Dynamic fracture mechanics of FRP-concrete interface

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**DYNAMIC FRACTURE MECHANICS OF FRP-
CONCRETE INTERFACE**

LI GEN

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

2020

DYNAMIC FRACTURE MECHANICS OF FRP- CONCRETE INTERFACE

LI GEN

School of Civil and Environmental Engineering

A thesis submitted to the Nanyang Technological University
in partial fulfilment of the requirement for the degree of
Doctor of Philosophy

2020

Statement of Originality

I hereby certify that the work embodied in this thesis is the result of original research, is free of plagiarised materials, and has not been submitted for a higher degree to any other University or Institution.

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25/March/2020

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Date

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Prof. Fung Tat Ching

Authorship Attribution Statement

This thesis contains material from 2 paper(s) published in the following peer-reviewed journal(s) / from papers accepted at conferences in which I am listed as an author.

Chapter 3 is published as G. Li, K. H. Tan, and T. C. Fung. Experimental study on CFRP-concrete dynamic debonding behaviour. *Engineering Structures*, 206 (2020), 110055.

The contributions of the co-authors are as follows:

- I designed this experimental study with Prof. Fung.
- I performed all the laboratory work at the School of Civil and Environmental Engineering. I also analysed the data.
- Prof. Tan and Prof. Fung have discussed the test results with me.
- I prepared the manuscript drafts. The manuscript was revised by Prof. Tan and Prof. Fung.

Chapter 5 is published as G. Li, K. H. Tan, T. C. Fung, and P. Del Linz. Mode I fracture characterisation of FRP-concrete interfaces under dynamic loading. *Composite Structures*, 254(2020), 112824.

The contributions of the co-authors are as follows:

- I designed this experimental study with Asst. Prof. Del Linz.
- I performed all the laboratory work at the School of Civil and Environmental Engineering. I also analysed the data.
- Prof. Tan, Prof. Fung and Asst. Prof. Del Linz have discussed the test results with me.

- I prepared the manuscript drafts. The manuscript was revised by Prof. Tan, Prof. Fung and Asst. Prof. Del Linz

01/Oct/2020

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Date

Li Gen

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Li Gen

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Summary

External bonding of fibre reinforced polymer (FRP) has been widely used as an effective technique for strengthening reinforced concrete (RC) structures in quasi-static and dynamic scenarios. To strengthen RC structures, FRP is usually made by several layers and bonded to the tension zone subjected to a tension load. Due to the inconsistent deformation in concrete and FRP laminate, relative slip and corresponding shear stress is created between the concrete and FRP and the bond of the interface eventually fails by shear debonding. However, tensile fracture failure is also noticed in many tests.

Although the quasi-static shear bond-slip response has already been extensively investigated to aid the FRP strengthening design, studies on their dynamic behaviour are rather limited, especially those under high loading rates. To shed light on the FRP bond-slip behaviour under high loading rates (above 800 mm/s), the author proposed a novel experimental method for high loading-rate impact tests using a modified Split Hopkinson Pressure Bar (SHPB) set-up. Dynamic enhancing effect on ultimate load, shear bond stress and shear fracture energy could be quantified for a wide range of loading rates varying from 0.02 mm/s to 2150 mm/s. The effect of parameters on the bond behaviour was better understood from a detailed parametric study.

A novel continuum damage model was proposed in this research to study the dynamic fracture mechanics of FRP-concrete interface in shear debonding. Model analyses and regression test data were conducted to formulate the relationship between model coefficients and material parameters of tested specimens in literature. It was found that the dynamic enhancing effect on interfacial bond properties became more significant when a specimen has a larger FRP load capacity or a lower concrete load capacity. The proposed damage model was verified through comparing numerical

predictions of load-displacement curve and FRP strain distribution along the bond length with published test results.

Apart from the studies on the shear debonding, this research consisted of two sets of experiments to study tensile fracture of FRP-concrete interface and to obtain the principal interfacial properties of such fracture. Direct tension tests were used to determine the tensile bond strength and notched three-point bending tests were used to determine the tensile fracture energy. Digital image correlation measurements were used to characterise the fracture process. It was found that the two bond properties were close to those of plain concrete in the quasi-static regime and showed significant dynamic enhancing effect at a loading rate of 20 mm/s. Dynamic increase factor (DIF) equations for the two bond properties are provided to predict the interfacial response of FRP strengthened RC structures under dynamic loads.

Through previous experimental investigations, it was found that FRP-concrete bond interface contains diverse traction-separation performance in mode I and mode II fracture. To continuously describe the fracture behaviour from mode I to mode II under a dynamic loading, current study proposed a coupled dynamic cohesive zone model (CZM) to analyse FRP-concrete mixed-mode separation. The model was applied in finite element analysis (FEA) of various dynamic tests with FRP-concrete interface bond, e.g. single-lap shear tests, three-point bending tests, FRP strengthened RC beams subjected to dynamic loading and an FRP strengthened RC wall under blast load. From the comparison with test results, the model was shown to be reliable and accurate in simulating the behaviour of FRP-concrete mixed-mode separation.

List of publications

Chapter 3 is published as G. Li, K. H. Tan, and T. C. Fung. Experimental study on CFRP-concrete dynamic debonding behaviour. *Engineering Structures*, 206 (2020), 110055.

Chapter 5 is published as G. Li, K. H. Tan, T. C. Fung, and P. Del Linz. Mode I fracture characterisation of FRP-concrete interfaces under dynamic loading. *Composite Structures*, 254(2020), 112824.

