a)^2 \right] \]  
Proposed new expression account for the tension stiffening effect on curvatures. |
| ACI 440.11-22  
\[ I_e = I_{cr} / \left[ 1 - \gamma \left( 1 - \frac{I_{cr}}{I_g} \right) (0.8M_{cr} / M_a)^2 \right] \]  
\[ \gamma = 1.72 - 0.72(0.8M_{cr} / M_a) \]  
Modified Bischoff’s expression to account for the variation in stiffness and \( M_{cr} \) multiplied by reduction safety factor to consider the temperature and shrinkage cracks. |
| CSA S806-12  
\[ \Delta_{max} = \frac{PL^3}{48E_s I_{cr}} \left[ 1 - 8(1 - I_{cr} / I_g)(L_g / L)^3 \right] \]  
Where \( L_g \) is calculated in case of \( M_{cr} = M_a \)  
Recommended derived equation based on the integration of curvature along the span for three-point loading. |
7.5.2 Modified Bischoff Equation for PCTL Segments

Given the experimental $I_{\text{exp}}$, Bischoff’s equation was chosen due to its accuracy and is considered the most adequate model (Mousa et al., 2019). In this section, the ACI 440.11-12 approach has been modified and rewritten—based on Bischoff and Gross (2011)—to account for the actual response of the tested curved specimens. The arch-shape effect has been incorporated into the modified equation by using the correction shape factor ($\zeta$). The applied bending moment ($M_a$) for a simply supported non-curvature beam is given by:

$$M_a = \frac{PL}{4}$$  \hspace{1cm} (7.4)

The tested PCTL segments were, however, exposed to applied forces, as shown in Fig. 7.11, and the actual bending moment at the center is expressed in the following equation:

$$M = R(0.5l \cos \theta + \Delta \sin \theta)$$  \hspace{1cm} (7.5)
The actual moment on the tunnel segment can be redefined as follows:

\[ M = 0.5P / \cos \phi (0.5 \cos \phi + \Delta \sin \phi) \]  \hspace{1cm} (7.6a)

\[ M = PL / 4 - 0.5Px + 0.5P\Delta \tan \phi \]  \hspace{1cm} (7.6b)

\[ M = M_a - M_a h / L \tan \phi + 2M_a \Delta \tan \phi / L \]  \hspace{1cm} (7.6c)

\[ M = M_a (1 - h \tan \phi / L + 2\Delta \tan \phi / L) = \zeta M_a \]  \hspace{1cm} (7.6d)

Thus, the effective moment of inertia \((I_e)\) for PCTLs considering the suggested modification can be rewritten as:

\[ I_e = I_{cr} / \left[ 1 - \gamma / \zeta^2 (1 - (I_{cr} / I_e)(0.8M_{cr} / M_a)^2) \right] \]  \hspace{1cm} (7.7)

Figure 7.11 Applied forces and actual bending moment acting on tested PCTL segments.

### 7.5.3 Comparative Experimental Deflection to Code Models and the Modified Bischoff’s Equation

The experimental deflections were compared to prediction models at the service stage to evaluate accuracy. As mentioned above, the service loads according to Conforti et al. (2019) and design loads introduced by Abbas (Abbas et al. 2014) for tunnel segments were low. Accordingly, using these loads might be an unreliable and nonrational assumption in evaluating theoretical models. In this section, the reference load was defined at 30% of the peak load (Bischoff and Gross 2011)—in addition to the reference load at a strain limit of 2000 \(\mu\varepsilon\) (Mousa et al. 2019)—to provide an appropriate fit with the experimental results. To support the assessment of the modified equation, the outcome is presented for \(M_a / M_{cr}\) of 1.5 in Table 7.6, as suggested by Barris et al. (2013). For all models, the average measured to predicted deflections \((\Delta_{exp} / \Delta_{pre})\) at 0.3\(P_u\) were overestimated,
while the predictions for tested segments at $M_a/M_{cr}$ of 1.5 were conservative. The interpretation is that the prediction at $0.3P_u$ was computed at loads lower than the theoretical cracking loads. Accordingly, using the effective moment of inertia in prediction equations before cracking increases deflection. At a strain limit of 2000 $\mu$ε, the ACI 440.11-22 and the modified equations yielded a more accurate prediction than the CSA S806-12 equation. This accuracy can be seen in the modified equation, which reached 98%. The accuracy of the calculated deflection is highly dependent on the accuracy predicted cracking moment, which is a function of concrete strength (Confrere et al. 2016). Therefore, segments G-0.46 and G-0.86 had an average $\Delta_{exp}/\Delta_{pre}$ of 0.96 at $0.3P_u$. In contrast, the segment fabricated with HSC (G-0.46-H) had high predicted deflection for all models (Fig. 7.12). At $M_a/M_{cr} = 1.5$, the modified equation provided a slightly better prediction than ACI 440.11-22, with an average ratio ($\Delta_{exp}/\Delta_{pre}$) of 1.32±0.17 and a corresponding COV of 13%. Integrating the correction shape factor into the modified Bischoff’s equation decreased the predicted effective moment of inertia, resulting in an increase in the calculated deflection. This, in turn, enhanced the predicted deflection to be close to the measured deflection and improved the prediction of curved PCTL segments.

Table 7.6 Comparison between experimental and predicted deflection.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Measured deflection (mm)</th>
<th>ACI 440.11-22</th>
<th>CSA S806-12</th>
<th>Modified equation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\Delta_{exp}/\Delta_{pre}$ at $M_a/M_{cr} = 1.5$</td>
<td>$\Delta_{exp}/\Delta_{pre}$ at $M_a/M_{cr} = 1.5$</td>
<td>$\Delta_{exp}/\Delta_{pre}$ at $M_a/M_{cr} = 1.5$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.3 $P_u$</td>
<td>2000 $\mu$ε</td>
<td>0.3 $P_u$</td>
<td>2000 $\mu$ε</td>
</tr>
<tr>
<td>G-0.46</td>
<td>3.36</td>
<td>3.64</td>
<td>7.44</td>
<td>0.98</td>
</tr>
<tr>
<td>G-0.86</td>
<td>1.96</td>
<td>2.54</td>
<td>5.11</td>
<td>0.92</td>
</tr>
<tr>
<td>G-0.46-H</td>
<td>2.19</td>
<td>3.54</td>
<td>7.01</td>
<td>0.49</td>
</tr>
<tr>
<td>Mean</td>
<td>0.81</td>
<td>0.95</td>
<td>1.38</td>
<td>0.74</td>
</tr>
<tr>
<td>S.D.</td>
<td>0.22</td>
<td>0.17</td>
<td>0.17</td>
<td>0.14</td>
</tr>
<tr>
<td>COV (%)</td>
<td>27</td>
<td>17</td>
<td>12</td>
<td>19</td>
</tr>
</tbody>
</table>
Figure 7.12 Experimental versus predicted deflection.