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List of symbols

A	Coefficient related to external work in three-point bending test
A_{con}	Concrete cross-sectional area
A_f	FRP cross-sectional area
a	Coefficient related to dynamic increase factor of shear bond strength
a_I, a_{II}	Coefficients related to dynamic enhancing effect on dynamic bond strength of mode I and mode II
B	Coefficient in damage rate equation, related to static material properties
b	Coefficient related to dynamic increase factor of shear fracture energy
b_c	Concrete substrate width
b_f	FRP bond width
$\cos I, \cos II$	Direction cosines for mode I and II required for calculation of mode mixity in cohesive zone model
D	Damage parameter in softening range
$\dot{D}$	Damage rate
DIF_{GI}	Dynamic increase factor of mode I fracture energy
DIF_{GII}	Dynamic increase factor of mode II fracture energy
DIF_τ	Dynamic increase factor of shear bond strength
E	Elastic modulus
E_c	Elastic modulus of concrete
E_e	Elastic modulus of epoxy
E_f	FRP elastic modulus in fibre direction
$E_f t_f$	FRP stiffness in fibre direction
E_x, E_y, E_z	Elastic moduli in x, y, z directions respectively
F_{con}	Static load capacity of concrete
F_f	Static load capacity of FRP
F_0	Constant, 1kN
f_0	Reference strength, 10 MPa
f'_c	Concrete cylinder strength
f_t	Concrete tensile strength
f_{cf}	FRP compressive strength in fibre direction
f_{sf}	FRP in-plane shear strength
f_{tf}	FRP tensile strength in fibre direction

G	Released energy
G_a	Linear elastic fracture energy of initial ascending branch
G_{FC}	Static fracture energy of plain concrete
G_{Id}	Dynamic fracture energy in mode I
$G_{II d}$	Dynamic fracture energy in mode II
$G_{M d}$	Dynamic fracture energy in mixed mode
G_{Ist}	Static fracture energy in mode I
G_{IIst}	Static fracture energy in mode II
G_i	Released energy at time step i
G_c	Concrete shear modulus
G_e	Epoxy shear modulus
G_s	Size-effect fracture energy
G_{xy}, G_{yz}, G_{zx}	Shear moduli in xy, yz, zx planes respectively
H	Coefficient related to dynamic shear bond strength
h	Height of the beam for three-point bending test
h_n	Height of the notch depth for three-point bending test
K	Stiffness
k	Initial stiffness of bond-slip model
K_c	Concrete stiffness
K_e	Epoxy stiffness
K_{sc}	Concrete shear stiffness
K_{se}	Epoxy shear stiffness
K_I	Initial stiffness of cohesive zone model in mode I
K_{II}	Initial stiffness of cohesive zone model in mode II
K_{I0}	Measured stiffness in mode I, used in BK law for determining mixed-mode energy
K_{II0}	Measured stiffness in mode I, used in BK law for determining mixed-mode energy
L	FRP bond length
L_e	Effective bond length
m	Coefficient in damage rate equation, related to static material properties
n	Coefficient in damage rate equation, related to static material properties
P	External load value
P_C	Load value of three-point bending test at point C
P_i	Load value at time step i
P_u	Ultimate load of single-lap shear test
p	Pressure

Q	Activation energy
R	Gas constant
s	Slip in single-lap shear test
s_i	Slip at position i in single-lap shear test
$\dot{s}$	Average slip rate of single-lap shear test
$\dot{s}_0$	Reference slip rate, 0.004 mm/s
s_1	Relative displacement of the cohesive zone model in normal direction
s_2, s_3	Relative displacement of the cohesive zone model in two in-plane shear direction
s_I	Relative separation of the cohesive zone model in tensile direction
s_{II}	Relative separation of the cohesive zone model in shear direction
Δs_{II}	Separation increment in mode II
s_{Id}	Mode I separation corresponding to mode I dynamic bond strength
$s_{II d}$	Mode II separation corresponding to mode II dynamic bond strength
s_{If}	Final separation in mode I
$s_{II f}$	Final separation in mode II
s_{Ist}	Mode I separation corresponding to mode I static bond strength
$s_{II st}$	Mode II separation corresponding to mode II static bond strength
$\dot{s}_I$	Separation rate of mode I
$\dot{s}_{II}$	Separation rate of mode II
$\dot{s}_m$	Separation rate of mixed mode
$\dot{s}_{I0}$	Reference separation rate of mode I
$\dot{s}_{II 0}$	Reference separation rate of mode II
s_m	Separation of the cohesive zone model in mixed mode
s_{mst}	Mixed-mode separation corresponding to the static bond strength
s_{md}	Mixed-mode separation corresponding to the dynamic bond strength
T	Absolute temperature
T_0	Reference temperature
t	Time
t_b	Specimen thickness of three-point bending test
t_c	Thickness of concrete
t_e	Thickness of epoxy
t_f	FRP thickness
u	Displacement of three-point bending test
u_B	Displacement corresponding to point B (three-point bending test)

u_C	Displacement corresponding to point C (three-point bending test)
u_f	Longitudinal displacement of FRP
u_{con}	Longitudinal displacement of concrete
W_F	External work in three-point bending test
W_m	Upper part of External work in three-point bending test
w	Concrete separation
$\dot{w}$	Concrete separation rate
$\dot{w}_0$	Reference separation rate
Δx	Distance between strain gauges in single-lap shear test
α	Parameter to control the damage shape of the softening range
α_1	Coefficient, given by 1.5
β_L	Coefficient of bond length
β_w	Width calibration factor
β_0	Constant, $1/\sqrt{2}$
γ	Fitting coefficient for mode II stiffness
ε	Strain
ε^P	Plastic strain
ε_f	Strain in FRP sheet
ε_{f0}	FRP strain measured near the free end (position 0)
$\varepsilon_x, \varepsilon_y, \varepsilon_z$	Strains in x, y, z directions respectively
$\varepsilon_{xy}, \varepsilon_{yz}, \varepsilon_{zx}$	Shear strains in xy, yz, zx directions respectively
$\vartheta_{xy}, \vartheta_{yz}, \vartheta_{zx}$	Poisson's ratios in xy, yz, zx directions respectively
δ, φ	Exponential coefficients
η	Coefficient related to dynamic enhancing effect in mode II
Δ	Increment component of dynamic enhancing effect in mode II
μ	Ultimate load ratio
σ	Stress
σ_1	Stress of the cohesive zone model in normal direction
σ_2, σ_3	Stress of the cohesive zone model in two in-plane shear direction
σ_c	Concrete tensile stress
σ_f	FRP axial stress
σ_I	Tensile stress of the cohesive zone model
σ_{II}	Shear stress of the cohesive zone model
σ_{Id}	Dynamic bond strength in mode I
σ_{IId}	Dynamic bond strength in mode II
σ_{md}	Dynamic bond strength in mixed mode
σ_{Ist}	Static bond strength in mode I
σ_{IIst}	Static bond strength in mode II

σ_{mst}	Static bond strength in mixed mode
$\sigma_x, \sigma_y, \sigma_z$	Stresses in x, y, z directions respectively
$\sigma_{xy}, \sigma_{yz}, \sigma_{zx}$	Shear stresses in xy, yz, zx directions respectively
$\Delta\sigma_y$	Deviatoric stress of yield surface in K&C concrete model
$\Delta\sigma_m$	Deviatoric stress of maximum surface in K&C concrete model
$\Delta\sigma_f$	Deviatoric stress of residual failure surface in K&C concrete model
τ	Shear bond stress
τ_{con}	Concrete shear strength
τ_f	Shear bond strength

Chapter 1 Introduction

1.1 Research background

Considering risks and security challenge from civil attacks in recent years, methods to strengthen reinforced concrete (RC) structures against impact and blast loads have been widely discussed and explored. As RC structures are commonly used in civilian and military facilities, appropriate methods for strengthening and rehabilitation are investigated by a variety of researchers and engineers. External bonding of fibre reinforced polymer (FRP) has been widely used as an effective technique for strengthening reinforced concrete (RC) structures in quasi-static and dynamic scenarios [1], as shown in Figure 1.1. With FRP strengthening system, additional protection and reduction on the damage extent were viewed in RC structures. FRP is a kind of composite material fabricated by saturating dry fibres with epoxy, which has advantages of easy application, high tensile strength, light weight and durability. To strengthen RC structures, FRP is usually made by several layers and bonded to the tension zone subjected to a tension load.

Figure 1.1 FRP strengthened RC structures [2]

The effectiveness of this method primarily depends on the interfacial bond behaviour between the FRP and the concrete substrate. Due to the inconsistent deformation in concrete and FRP laminate, relative slip and corresponding shear stress is created

between the concrete and FRP. With increase of structural deformation, the bond of the interface eventually fails by shear debonding, as shown in Figure 1.2(a). However, tensile fracture failure and mixed-mode fracture shown as Figures 1.2(b) and (c) are also noticed in many tests. This failure mode should be attributed to peel behaviour caused by concrete spalling or intermediate crack. To aid the FRP strengthening design in quasi-static scenarios, the bond behaviour of the interface has been widely investigated. Since the shear debonding is the primary failure mode [3], the shear bond behaviour has been extensively studied through single-lap and double-lap shear tests under quasi-static loads [1, 3-5]. Moreover, the normal bond behaviour under quasi-static load has been studied by some researchers [6, 7] using the three-point bending tests and tensile splitting tests.

Figure 1.2 FRP-concrete interface debonding in different directions

To the author's best knowledge, although local fracture failure between FRP and concrete has been noticed in many dynamic tests, investigations of such dynamic fracture behaviour were limited. It should be noted that, the dynamic debonding behaviour has not been as well investigated as the quasi-static loading cases. This lack of knowledge would limit the accuracy of high-fidelity numerical simulations and design of FRP strengthening under impact or blast loading regime.

1.2 Objective of this research

The quasi-static debonding behaviour of FRP-concrete interface has been studied by many researchers and applications of their results were proved to be effective and meaningful. Considering the lack of understanding in the dynamic debonding behaviour, this research conducted a great deal of single-lap shear tests, direct tension tests and three-point bending tests to study the FRP debonding in various scenarios. Theoretical studies and interface models were proposed to understand the fracture mechanism and simulate the interface bond. The objective of this research are summarised as follows:

(1) To comprehensively study the CFRP dynamic shear debonding behaviour, the present research experimentally investigates the dynamic enhancing effect of CFRP-concrete bond ranging from quasi-static to high loading rates (0.02 mm/s to about 2000 mm/s). The mechanical behaviour associated with the material parameters in the dynamic tests is analysed and understood.

(2) To do a theoretical study on the dynamic shear bond-slip behaviour, a continuum damage model is proposed for the FRP-concrete interface. Analytical solutions of dynamic shear bond strength and its corresponding slip are found through differentiating the constitutive equations. Energy release rate is obtained by numerical integration of these equations. A better understanding for the dynamic effect of the interface bond is achieved. The model is substituted into interfacial stress equilibrium equations to verify its ability to predict the FRP-concrete dynamic debonding process. Finally, the continuum damage model is converted into three-dimension scalar formulation to achieve its application in the FE simulations of CFRP-concrete debonding behaviour via a LS-DYNA user-defined material model.

(3) To characterise the tensile fracture behaviour of the FRP-concrete interface, two dynamic test procedures are designed. Dynamic direct tension tests and the notched three-point bending tests aim to provide the tensile bond strength and tensile fracture energy from quasi-static to dynamic load, respectively. Considering the possible material effect of concrete strength, concrete with two different strengths are used in specimen preparation. Similarly, carbon and glass fibre reinforced polymer (CFRP and GFRP) are bonded to concrete substrate to study the effect of different kinds of FRP.