7.6 Shear prediction of PCTL segments

Service loads imposed on tunnel segments from soil settlement underneath tunnels and/or internal vehicular accidents can induce shear failures. Additionally, PCTL segments are subjected to shear loads during transportation and storage in fabrication processes. This section presents shear capacity predictions specific to practical tunnels and related designs.

7.6.1 Modified Compression Field Theory (MCFT)

Vecchio and Collins (1986) first introduced the MCFT as a general shear model to explain the cracked concrete behavior of steel-RC elements. This approach depends on the maximum allowable aggregate interlock stress, which is a function of crack width, concrete strength, and maximum aggregate size. Based on the average strain perpendicular to the crack and crack perpendicular spacing (Fig. 7.13(a)), the following equation can be used to estimate crack width:
\[ w = \varepsilon_x s_x \quad (7.8) \]

The hypothesis aimed to account for the principal tensile stresses in the cracked element, equilibrium, compatibility, and constitutive relationships to form the full equations of the MCFT. The 15 nonlinear equations in the MCFT can be used to obtain a complete load–deformation analysis and predict shear strength accurately (El-Sayed and Soudki 2011). Practical design codes, however, require a means of quickly calculating shear capacity rather than a full load–deformation response that requires unreliable simulations. Some simplified assumptions were implemented to make the design practical by reducing the number of equations to two while still providing an accurate prediction of shear strength (Bentz et al. 2006). The shear design procedure in the current Canadian code (CSA-A23.3-19) for steel-reinforced concrete structures is based on the simplifications of Bentz and Collins (2006). Hoult et al. (2008) optimized an equation of concrete shear strength to account for the wide range of strains associated with FRP bars. This equation depends on the second-order correlation between the average strain \( \varepsilon_x \) in longitudinal bars and the diagonal crack width and is rewritten as follows:

\[
V_{sf} = \left( \frac{0.3}{0.5} + (1000 \varepsilon_x + 0.15)^{0.7} \right) \left( \frac{1300}{1000 + s_{ze}} \right) \sqrt{f_c b_w d_v} 
\quad (7.9)
\]

A simplified version of the MCFT ignored the interdependence between the strain effect factor (first term) and the size effect factor (second term). The size effect term refers to the effective crack spacing \( S_{ze} \), which is calculated for sections without transverse reinforcement or less than the minimum as:

\[
S_{ze} = \frac{31.5d}{16 + d_g} \geq 0.77d
\quad (7.10)
\]

For non-prestressed members with no axial force, the average longitudinal strain \( \varepsilon_x \) calculated at mid-height of the critical section (Fig. 7.13(b)) is:

\[
\varepsilon_x = \frac{(M_f/d_v + V_f)}{2E_h A_h}
\quad (7.11)
\]

The factored moment \( M_f \) and shear \( V_f \) can be calculated from the applied forces acting on the PCTL segment at the control section.
7.6.2 Critical Shear Crack Theory (CSCT)

Muttoni and Schwartz (1991) were the pioneers of the critical shear crack theory for steel-reinforced elements, which extends the diagonal shear failure envelope. The assumption of this theory depends on the width of the critical shear crack and the development of reinforcement strain in the adjacent section. The CSCT theory posits that the concrete contribution is determined by the strength of an inclined compressive strut, which is directly related to the characteristics of the critical shear crack, such as its width and roughness. Muttoni and Ruiz (2008) introduced the strut resistance as:

\[ V_{r,c} = b_c d \sqrt{f_c} f(w, d_g) \]  

(7.12)

The failure criteria provided in Eq. (7.12) links the function of width and roughness of the critical shear crack \( f(w, d_g) \) (assessed through the maximum aggregate size \( d_g \)) to concrete resistance \( (V_{R,c}) \).

7.6.2.1 Modifications of the CSCT of GFRP-Reinforced PCTL Segments

This study proposes, consistent with main assumption of the CSCT, that the critical crack opening is assumed to be proportional to a reference flexural strain present in GFRP bars multiplied by the effective depth \( (W \propto \varepsilon_{uf} d) \). Based on this correlation, concrete shear resistance can be computed as in Eq. (7.13):
where \( d_{go} \) is set to 16 mm. In the control section (Fig. 13(a)), the linear elastic behavior of the concrete’s compression region can be assumed while neglecting the tensile strength of the cracked concrete. The reference strain \( (\varepsilon_{fr}) \) can be rewritten in the control section located at 0.5\( d \) from the point load at a corresponding depth of 0.6\( d \) from the intrados surface (Campana et al. 2014) as follows (Fig. 7.13(b)):

\[
\varepsilon_{fr} = \frac{M}{A_f E_f z} \left( \frac{0.6d - x}{d - x} \right) = \frac{V(a - 0.5d)}{A_f E_f (d - \frac{x}{3})} \left( \frac{0.6d - x}{d - x} \right)
\]

(7.14)

where \( M \) is the external moment at the control section; \( A_f \) and \( E_f \) are the area and modulus of elasticity of longitudinal reinforcement; and \( z \) is the flexural lever arm between the compression force in the concrete and the tensile force of the reinforcement. The depth of the compression zone \( (x) \) can be calculated considering the factor of the neutral-axis depth for the cracked transformed section \( (k) \) as follows:

\[
x = kd
\]

(7.15a)

\[
k = \sqrt{2 \rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f
\]

(7.15b)

### 7.6.3 Influence of the Negative Curvature of the GFRP-Reinforced PCTL Segments

The curved geometry of the tunnel segments caused deviations in the applied forces and stresses, which, in turn, affected the shear force transfer through the critical shear crack. Therefore, it is necessary to calculate the additional force due to the change in the tension force along the curvilinear GFRP bars in concave (negative curvature) PCTLs. Based on the free bodies and equilibrium forces of shear transfer by a critical shear crack shown in Figs 7.14 (a) and (b), the following equations have been derived:

\[
Va = z F_c \cos \theta
\]

(7.16)

\[
\tan \alpha = \frac{\Delta V}{F_c \cos \theta} = \frac{\lambda d}{R_f}
\]