(4) To continuously describe the dynamic fracture behaviour in various directions, a new coupled dynamic cohesive zone model (CZM) is proposed for the interface bond. This model includes elastic, rate-dependent hardening and exponential softening ranges. It is programmed into a commercial finite element code to model the FRP-concrete debonding behaviour in various tests, including single-lap shear tests, three-point bending tests and FRP strengthened RC beams under dynamic tests. Finally, the validated model is used in finite element analysis (FEA) of FRP strengthened reinforced concrete walls subjected to blast load.

1.3 Scope of research

This study mainly concerns dynamic fracture mechanics of FRP-concrete interface bond, including shear debonding, tensile delamination and proper equations to model these behaviours. For an overview of dynamic behaviour, the tests and numerical simulations are operated from quasi-static to impact loading regime. Theoretical study is conducted to understand the physical fracture mechanism. The local fracture process and general loading history are investigated and correlated.

The effect of material and geometric parameters on the interface bond behaviour are investigated to gain insight into the mechanical response. For shear debonding

behaviour, four main parameters are considered, viz FRP stiffness, FRP bond width and bond length, and concrete strength. Since tensile delamination is uniaxial and is simpler than the shear debonding, FRP bond width and length are not investigated here. Instead, different FRP materials and concrete strength are tested to view their effect on the tensile fracture behaviour.

The dynamic fracture behaviour varied from pure shear direction to pure tensile direction, including inclined fracture between shear and tension, can be simulated by a coupled cohesive zone model (CZM). This model is proposed to consistently describe the fracture behaviour of FRP-concrete interface from mode II to mode I. It is programmed into a commercial finite element code as a user-defined material model. The applications in finite element (FE) models verify its ability to simulate the bond behaviour in FRP debonding tests and FRP strengthened RC structures subjected to dynamic load.

The fracture behaviour of concrete-metal interface and interface bond of other materials is outside the scope.

1.4 Originality

This study investigated the dynamic fracture behaviour of FRP-concrete interface bond and the original works are summarized as follows:

- (1) A novel experimental method was proposed for dynamic single-lap shear tests by using a modified split Hopkinson pressure bar (SHPB) setup.
- (2) Effect of all the dominant material parameters in the single-lap shear tests has been investigated.
- (3) A dynamic continuum damage model for FRP-concrete shear debonding was proposed and validated by comparison with test results.
- (4) The FE models of FRP-concrete material with interface elements were proved to

be more accurate than the models using perfect bond at the impact regime.

(5) Dynamic direct tension tests were conducted for FRP-concrete interface.

(6) Dynamic increase factor (DIF) equations for the bond strength and fracture energy in tension have been provided.

(7) A rate-dependent CZM was proposed for FRP-concrete interfacial debonding in mode I/II and mixed mode.

(8) The CZM was programmed as the user-defined material model and has been used in the FE models of the FRP strengthened RC structures subjected to dynamic loading.

1.5 Layout of the thesis

The thesis contains seven chapters. The contents of the following chapters are briefly introduced as follows:

Chapter 2 provides an overview of previous studies on the FRP-concrete debonding behaviour. Dynamic tests for FRP strengthened RC structures are reviewed to understand the strengthening effect and their failure modes. Experiments and simulations on the shear debonding and tensile fracture are reviewed for understanding the bond behaviour of the interface. Moreover, cohesive zone models (CZMs) are introduced for its ability to model the various fracture behaviours.

Chapter 3 presents the experimental study on CFRP-concrete dynamic debonding behaviour, especially for tests under high slip rates. A novel experimental method at impact regime is proposed using a modified Split Hopkinson Pressure Bar (SHPB) set-up. Dynamic enhancing effect on ultimate load, shear bond strength and fracture energy are quantified for a wide range of slip rates varying from 0.02 mm/s to around 2000 mm/s. Effect of material parameters on the bond behaviour was better understood from a detailed experimental parametric study. Constitutive equations

were proposed to model the dynamic bond-slip behaviour of the CFRP-concrete interface.

Chapter 4 introduces an analytical continuum model to describe the nonlinear dynamic debonding behaviour of the FRP-concrete interface in a view of nonlinear fracture mechanics. Model analyses and regression test data are used to reveal the relationship between the model coefficients and material parameters of the actual specimens. The model is verified through comparing its predictions with existing experimental study. Finally, the continuum model is converted into three-dimension scalar formulae to achieve its application in the FE simulations of FRP-concrete debonding behaviour via a LS-DYNA user-defined material model.

Chapter 5 shows a study on the tensile fracture of FRP-concrete interface under dynamic load. The direct tension tests and the notched three-point bending tests are conducted to determine tensile bond strength and fracture energy, respectively. Dynamic enhancing effect on both properties are quantified for the loading rates ranging from 0.003 mm/s to 20 mm/s. Remarkable dynamic enhancing effect on the bond properties is found at the loading rate of 20 mm/s. The effect of material parameters is investigated. Dynamic increase factor (DIF) equations of the two bond properties are provided for the dynamic analysis.

Chapter 6 addresses a coupled dynamic CZM to analyse FRP-concrete mixed-mode separation. The model is a rate-dependent exponential damage model. The mixity assessment is conducted to evaluate the model response under various mixed-mode ratios. After presenting formulae of the model, it is verified by predicting various FRP-concrete dynamic separation tests, e.g. single-lap shear tests, three-point bending tests and beam tests. Finally, the model is used in finite element analysis (FEA) of FRP strengthened reinforced concrete walls under blast loading. According

to such detailed investigation, the model is validated for simulating FRP-concrete mixed-mode separation in FEA.

Chapter 7 concludes the current research and recommends future study.

Chapter 2 Literature review

2.1 Overview

To understand the strengthening effect of FRP to RC structures, the interfacial bond behaviour has been extensively investigated. In this chapter, an overview of existing studies on this subject are presented. Firstly, FRP materials and FRP strengthened RC structures are reviewed to understanding the strengthening method and its performance. After this, the studies on shear debonding of FRP-concrete interface bond are reviewed to understand the failure behaviour, including experiments, numerical simulations and proposed models. Thereafter, the tensile fracture behaviour of the interface bond is discussed to view the state of research progress for some infrequent failure modes. Finally, related cohesive zone models (CZM) are reviewed for the modelling of the fracture behaviour under shear, tensile and inclined load.

2.2 FRP strengthened RC structures

2.2.1 FRP strengthening method

Accidental danger and aging effect could threaten the service life and stability of RC structures. In order to protect these structures and extend their durability, researchers and engineers explored many effective strengthening techniques. External bonding of FRP has been widely used as an effective method to strengthen RC structures [1]. Such strengthening technique has the advantages of light weight, no corrosion in wet environment, easy installation, and little effect on the original structures.

FRP materials are composites made with continuous dry fibres saturating with epoxy. Epoxy has good mechanical properties and desired bond strength with concrete and steel substrates. According to the materials of dry fibres, such as glass, carbon and aramid, FRP composites could be named as glass fibre reinforced polymer (GFRP),

carbon fibre reinforced polymer (CFRP) and aramid fibre reinforced polymer (AFRP). Due to the directions of fibres, FRP is an anisotropic material. Normally, there are unidirectional FRP and cross-directional FRP. Typical mechanical properties of unidirectional GFRP, CFRP and AFRP are shown in Table 2.1. Compared with traditional strengthening material steel, FRP is quite light and has high erosion resistance.

Table 2.1 Typical mechanical properties of unidirectional GFRP, CFRP and AFRP

[8]

Composite materials	Fibre ratio by weight (%)	Density (kg/m ³)	Elastic modulus (GPa)	Tensile strength (MPa)
GFRP	50-80	1600-2000	20-55	400-1800
CFRP	65-75	1600-1900	120-250	1200-2250
AFRP	60-70	1050-1250	40-125	1000-1800

2.2.2 FRP strengthened RC structures in quasi-static regime

To increase the flexural capacity of RC beams, FRP plate is generally bonded to the tension zone of the beam subjected to tension [8]. Figure 2.1(a) shows a comparison of typical load-deflection curves between FRP strengthened RC beam and normal RC beam in four-point bending tests [8]. These curves indicate that strengthened RC beams can achieve higher stiffness and load capacity than normal RC beams. The failure mode of FRP rupture is shown in upper diagram in Figure 2.1(b), which means that the load capacity of FRP plate has been fully used in the loading process. However, if the FRP bonding end is not properly anchored and high interfacial stress exists in this area, the strengthening effect will not be fully utilised due to debonding of the FRP plate at the end, as shown in the lower diagram in Figure 2.1(b).

(a) Load-deflection curve

(b) Schematic diagram of failure modes

Figure 2.1 Failure behaviour of FRP strengthened RC beams under static load [2, 8]

Other four failure modes are also presented in Figure 2.1(b) for reference. When a major flexural is formed in FRP plated RC beam, interfacial debonding failure may be induced by flexural crack (intermediate crack) in the middle section and extend towards a plate end, as shown in Figure 2.1(b) [9]. If the steel shear reinforcement is limited and caused a critical diagonal crack in the beam, the propagation of this crack along the interface bond may induce the debonding of FRP plate. If the crack forms in a plate end and propagates to steel tension reinforcement, the specimen failure may start at concrete cover separation. When the interface bond between FRP and concrete works well and no debonding happens at the beam, the failure may also initiate at concrete crushing at the top [9]. Indeed, interfacial debonding of FRP plate has been reported to be one of major failure modes of such strengthened RC structures under quasi-static load [8].

The flexure strengthening of RC slabs is generally at tensile side, which is similar to the method of strengthening RC beams. As shown in Figure 2.2, FRP sheets are bonded along the longitudinal direction for simply supported one-way slab and cross directions for two-way slabs. The bonded FRP provided additional reinforcement to resist sectional flexure of the slabs. The strengthening system normally failed by intermediate crack induced debonding and plate end debonding [8, 10].

Figure 2.2 (a) One-way and (b) two-way slabs bonded with FRP plates

2.2.3 FRP strengthened RC structures in impact and blast regimes

Pham and Hao [11] presented an overview for FRP strengthened RC structures against impact loading. They found that FRP plate could improve the impact resistance of these structures by increasing load-carrying capacities, ductility and energy absorption ability. However, the FRP debonding mechanism under impact loads were still unclear and further studies were needed.

White et al. [12] investigated the strain rate effect on the 3-meter long CFRP strengthened RC beams. The loading rates ranged from 0.02 mm/s to 36 mm/s and corresponding induced strain rates in CFRP were from $3\text{e-}6\text{ s}^{-1}$ (slow rate) to $6.9\text{e-}3\text{ s}^{-1}$ (fast rate). The strengthening of CFRP plates significantly increased the stiffness and load capacity of the beam but decreased the ductility and energy absorption. Compared with control beam, CFRP strengthened beam shows 12% to 29% increase in yield capacity and 17% to 41% increase in maximum flexural capacity. The beam tested at fast rate showed 5% increase in stiffness, load capacity and energy absorption. Premature delamination of CFRP was noticed in their study, as shown in Figure 2.3(b), which could reduce the maximum load capacity of the beam. CFRP strains of the specimens failed by premature debonding were about $5000\text{ }\mu\epsilon$, and CFRP strains for other specimens with good bond were $6200\text{ }\mu\epsilon$. The failure mode of the beams with good bond was a combination of bond splitting and flexural-shear cracks.