(7.17)
Hence, the additional force induced by deviation forces is calculated as follows:

\[
\Delta V = \frac{V_a \lambda d}{z_1} R_f = \frac{V_a \lambda d}{z - \frac{(\lambda d)^2}{2R_f}}
\]  

(7.18)

where \( \theta \) and \( \alpha \) are the slope angles of the resultant forces of the concrete and reinforcement, respectively; \( R_f \) is the curvature of the GFRP longitudinal bars; \( z_1 \) is the length of the flexural lever arm at mid-span; \( a \) is the length of the shear span; and \( \lambda \) is a factor standing for the length of deviation forces. The literature introduced the factor \( \lambda \) to estimate concrete contribution based on segment geometry, control section location, critical shear crack location and shape, and loading conditions (concentrated or distributed loads). The value of factor \( \lambda \) was assumed to be 0.50 for segment S-0.47 consistent with Campana et al. (2014). The adopted value to consider the length of deviation forces for PCTL segments with GFRP bars was, however, 0.65. Once failure occurs, the predicted concrete shear strength is the sum of the shear capacity of the CSC (contribution compressive concrete strut) and deviation forces as expressed by Eq. (7.19):

\[
V_{cf} = V_{csc} + \Delta V = V_{r,c} + \Delta V
\]  

(7.19)

### 7.6.4 Contribution of GFRP Closed Ties to Shear Strength

The experimental results demonstrate that stirrups are important to improve segment confinement, even though the ratio of the closed ties in the PCTLs was less than the minimum shear reinforcement (ACI 440.11-22; S806-12). In this context, Ruis and Muttoni (2009) suggest that the concrete carries a portion of the applied shear force and that the remaining portion is carried by shear reinforcement when a critical shear crack breaks the concrete teeth. As a result, the predicted contribution of closed ties is computed in the preceding control section by Eq. (7.20):

\[
V_{sf} = \frac{A_p E_p E_{fr}}{S} d
\]  

(7.20)

The strain value of 0.0025 in GFRP-reinforced PCTL segments was adopted to control the shear crack width, maintain concrete shear integrity, and avoid failure at the bent portion of ties (Jumaa and Yousif 2019). It should be mentioned that the strain limit in segment S-0.47 was replaced by
the yield strain ($\varepsilon_y$) of the steel stirrups. Figure 7.14(c) shows the contributions of the concrete and stirrups of the PCTLs, and the total shear strength can be determined as:

$$V_{\text{pred.}} = V_{cf} + V_{sf}$$  \hspace{1cm} (7.21)

Figure 7.14 (a) free bodies of influence of deviation forces; (b) equilibrium of corresponded forces; and (c) concrete and shear reinforcement strengths at CSC.

### 7.6.5 Plasticity Theory (PT) Analysis of Concave GFRP-Reinforced PCTL Segments

Kragh-Poulsen (2020) developed a new model for predicting the shear strength of curved steel-reinforced beams. In this model, the major shear crack is assumed to transform into a yield line. Its length starts at point A on the extrados surface and extends to point B, which represents the location of the applied load and can be calculated by:

$$L_{ab} = (h + R_m) \sin \beta_1 / \cos \beta_2$$  \hspace{1cm} (7.22)

Figure 7.15 shows the relation between angles $\beta_1$, $\beta_2$, and $\beta_3$ as follows:

$$\beta_3 = \pi / 2 - \beta_1 - \beta_2$$  \hspace{1cm} (7.23)

$$\sin \beta_3 = \cos \beta_1 \cos \beta_2 - \sin \beta_1 \sin \beta_2$$  \hspace{1cm} (7.24)

$$\tan \beta_2 = ((h + R_m) \cos \beta_1 - R / (h + R_m) \sin \beta_2)$$  \hspace{1cm} (7.25)

The ultimate concrete shear strength ($V_{cf}$) can be expressed as (Kragh-Poulsen 2020):
\[ V_{cf} = 0.5v f_c b_w (1 - \sin \beta_1)(h + R_m) \tan \beta_1 / \cos \beta_2 \]  

(7.26)

For PCTL segments reinforced with GFRP bars, the effective strength factor \((v)\) can be rewritten as:

\[ v = 2.81(1 + 0.37h / (16 + a_g))^{-0.59} (1 + 5.79 \rho_f E_f / E_c) / f_c^{0.6} \]  

(7.27)

To calculate the angle \(\beta_1\), the ultimate shear strength (Eq. 26) is set to equal the cracking load \((V_{cr})\) as follows:

\[ V_{cf} = V_{cr} = 0.5 \left( (h + R_m \sin \beta_1)^2 + (h + R_m \cos \beta_1 - R_m)^2 \right) f_{tef} b_w / a \]  

(7.28)

where the effective tensile strength \((f_{tef})\) for the GFRP-reinforced PCTLs is rewritten as:

\[ f_{tef} = 0.32 f_c^{2/3} (1 + 0.37h / (16 + a_g))^{-0.59} (1 + 5.79 \rho_f E_f / E_c) \]  

(7.29)

Figure 7. 15 Idealized diagonal crack (AB) based on plasticity theory (PT).

7.6.6 Comparisons between the experimental and shear predictions of PCTL segments

A comparison between the experimental results and the predicted shear strength was performed using the simplified approaches in current provisions (ACI 440.11-22; CSA S806-12). Moreover, these experiments were compared to those values predicted by selected shear models of FRP-RC elements (Nehdi et al. 2020; Ali et al. 2021), the modified critical shear crack theory, plasticity...
theory (Kragh-Poulsen 2020), and modified compression field theory with some amendments. All comparisons aim to evaluate the applicability of the aforementioned methods to predict the shear strength of PCTL segments with GFRP bars and ties. As the MCFT (Hoult et al., 2008.) and CSCT have similar assumptions, the modifications incorporated into the CSCT to account for the effect of negative curvature of GFRP-reinforced PCTL segments on concrete strength and the contribution of the stirrups can also be applied to the simplified MCFT. The selected FRP-RC models (Nehdi et al. 2020; Ali et al. 2021) and plasticity theory (Kragh-Poulsen 2020) only considered concrete contributions. The calculations of the models listed in Table 7.7 include the stirrups’ contribution according to ACI 440.11-22.

Table 7.7 Experimental-to-predicted shear capacity.