(a)

(b)

Figure 2.3 (a) Schematic diagram of the strengthened beam and (b) CFRP debonding in actual tests [12]

Pham and Hao [13] experimentally studied FRP strengthened RC beams under static and impact loads. The impact loads were applied by a solid steel cylinder of 203.5 kg at the mid span. The strengthening methods of just using longitudinal FRP strips and using both longitudinal FRP strip and FRP U-wraps were examined, as shown in Figure 2.4. In their static tests, the longitudinal FRP strip initially debonded at the midspan and the crack propagated towards the beam end. For the specimens with U-wraps, the longitudinal FRP strip suffered tensile load and pulled the FRP U-wraps, which caused shear stress in these U-wraps. Therefore, some specimens failed by U-wraps fracture and FRP strip debonding. Only one specimen was failed by rupture of FRP strip. Compared with reference beam, the FRP strengthened beams showed increase of 8% to 59% on the maximum load capacity. It was found that using FRP U-wraps could maximize the strengthening effect of longitudinal FRP strips in the tests.

In the dynamic tests, local concrete crushing, inclined and vertical cracks were viewed. FRP U-wraps fracture and FRP strips debonding were also noticed in dynamic tests. For the failure mode, the flexure failure of RC beams under static loads might shift to the shear-flexure failure under impact loads. The specimens with U-

wraps presented less inclined cracks than the specimens just with FRP strips. This finding indicated that the failure between these two kinds of beams was determined by shear capacity. Compared with reference beam, the FRP strengthening reduced the maximum displacements and residual displacements by 16% to 35%. Due to the local shear failure at the impacting region, it was important to strengthen the expected impacting position.

Figure 2.4 Schematic diagram of strengthening method [13]

Bhatti et al. [14] used 300 kg steel striker to do drop-weight impact tests for FRP strengthened RC slabs, as shown in Figure 2.5(a). FRP plates were bonded at back side for flexural strengthening. They investigated the loading effect of single load and iterative load, respectively. They found that FRP strengthening system helped the RC slabs showing 20% to 30% increase on the maximum loading capacity in both static and dynamic regimes. Under a high loading velocity of 5 m/s, the tested RC slab showed punching shear cone and severe spalling. However, strengthened FRP sheet could prevent this spalling with peeled debonding in a limited area, as shown in Figure 2.5(b). Additionally, the bi-directional FRP system was found to be more effective than the uni-directional FRP sheet in dispersing tensile stress of the back surface. Through tests on 12 slabs, they concluded that the impact resistance of RC slabs was improved by bonding the FRP plates at back side. Compared with the

corresponding static load capacities, the dynamic load capacities of the slabs under impact were double. The bi-directional FRP strengthened slabs performed better than those of unidirectional FRP under the same load, since the tensile stress in bi-directional FRP could occur in two directions. The impact response of RC slabs was greatly influenced by the loading history.

(a) Test setup

(b) Comparison between RC slab and FRP strengthened RC slab

Figure 2.5 Experiment of Bhatti et al. [14]

Apart from the impact loads, blast loading effect on the FRP strengthened RC structures were also investigated around the world. A general review of FRP strengthened concrete and masonry structures subjected to blast loads has been

reported by Buchan and Chen [15]. It was found that FRP strengthening system could significantly enhance the blast resistance of RC structures through increasing the load capacity, ductility and reducing fragmentation. However, due to too many variables in the experiments, there is a lack of effective design method and design guideline for the strengthening method against blast loading. Therefore, an in-depth understanding and modelling of the structural behaviour under blast loads were needed. This research introduced two typical blast tests on FRP strengthened RC slabs.

Tanapornraweekit et al. [16] tested GFRP strengthened panels subjected to single, twice and triple independent blast loadings, respectively. Two normal RC panels without GFRP strengthening were prepared as control panels. The slabs were strengthened at top and bottom surfaces, and were supported in one way by a steel frame as shown in Figure 2.6. The strengthening effect of double sides of the RC slabs were evaluated. In the blast tests, they found that slabs would deflect towards the loading direction in the vibration stage after the blast loading. At this moment, incidence face of the slab experienced significant tensile stress, which needed strengthening by GFRP sheet.

Figure 2.6 Test setup by Tanapornraweekit et al. [16]

The GFRP strengthened panels under first explosion presented less damage than the control panels. Compared with the larger cracks seen on the control panels, many

small cracks appeared on the GFRP strengthened specimens. This phenomenon indicated how the GFRP contributed to the load resistance of the RC slabs. Following the first explosion, the slabs retrofitted with GFRP on both sides sustained the second explosion. Finally, the sample tests under triple explosions failed by a combination of large diagonal shear crack and peeling of the GFRP sheets, as shown in Figure 2.7. Concrete spalling was observed on the rear side. GFRP peeling was attributed to the propagation of diagonal shear cracks near the support and impact from fragmented concrete. They found that the maximum and residual deflections of specimen with a single layer of GFRP sandwich under explosion of 0.45 kg of TNT can be decreased by 45% and 82%, respectively. From the observation of these experiments, FRP debonding was found to be the main failure mode, together with diagonal shear cracks of concrete.

(a)

(b)

(c)

(d)

Figure 2.7 Failure of the slab after 3rd explosion: (a) side view; (b) shear crack at the support end; (c) back view; (d) front view [16]

Tanapornraweekit's study showed that FRP strengthening system could greatly improve the blast resistance of RC slabs. However, such strengthening effect might be less useful in close-in blasts. Orton et al. [17] conducted close-in blast tests with CFRP strengthened RC slabs. In Figure 2.8(b) and 2.8(c), at a scaled distance of $0.6 \text{ m/kg}^{1/3}$, comparison between the CFRP-strengthened slab and the control slab on the damage of both front face and rear side indicated that the CFRP helped decreasing damage level. On the rear side, the CFRP ruptured and less debris flew out of the CFRP-strengthened panel. The application of the composite successfully reduced residual deformation by 60%.

(a) Test setup

(b) Control slab

(c) CFRP strengthened slab

Figure 2.8 Experiment results after blasting at a scaled distance of $0.6 \text{ m/kg}^{1/3}$ [17]

At a scaled distance of $0.4 \text{ m/kg}^{1/3}$, the strengthened panel suffered a higher concentrated load and appeared a breach with a diameter of 450 mm, which was smaller than breach hole of the control slab (510 mm). Recorded by the high speed camera, the average velocity of the broken debris from the rear side of the panel was reduced from 64 m/s (control slab) to 47 m/s (CFRP strengthened slab). FRP failed by debonding and rupture. Orton's experiments showed that CFRP at the rear side reduced the residual deflection of the panels and the velocity of the flying debris. The effect of CFRP on these behaviours were worthy of further investigation.

(a) Test setup

(b) Control slab

(c) CFRP strengthened slab

Figure 2.9 Experiment results after blasting at scale of $0.4 \text{ m/kg}^{1/3}$ [17]

2.3 FRP-concrete shear debonding behaviour

2.3.1 Quasi-static studies

In early studies, there was no bond-slip models available for researchers to simulate the bond behaviour of the FRP-concrete interface and perfect bond was generally assumed in FE models. With the increasing use of FRP strengthened RC structures, the interface bond behaviour has been extensively investigated through experiments and numerical simulations. As the shear debonding is the primary failure mode in quasi-static scenarios, extensive studies have been conducted for this behaviour. The shear debonding behaviour was commonly studied through single-lap and double-lap shear tests, as shown in Figures 2.10(a) and 2.10(b), respectively.

Figure 2.10 Comparison between (a) single-lap shear test and (b) double-lap shear test

In these studies, specimens commonly failed due to concrete substrate cracking at a

layer a few millimetres below the FRP-concrete interface. Consequently, concrete strength f'_c significantly influences bonding behaviour and load-carrying capacity of specimens. The effect of FRP was also evaluated. Specimens with a greater FRP stiffness ($E_f t_f$) require a greater load to reach the same deformations compared to those with a smaller stiffness. Therefore, for the same failure strain, load-carrying capacities of specimens with higher FRP stiffness are significantly greater. Apart from the material properties, FRP bond width and bond length also affect the interfacial bond properties and the load-carrying capacity [1, 3, 4]. Chen and Teng [4] investigated different FRP bond widths b_f and incorporated a bond width coefficient β_w to predict the width effect due to different widths of FRP and concrete substrate (b_f and b_c). From researchers investigating the FRP-concrete bond length L [1, 4], it was found that any increase in bond length beyond the effective bond length L_e would not improve the load-carrying capacity. Accordingly, Chen and Teng [4] also proposed an equation to predict the effective bond length. In summary, four main parameters govern the load-carrying capacity and the bond-slip behaviour of FRP strengthened concrete in quasi-static tests, namely, FRP stiffness $E_f t_f$, the width and length of the FRP-concrete bonding interface (b_f and L) and the concrete strength f'_c .

Based on regression study of numerous published experimental data, Chen and Teng [4] proposed a practical engineering model to predict the ultimate load of single-lap shear test in quasi-static regime. Later Yao et al. [1] performed a series of tests with various material and geometric parameters to assess Chen's model. According to Yao's study, the model proposed by Chen and Teng could predict quasi-static ultimate load accurately. The specimens failed due to concrete cracking at a depth of a few millimetres below the FRP-concrete interface, as show in Figure 2.11.

(a) Test setup

(b) Failure specimen

Figure 2.11 Failure specimen with a thin layer concrete adjacent to FRP [1]

The ultimate loads of single-lap shear tests could be predicted using Chen's model [4] as follows.

$$P_u = 0.427 \beta_w \beta_L \sqrt{f'_c} b_f L_e \quad (2.1)$$

where P_u is the ultimate load on FRP sheet; β_w is the bond width coefficient; β_L is the bond length coefficient; f'_c is the concrete cylinder strength; L_e is the effective bond length; b_f is the bond width of FRP sheet.

The bond width coefficient is given by

$$\beta_w = \sqrt{\frac{2 - b_f / b_c}{1 + b_f / b_c}} \quad (2.2)$$

where b_c is the width of concrete sample.

The effective bond length is:

$$L_e = \sqrt{\frac{E_f t_f}{\sqrt{f'_c}}} \quad (2.3)$$

where E_f is the Young's modulus of FRP, t_f is the thickness of FRP.

The bond length related coefficient is:

$$\beta_L = \begin{cases} 1, L \geq L_e \\ \sin \frac{\pi L}{2L_e}, L < L_e \end{cases} \quad (2.4)$$

where L is the bond length of FRP sheet.

Such investigation also included numerical simulations. Lu et al. [3, 18] simulated the single-lap shear tests via finite element analysis (FEA). The FE models evaluated the samples with concrete strengths varying from 20 to 73 MPa and FRP stiffnesses from 20 to 60 GPa·mm. After the simulations, the predicted ultimate loads were compared with those of actual tests, which indicated a good average predicted-to-measured ratio of 1.001.