<table>
<thead>
<tr>
<th></th>
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<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$V_{\text{exp}}/V_{\text{pred}}$ (kN)</td>
<td>$V_{\text{exp}}/V_{\text{pred}}$ (kN)</td>
<td>$V_{\text{exp}}/V_{\text{pred}}$ (kN)</td>
<td>$V_{\text{exp}}/V_{\text{pred}}$ (kN)</td>
<td>$V_{\text{exp}}/V_{\text{pred}}$ (kN)</td>
<td>$V_{\text{exp}}/V_{\text{pred}}$ (kN)</td>
<td>$V_{\text{exp}}/V_{\text{pred}}$ (kN)</td>
<td>$V_{\text{exp}}/V_{\text{pred}}$ (kN)</td>
</tr>
<tr>
<td>G-0.46</td>
<td>1.66</td>
<td>0.98</td>
<td>1.10</td>
<td>0.99</td>
<td>2.19</td>
<td>1.38</td>
<td>1.34</td>
<td>1.33</td>
</tr>
<tr>
<td>G-0.86</td>
<td>1.45</td>
<td>0.81</td>
<td>1.00</td>
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<td>1.64</td>
<td>1.14</td>
<td>1.05</td>
<td>1.08</td>
</tr>
<tr>
<td>G-0.46-H</td>
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<td>1.13</td>
<td>1.02</td>
<td>2.62</td>
<td>1.44</td>
<td>1.42</td>
<td>1.16</td>
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<tr>
<td>Mean</td>
<td>1.61</td>
<td>0.93</td>
<td>1.08</td>
<td>0.98</td>
<td>2.03</td>
<td>1.32</td>
<td>1.27</td>
<td>1.19</td>
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<tr>
<td>S.D.</td>
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<td>0.05</td>
<td>0.04</td>
<td>0.34</td>
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<td>0.15</td>
<td>0.10</td>
</tr>
<tr>
<td>COV (%)</td>
<td>7</td>
<td>10</td>
<td>5</td>
<td>4</td>
<td>16</td>
<td>10</td>
<td>11</td>
<td>8</td>
</tr>
<tr>
<td>S-0.47</td>
<td>2.55</td>
<td>1.01</td>
<td>2.45</td>
<td>1.14</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

1 The shear capacity for the steel-reinforced segment was calculated according to ACI 318-19 and CSA A.23-19.
2 Obtained due to the concrete compressive strength of 60 MPa as the maximum limit according to CSA S806-12.

Figure 7.16 depicts the ratio between the experimental results and predicted shear strengths of the tested GFRP-reinforced PCTL segments. Both the experimental and predicted shear capacities were higher than the ultimate shear resistance induced in field projects (Conforti et al. 2019). The MCFT (Hoult et al. 2008) yielded extremely underestimated predictions, with an average $V_{\text{exp}}/V_{\text{pred}}$ of 2.03. The combination of the concrete contributions, including the effect of the negative curved shape of PCTLs and the ties’ resistance in the simplified MCFT, provided considerably conservative predictions with an average $V_{\text{exp}}/V_{\text{pred}}$ of 1.32 $\pm$ 0.13. In this context, based on the modified CSCT, the estimated shear strengths were reasonable compared to the actual capacities of the segments with a $V_{\text{exp}}/V_{\text{pred}}$ of 1.27 $\pm$ 0.15 and a corresponding COV of 11%. Moreover, the modified PT yielded accurate predictions with a $V_{\text{exp}}/V_{\text{pred}}$ of 1.19. Adding the stirrup contributions to the shear resistance of Nehdi et al. 2020 and Ali et al. 2021 models predicted the actual shear resistance accurately with COVs of 5% and 4%, respectively. This demonstrates the feasibility of
calculating the shear strength of GFRP ties for PCTLs, which enhanced the prediction of the shear capacity of the tested segments. CSA S806-12 yielded slightly overestimated predictions with an average $V_{\text{exp}}/V_{\text{pred}}$ of 0.93. Conversely, the ACI 440.11-22 equation provided lower concrete strength, which, in turn, yielded high conservative predictions with an average $V_{\text{exp}}/V_{\text{pred}}$ of $1.61 \pm 0.12$ and a COV of 7%.

![Figure 7.16 Experimental versus predicted shear capacity.](image)

### 7.7 Conclusions and Recommendations

The main findings are as follows:

1. Regardless of the test parameters, similar features of crack propagation were observed in all of the tested segments. At reference loads—at the same GFRP and steel reinforcement ratios—the crack widths of the segments were less than the maximum code limits. It was higher, however, when the steel bars yielded in segment S-0.47 and increased extensively upon failure. The effect of the negative arch shape on crack width and the related deflection was greater in segment S-0.47.

2. The GFRP-reinforced PCTL segments experienced shear failure characterized by a typical diagonal tension mode. In contrast, failure initiated in the steel-reinforced segment by flexural bars yielding before diagonal tension failure occurred. Despite the segments having comparable levels of shear strength, the major shear crack angle was steeper in G-0.46 and deviated significantly from the theoretical result.

3. Increasing the reinforcement ratio and concrete strength enhanced the structural performance of the GFRP-reinforced PCTLs under shear loads. The additional forces due
to the inclined tension force in the curved bars increased the shear capacity of all the segments.

4. Considering the geometry of the curved PCTL segments in the modified Bischoff’s equation improved deflection prediction. The modified equation yielded an accurate prediction with an average ratio \( \frac{\Delta_{\text{exp}}}{\Delta_{\text{pre}}} \) of 0.98 at reference load (at a strain limit of 2000 \( \mu \epsilon \)). ACI 440.11-22 and the modified equation predicted deflection more accurately than CSA S806-12.

5. The experimental and theoretical shear strengths for the tested segments were higher than the practical capacity in field projects. CSA S806-12 slightly overestimated predictions, with an average \( \frac{V_{\exp}}{V_{\text{pred}}} \) of 0.93. Conversely, the MCFT and ACI 440.11-22 were highly conservative.

6. The simplified MCFT, modified CSCT, and modified PT consider the effect of the arch shape on and the stirrup contribution to concrete strength, which enhanced the predictions of the actual capacities of the GFRP-reinforced segments. The modified CSCT provided a reasonable prediction, with a \( \frac{V_{\exp}}{V_{\text{pred}}} \) of 1.27 ± 0.15. The length coefficient for deviation forces and an effective stirrup strain limit were proposed in this study to calculate the shear strength of PCTL segments reinforced with GFRP bars and ties. Further experimental research, however, is needed to verify these values.
CHAPTER 8
GENERAL CONCLUSIONS AND RECOMMENDATIONS

8.1 Summary

The current research aims to experimentally and theoretically investigate the punching and shear behavior of GFRP-reinforced PCTL segments. The experimental program was completed in two phases. The first phase was conducted on an investigation of the punching-shear behavior of PCTL segments induced by soil conditions, such as rock expansion or geotechnical conditions surrounding a tunnel. A total of eight full-scale rhomboidal PCTL segments measuring 1500 mm in width and 250 mm in thickness, and with an arc length of 3100 mm and 2100 mm were designed, cast, and tested under concentrated load until failure. The second phase included testing of five convex and four concave full-scale PCTL segments to investigate the shear behavior of tunnel segments under fabrication and service loads. Segments were constructed with a rhomboidal shape measuring 2100 x 1500 x 250 mm and subjected to line loading on their extrados and intrados surfaces up to failure. The test parameters were the reinforcement type (GFRP and steel), reinforcement ratio, concrete strength (NSC and HSC), transverse reinforcement configuration (closed versus bars), arc length, and shear reinforcement. The test results were carefully analyzed in terms of punching and shear strength, cracking behavior, failure modes, load-deflection response, and strains in reinforcement and concrete. The effects of test parameters on punching and shear behavior were discussed and evaluated. The experimental results were compared to the shear and punching capacity predictions provided by the current design equations of FRP standards and available models. In addition, a theoretical shear prediction according to CSCT, MCFT, and PT theories with some amendments, including the contribution of closed ties and curved geometry of GFRP-reinforced PCTL segments, and comparison with the experimental results. Furthermore, using the measured deflection, a comparison between the experimental values of the effective moment of inertia and those predicted using modified models to evaluate their capability for tunnel segments reinforced with GFRP bars was performed.
8.2 Conclusions

Based on the experimental and analytical results acquired in this research, the following general conclusions can be drawn:

8.2.1 Phase I: GFRP-Reinforced PCTL Segments under Punching Loads

8.2.1.1 Experimental Results

1. The short-span GFRP-reinforced PCTL segments exhibited punching–shear failure, while a mixed flexural–punching mode was observed in the segments with a longer span. The shear stirrups eliminated and converted the brittle failure to a more ductile mode. The steel-reinforced segment failed in flexure due to yielding of the steel.

2. The punching-failure mode of short-span specimens is triggered by an instant reduction in load-carrying capacity and a classical punching cone. Moreover, the steel plate sank into the concrete with no signs of flexural failure.

3. Using GFRP reinforcement at the same ratio as the steel reinforcement in the PCTL segments satisfied code requirements in terms of crack opening at service load. Furthermore, both segments G(0.46)-2.1 and S(0.47)-2.1 failed at a load of 288 kN (64.6 kips) and 321 kN (72.1 kips), respectively, in a range of only an 11% difference.

4. The higher GFRP reinforcement ratio yielded higher punching capacity, greater failure surface, lower deflection, lower strain, and narrower crack openings. Increasing the reinforcement ratio from 0.46% to 0.86% in G(0.86)-2.1, G(0.86)-2.1-H, and G(0.86)-3.1 increased the normalized punching capacity by 30%, 29%, and 22%, respectively.

5. The segment length significantly affected the punching–shear behavior. The flexural stresses for segments increased for the longer span at the same applied load. Wider and more cracks appeared, resulting in an increase in deflection.

6. A small amount of GFRP shear stirrups improved the confinement of the punching region and contributed to an enhancement in normalized punching-shear resistance by 14% and segment deformation by 7%.
7. Employing HSC in the GFRP-reinforced PCTL segments significantly enhanced the pre-cracking behavior (increased the cracking load and pre-cracking stiffness). While all the PCTL segments satisfied the service limit of crack width, the HSC specimens had narrower and fewer cracks.

### 8.2.1.2 Theoretical Results

8. The punching-shear capacity of PCTL segments reinforced with GFRP bars can be predicted with current FRP design provisions, while the researchers’ proposed equations overestimated punching capacities. CAN/CSA S806-12 yielded the most accurate predictions. Moreover, JSCE (1997) and BSI (1997) yielded good predictions, while the predictions of ACI 440.11-22, fib TG-9.3 (2007), and AASHTO (2018) were excessively conservative.

9. For large-span specimens, CAN/CSA S806-12, JSCE (1997), and BSI (1997) overestimated predictions due to neglecting the effect of span length. Adding the contribution of shear stirrups in specimen $G_{(0.46)} - 2.1$-SR enhanced the predictions of these provisions.

10. The ultimate normalized punching-shear stress was calculated by incorporating the cubic root of the concrete strength. Consequently, CSA S806-12 produced more accurate predictions—especially for HSC segments—than JSCE (1997), which used the square root of the concrete strength.

11. CSA S806-12 specified a maximum limit of concrete compressive strength of 60 MPa. Despite that, using high-strength concrete of 70.7 and 66.9 MPa in specimens $G_{(0.46)} - 2.1$-H and $G_{(0.86)} - 2.1$-H, respectively, yielded good predictions. The $V_{exp}/V_{pred}$ ratios for those segments were 0.95 and 1.04, respectively.

### 8.2.2 Phase II: GFRP-Reinforced PCTL Segments under Shear Loads

#### 8.2.2.1 Convex PCTL Segments (Positive Curvature)

1. The observed failure mode of almost all the GFRP-reinforced PCTL segments tested was diagonal tension failure. The closer tie spacing in specimen $G_{(0.86)/C100}$ led to shear compression failure mode, followed by rupture of closed ties. The average theoretical
inclination angle of the critical shear crack is 63°, a variation that is uneven and very different that the experimentally observed value (30°).

2. Replacing the transverse bars with closed ties significantly improved PCTL shear behavior (post-peak) and strength, despite the low reinforcement ratio. The presence of stirrups increased the ultimate deflection and normalized shear strength, while decreasing shear-crack width and achieving ductile failure.

3. Compared to segment G(0.86)/C200, the higher shear reinforcement ratio in specimen G(0.86)/C100 increased the normalized shear strength and ultimate stiffness by 16% and 12%, respectively. This is evidence of effective control of the widening and propagation of shear cracks as well as maintaining the confinement of the concrete core in the PCTLs.

4. Increasing the longitudinal reinforcement ratio increased the ultimate stiffness by 90%. The high number of flexural bars, however, had the same effect as using stirrups on the normalized shear strength of the PCTL segment, which increased it by 12%. Based on the shear strength, using stirrups would be a cost-effective alternative solution for tunnels.

5. The increase in concrete strength from 50 to 70 MPa increased the cracking load and initial stiffness, which enhanced the pre-cracking behavior of the segment. In addition, the HSC in segment G(0.86)/C200-H produced large numbers of narrower cracks.

6. The shear strengths of the tested segments predicted with JSCE (1997) and CSA S6-19 were significantly lower than those measured experimentally, whereas CSA S806-12 overestimated the predicted strengths. The average $V_{\text{exp}}/V_{\text{pred}}$ for ACI 440.11-22 and AASHTO (2018) was approximately 1.25, indicating conservative predictions for shear capacity.

7. By ignoring the shear resistance of closed ties, the redefined CSCT (2014) for GFRP-reinforced PCTL segments reasonably predicted actual capacities. The modified CSCT—which considers stirrup strength—yielded the most accurate agreement with experimental results, with an average $V_{\text{exp}}/V_{\text{pred}}$ of 0.98 and a COV of 14%.

8. Using a 0.65 length coefficient for deviation forces and an effective stirrup strain limit of 0.004 provided appropriate results with the equations in the modified CSCT.
8.2.2.2 Concave PCTL Segments (Negative Curvature)

9. Regardless of test parameters, similar features of crack propagation were observed for all tested segments. At reference loads—with the same GFRP and steel reinforcement ratio—the crack widths of segments were less than the maximum code limits. However, it was higher when the steel bars yielded in segment S-0.47 and extensively increased upon failure. The effect of negative arch shape on crack width and related deflection is more pronounced in segment S-0.47.

10. The GFRP-reinforced PCTL segments exhibited shear failure characterized by a typical diagonal tension mode. However, the failure in the steel-reinforced segment was initiated by flexural bars yielding before diagonal tension failure. Despite a comparable shear strength, the major shear crack angle was steeper at G-0.46 and highly deviated from the theoretical result.

11. Increasing the reinforcement ratio and concrete strength enhanced the structural performance of GFRP-reinforced PCTLs under shear loads. The additional forces due to the inclined tension force in the curved bars increased the shear capacity of all segments.

12. Considering the geometry of curved PCTL segments in the modified Bischoff equation improves deflection prediction. The modified equation yielded an accurate prediction with an average ratio \((\Delta_{\text{exp}} / \Delta_{\text{pred}})\) of 0.98 at reference load (at a strain limit of 2000 \(\mu\varepsilon\)). ACI 440.11-22 and the modified equation predicted deflection more accurately than CSA S806-12.

13. The experimental and theoretical shear strengths for tested segments were higher than the practical capacity in field projects. CSA S806-12 provided slightly overestimated predictions, with an average \(V_{\text{exp}}/V_{\text{pred}}\) of 0.93. Conversely, MCFT and ACI 440.11-22 were highly conservative.

14. The simplified MCFT, modified CSCT, and modified PT consider the effect of the arch shape and stirrup contribution on concrete strength, which enhances predictions of actual capacities of the GFRP-reinforced segments. The modified CSCT provided a reasonable prediction, with a \(V_{\text{exp}}/V_{\text{pred}}\) of 1.27 ± 0.15. The length coefficient for deviation forces and an effective stirrup strain limit were proposed in this study to calculate the shear strength of PCTL segments with GFRP bars and ties. Further experimental research, however, is still needed to verify these values.
8.3 Recommendations for Future Work

Results of the current research provide unique experimental results and analytical predictions for the punching and shear behavior of GFRP-reinforced PCTL segments. Despite these findings and conclusions, it is important to continue the research studies in this promising field, knowing that there is still a lot of work to do. Some of the recommendations for future research are:

1. It is interesting to investigate the contribution of shear reinforcement on the punching behavior of GFRP-reinforced PCTL segments. More experimental work should be conducted on distribution, amount, and extension limit of GFRP shear stirrups in PCTL segments due to their complex curvature cages.
2. Investigate the punching and shear behaviors of tunnel segments with hybrid solutions (GFRP-reinforced FRC PCTL segments) with various types and dosages of both FRP reinforcement and fibers.
3. Analytical models should be developed using the obtained experimental results to predict the punching strength of curved GFRP-reinforced PCTLs with different parameters.
4. In this research, the modifications in critical shear crack theory (CSCT) are considered a promising step towards using this theory as a beneficial model to predict the shear capacity of convex and concave tunnel segments reinforced with FRP bars and closed ties. However, further analytical analysis is needed to verify the proposed values of length coefficient for deviation forces and an effective stirrup strain limit.
5. It is interesting to perform more experimental work on the contribution of closed ties on the shear strength of GFRP-reinforced PCTL segments to assess their effectiveness and pave the way to incorporate these findings into tunnel design provisions.
6. Investigate the size effect (a/d) and degree of curvature on the shear behavior of GFRP-reinforced PCTLs. Then, an analytical model concerning parametric study assessing the effect of different parameters should be developed.
7. Develop finite element model to investigate the punching and shear behavior of PCTL segments with GFRP bars considering the effect of real loading and boundary conditions.
8. Investigate the effect of thrust loads induced by tunnel boring machine (TBM) on the structural performance of GFRP-reinforced PCTL segments, both experimentally and numerically.
8.4 Sommaire

La recherche actuelle intitulée ici visait à étudier expérimentalement et théoriquement le comportement au poinçonnement et au cisaillement de segments de RTBP renforcés par du PRFV. Le programme expérimental a été réalisé en deux phases. La première phase a porté sur l'étude du comportement au poinçonnement des segments RTBP induit par les conditions du sol, telles que l'expansion de la roche ou les conditions géotechniques entourant un tunnel. Au total, huit segments RTBP rhomboïdaux grandeur nature mesurant 1500 mm de largeur et 250 mm d'épaisseur, et ayant une longueur d'arc de 3100 mm et 2100 mm, ont été conçus, coulés et testés sous une charge concentrée jusqu'à la rupture. La deuxième phase comprenait des essais sur cinq segments RTBP convexes et quatre concaves en grandeur réelle afin d'étudier le comportement en cisaillement des segments de tunnel sous les charges de fabrication et de service. Les segments ont été construits avec une forme rhomboïdale mesurant 2100 x 1500 x 250 mm et soumis à une charge linéaire sur leurs surfaces extrados et intrados jusqu'à la rupture. Les paramètres d'essai étaient le type de renforcement (PRFV et acier), le taux de renforcement, la résistance du béton (BRN et BHR), la configuration du renforcement transversal (fermé ou barres), la longueur de l'arc et le renforcement de cisaillement. Les résultats des essais ont été soigneusement analysés en termes de résistance au poinçonnement et au cisaillement, de comportement de fissuration, de modes de défaillance, de réponse charge-déformation et de déformations dans l'armature et le béton. Les effets des paramètres d'essai sur le comportement au poinçonnement et au cisaillement ont été discutés et évalués. Les résultats expérimentaux ont été comparés aux prévisions de capacité de cisaillement et de flexion fournies par les équations de conception actuelles des normes de PRF et des modèles disponibles. En outre, une prédiction théorique du cisaillement selon les théories CSCT, MCFT et PT avec quelques modifications, y compris la contribution des liens fermés et la géométrie incurvée des segments RTBP renforcés de PRFV, et une comparaison avec les résultats expérimentaux. De plus, en utilisant la flèche mesurée, une comparaison entre les valeurs expérimentales du moment d'inertie effectif et celles prédites en utilisant les modèles modifiés pour évaluer leur capacité pour les segments de tunnel renforcés avec des barres PRFV a été réalisée.
8.5 Conclusions

Les résultats expérimentaux et analytiques obtenus dans le cadre de cette recherche permettent de tirer les conclusions générales suivantes :

8.5.1 Segments de RTBP renforcés par des PRFV soumis à des charges de poinçonnement

8.5.1.1 Résultats expérimentaux

1. Les segments RTBP renforcés en PRFV de courte portée ont présenté une rupture par poinçonnement et cisaillement, tandis qu'un mode mixte de flexion et de poinçonnement a été observé dans les segments de plus grande portée. Les étiers de cisaillement ont éliminé et converti la rupture fragile en un mode plus ductile. Le segment renforcé en acier s'est rompu en flexion en raison de la déformation de l'acier.

2. Le mode de rupture par poinçonnement des spécimens à courte portée est déclenché par une diminution instantanée de la capacité de charge et un cône de poinçonnement classique. En outre, la plaque d'acier s'est enfoncée dans le béton sans aucun signe de rupture par flexion.

3. L'utilisation d'armatures en PRFV dans le même rapport que les armatures en acier dans les segments RTBP a satisfait aux exigences du code en ce qui concerne l'ouverture des fissures à la charge de service. En outre, les segments G_{0.46}-2.1 et S_{0.47}-2.1 se sont rompus à une charge de 288 kN (64.6 kips) et 321 kN (72.1 kips), respectivement, dans une différence de 11% seulement.

4. L'augmentation du taux de renforcement en PRFV a entraîné une plus grande capacité de poinçonnement, une plus grande surface de rupture, une flèche plus faible, une déformation plus faible et des ouvertures de fissures plus étroites. L'augmentation du taux de renforcement de 0.46 % à 0.86 % dans G_{0.86}-2.1, G_{0.86}-2.1-H et G_{0.86}-3.1 a augmenté la capacité de poinçonnement normalisée de 30 %, 29 % et 22 %, respectivement.

5. La longueur des segments a affecté de manière significative le comportement en poinçonnement-cisaillement. Les contraintes de flexion pour les segments ont augmenté
pour la plus grande portée à la même charge appliquée. Des fissures plus larges et plus nombreuses sont apparues, entraînant une augmentation de la flèche.

6. Une petite quantité d'étriers de cisaillement en PRFV a amélioré le confinement de la zone de poinçonnement et a contribué à une amélioration de la résistance normalisée au poinçonnement-cisaillement de 14 % et de la déformation du segment de 7 %.

7. L'utilisation de BHR dans les segments RTBP renforcés par PRFV a considérablement amélioré le comportement avant fissuration (augmentation de la charge de fissuration et de la rigidité avant fissuration). Alors que tous les segments RTBP respectaient la limite de service de la largeur de fissure, les spécimens BHR présentaient des fissures plus étroites et moins nombreuses.

8.5.1.2 Résultats théoriques


10. La contrainte ultime normalisée de poinçonnement-cisaillement a été calculée en incorporant la racine cubique de la résistance du béton. Par conséquent, la norme CSA S806-12 a produit des prévisions plus précises - en particulier pour les segments BHR- que la norme JSCE (1997), qui utilisait la racine carrée de la résistance du béton.

11. La norme CSA S806-12 spécifie une limite maximale de résistance à la compression du béton de 60 MPa. Malgré cela, l'utilisation d'un béton à haute résistance de 70,7 et 66,9 MPa dans les spécimens G(0.46)-2.1-H et G(0.86)-2.1-H, respectivement, a permis d'obtenir
de bonnes prédictions. Les rapports $V_{exp}/V_{pred}$ pour ces segments étaient respectivement de 0,95 et 1,04.

8.5.2 Phase II : Segments de RTBP renforcés par des PRFV soumis à des charges de cisaillement

8.5.2.1 Segments convexes RTBP (courbure positive)

1. Le mode de rupture observé pour presque tous les segments de RTBP renforcés par des PRFV testés était la rupture par traction diagonale. L'espacement plus étroit des traverses dans l'éprouvette G(0.86)/C100 a conduit à un mode de rupture par compression en cisaillement, suivi d'une rupture des cadres. L'angle d'inclinaison théorique moyen de la fissure de cisaillement critique est de 63°, une variation inégale et très différente de la valeur observée expérimentalement (30°).

2. Le remplacement des barres transversales par des cadres a amélioré de manière significative le comportement en cisaillement de la RTBP (après la pointe) et sa résistance, malgré le faible taux de renforcement. La présence d'étriers a augmenté la flèche ultime et la résistance au cisaillement normalisée, tout en diminuant la largeur des fissures de cisaillement et en obtenant une rupture ductile.

3. Par rapport au segment G(0.86)/C200, le taux de renforcement de cisaillement plus élevé dans l'éprouvette G(0.86)/C100 a augmenté la résistance au cisaillement normalisée et la rigidité ultime de 16 % et 12 %, respectivement. Ceci est la preuve d'un contrôle efficace de l'élargissement et de la propagation des fissures de cisaillement ainsi que du maintien du confinement du noyau de béton dans les RTBP.

4. L'augmentation du taux de renforcement longitudinale a augmenté la rigidité ultime de 90%. Le nombre élevé de barres de flexion a toutefois eu le même effet que l'utilisation d'étriers sur la résistance au cisaillement normalisée du segment RTBP, qui a augmenté de 12 %. Sur la base de la résistance au cisaillement, l'utilisation d'étriers serait une solution alternative rentable pour les tunnels.

5. L'augmentation de la résistance du béton de 50 à 70 MPa a augmenté la charge de fissuration et la rigidité initiale, ce qui a amélioré le comportement de pré-fissuration du
segment. En outre, le BHR dans le segment $G_{(0.86)/C200-H}$ a produit un grand nombre de fissures plus étroites.


7. En ignorant la résistance au cisaillement des cadres, le CSCT redéfini (2014) pour les segments RTBP renforcés par PRFV a raisonnablement prédit les capacités réelles. Le CSCT modifié - qui prend en compte la résistance des étriers - a produit l'accord le plus précis avec les résultats expérimentaux, avec un $V_{exp}/V_{pred}$ moyen de 0,98 et un COV de 14 %.

8. L'utilisation d'un coefficient de longueur de 0,65 pour les forces de déviation et d'une limite de déformation effective de l'étrier de 0,004 a donné des résultats appropriés avec les équations de la CSCT modifiée.

8.5.2.2 Segments RTBP concaves (courbure négative)

9. Indépendamment des paramètres d'essai, des caractéristiques similaires de propagation des fissures ont été observées pour tous les segments testés. Aux charges de référence - avec le même taux de renforcement en PRFV et en acier - les largeurs de fissure des segments étaient inférieures aux limites maximales prévues par le code. Cependant, elle était plus élevée lorsque les barres d'acier ont cédé dans le segment S-0.47 et augmentait considérablement lors de la rupture. L'effet de la forme négative de l'arc sur la largeur des fissures et la flèche correspondante est plus prononcé dans le segment S-0.47.

10. Les segments RTBP renforcés en PRFV ont présenté une rupture en cisaillement caractérisée par un mode de tension diagonale typique. Cependant, la rupture du segment renforcé par de l'acier a été initiée par des barres de flexion qui ont cédé avant la rupture par traction diagonale. Malgré une résistance au cisaillement comparable, l'angle de la fissure de cisaillement majeure était plus prononcé à G-0,46 et s'écartait fortement du résultat théorique.
11. L'augmentation du taux de renforcement et de la résistance du béton a amélioré la performance structurelle des RTBP renforcés par des PRFV sous des charges de cisaillement. Les forces supplémentaires dues à la force de tension inclinée dans les barres courbes ont augmenté la capacité de cisaillement de tous les segments.

12. La prise en compte de la géométrie des segments RTBP courbes dans l'équation de Bischoff modifiée améliore la prédiction de la flèche. L'équation modifiée a donné une prédiction précise avec un rapport moyen \( \Delta_{\text{exp}} / \Delta_{\text{pre}} \) de 0,98 à la charge de référence (à une limite de déformation de 2000 µε). L'ACI 440.11-22 et l'équation modifiée ont prédit la flèche avec plus de précision que la CSA S806-12.

13. Les résistances au cisaillement expérimentales et théoriques pour les segments testés étaient supérieures à la capacité pratique dans les projets sur le terrain. La norme CSA S806-12 a fourni des prévisions légèrement surestimées, avec un \( V_{\text{exp}} / V_{\text{pred}} \) moyen de 0,93. Inversement, les normes MCFT et ACI 440.11-22 ont été très conservatrices.

14. Le MCFT simplifié, le CSCT modifié et le PT modifié tiennent compte de l'effet de la forme de l'arc et de la contribution des étriers sur la résistance du béton, ce qui améliore les prévisions des capacités réelles des segments renforcés par des PRFV. Le CSCT modifié a fourni une prédiction raisonnable, avec un \( V_{\text{exp}} / V_{\text{pred}} \) de 1,27 ± 0,15. Le coefficient de longueur pour les forces de déviation et la limite de déformation effective de l'étrier ont été proposés dans cette étude pour calculer la résistance au cisaillement des segments RTBP avec des barres et des traverses en PRFV. D'autres recherches expérimentales sont toutefois nécessaires pour vérifier ces valeurs.

8.6 **Recommandations pour les travaux futurs**

Les résultats de la recherche actuelle fournissent des résultats expérimentaux uniques et des prévisions analytiques pour le comportement au poinçonnement et au cisaillement des segments de RTBP renforcés par des PRFV. Malgré ces résultats et conclusions, il est important de poursuivre les études de recherche dans ce domaine prometteur, tout en sachant qu'il reste encore beaucoup de travail à faire. Voici quelques-unes des recommandations pour les recherches futures:

1. Il est intéressant d'étudier la contribution de l'armature de cisaillement sur le comportement au poinçonnement des segments PCTL renforcés de PRFV. Des travaux expérimentaux
supplémentaires devraient être menés sur la distribution, la quantité et la limite d'extension des étiers de cisaillement en PRFV dans les segments RTBP en raison de leurs cages de courbure complexes.

2. Étudier les comportements au poinçonnement et au cisaillement des segments de tunnel avec des solutions hybrides (segments RTBP en BRF renforcés par des PRFV) avec différents types et dosages de renforts PRF et de fibres.

3. Des modèles analytiques devraient être développés en utilisant les résultats expérimentaux obtenus pour prédir la résistance au poinçonnement des RTBP courbes renforcées par des PRFV avec différents paramètres.

4. Dans cette recherche, les modifications de la théorie de la fissure de cisaillement critique (CSCT) sont considérées comme une étape prometteuse vers l'utilisation de cette théorie comme un modèle bénéfique pour prédir la capacité de cisaillement des segments de tunnel convexes et concaves renforcés avec des barres PRF et des traverses fermées. Cependant, une analyse analytique plus poussée est nécessaire pour vérifier les valeurs proposées du coefficient de longueur pour les forces de déviation et une limite de déformation effective de l'étier.

5. Il est intéressant d'effectuer davantage de travaux expérimentaux sur la contribution des cadres à la résistance au cisaillement des segments RTBP renforcés par des PRF afin d'évaluer leur efficacité et d'ouvrir la voie à l'incorporation de ces résultats dans les dispositions relatives au dimensionnement des tunnels.

6. Étudier l'effet de la taille (a/d) et du degré de courbure sur le comportement en cisaillement des RTBP renforcés par du PRFV. Ensuite, un modèle analytique concernant l'étude paramétrique évaluant l'effet des différents paramètres devrait être développé.

7. Développer un modèle d'éléments finis pour étudier le comportement au poinçonnement et au cisaillement des segments RTBP avec des barres PRF en tenant compte de l'effet de la charge réelle et des conditions aux limites.

8. Étudier l'effet des charges de poussée induites par le tunnelier sur la performance structurelle des segments RTBP en BHR renforcés par des PRFV, à la fois expérimentalement et numériquement.
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