Following the investigation of ultimate load in the shear tests, the local failure behaviour of the interface bond in these tests was studied in detail. The nonlinear bond stress-slip response in local position has been found in actual tests, which includes ascending (O-A-B) and descending ranges (B-C), and determines failure by reaching fracture energy, as presented in Figure 2.12. To describe such nonlinear fracture process, the local bond-slip models were derived from experiments and numerical simulations [5, 19, 20].

Figure 2.12 Typical sketch of the bond stress-slip curve

Based on the precise simulations, Lu et al. [3] proposed three models to describe the local bond-slip curves, including a precise model, a simplified model and a bilinear model. Since the simplified model is popular in related research areas, it is described in more detail below.

The ascending branch is given by:

$$\tau = \tau_f \sqrt{\frac{s}{s_{If}}}, \text{ if } s < s_{If} \quad (2.5a)$$

where τ_f is peak bond stress, s is slip and s_{If} is slip corresponding to the peak bond stress.

The descending branch is given by:

$$\tau = \tau_f \exp[-\alpha(\frac{s}{s_{If}} - 1)], \text{ if } s \geq s_{If} \quad (2.5b)$$

where parameter $\alpha = \frac{1}{\frac{G_{If}}{\tau_f s_{If}} - \frac{2}{3}}$; G_{If} is the interfacial fracture energy and is

determined by $G_{If} = 0.308 \beta_w^2 \sqrt{f_t}$.

The maximum bond stress and corresponding slip are determined by

$$\tau_f = \alpha_1 \beta_w f_t, \quad s_{lst} = 0.0195 \beta_w f_t \quad (2.6)$$

Cottone and Giambanco [21] presented a theoretical study for the quasi-static shear debonding behaviour. The FRP laminate was considered as a beam connected to a rigid substrate through an interface bond. The interface bond was modelled through elasto-plastic theory with softening behaviour. In their analytical study, it has been proved that there exists an effective bond length L_e which the increase of the bond length over L_e could not produce any increment of the ultimate load. Moreover, the size effect law of different FRP bond width was also derived by the analytical method in their study.

2.3.2 Dynamic studies

Due to the impulsive nature of impact or blast loading, the response of the FRP-concrete interface is not the same as that observed under quasi-static regime. It should be noted that, the dynamic bond-slip behaviour has not been as well investigated as the quasi-static loading cases. Only a few experiments have been carried out to study the dynamic shear debonding failure under relatively low loading rates.

Shi et al. [22] experimentally investigated the dynamic debonding behaviour of FRP-concrete interface with a servo-hydraulic test machine for loading rates below 20 mm/s, as shown in Figure 2.13. They investigated the material effect of FRP type and concrete strength. However, FRP geometric parameters such as bond width and bond length were not included in their studies.

Figure 2.13 Dynamic double-lap shear tests conducted by Shi et al. [22]

Strain gauges attached on the FRP surface were used to measure the strain distributions during the test process. Based on the measured strain history and FRP stiffness, the shear bond stress could be derived from an equation as follows.

$$\tau = |E_f t_f \frac{d\varepsilon_f}{dx}| \quad (2.7)$$

Based on regression study of test results from quasi-static to dynamic regime, the prediction equations for dynamic fracture energy and dynamic bond strength are given as follows.

$$\frac{G_{II d}}{G_{II st}} = \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_0}\right)^\delta, \quad \frac{\sigma_{II d}}{\sigma_{II st}} = \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_0}\right)^\varphi \quad (2.8)$$

where $G_{II d}$ and $G_{II st}$ are dynamic fracture energy and static fracture energy, respectively, $\sigma_{II d}$ and $\sigma_{II st}$ are dynamic bond strength and static bond strength, $\dot{\varepsilon}$ is strain rate, δ and φ are exponential coefficients.

Similar tests were also conducted by Shen et al. [23], and Salimian and Mostofinejad

[24]. Shen et al. [23] increased the loading rate of double-lap shear tests to 70 mm/s but no material and geometric parameter was considered. Salimian and Mostofinejad [24] used the digital image correlation (DIC) method to measure the strain data of dynamic single-lap shear tests at loading rates up to 1mm/s. The effect of concrete strength in the range of 22.3 to 49.1 MPa was also investigated in their study. Pereira and Lourenco [25] used a drop weight to apply impact loads on glass fibre reinforced polymer (GFRP)-masonry specimens. They did not use a concrete substrate in their study. In addition, majority of tests achieved were lower than 800 mm/s.

Additionally, an analytical study conducted by Caggiano et al. [26] provided an interface model to simulate the dynamic debonding behaviour of FRP-concrete interface at loading rates ranging from 0.07 to 70 mm/s. The softening range was controlled by an exponential damage function and the dynamic increase effect was determined by material viscosity. However, the determination methods for the coefficient in damage function and viscosity parameter were not provided. Its applicability for higher loading rates has not been verified.

2.4 Tensile fracture behaviour of FRP-concrete interface bond

The FRP tensile delamination from the concrete substrate can damage the bond at a lower loading level [6]. However, such tensile fracture failure, which was commonly found in dynamic tests [12, 14, 17, 27], was investigated by limited researchers. It was found that two key properties determined the tensile fracture behaviour, viz tensile bond strength σ_{Ist} and the tensile fracture energy G_{Ist} [7]. As the properties were hard to be obtained from a simple test, different test procedures have been proposed in the past. To clarify their difference, these test procedures are referred to tensile tests and fracture tests for determining σ_{Ist} and G_{Ist} , respectively.

2.4.1 Tensile tests

For concrete material, there are three test methods to measure the tensile strength, viz split cylinder test, beam bending test and direct tension test [28]. The tensile stress in the first two methods is imposed indirectly on the test specimens. In split test, concrete cylinder is placed horizontally and is subject to compression load at the two edge along the diametrical direction. Beam bending test works by adding three-point or four-point bending load on the concrete beam. However, direct tension test can measure the concrete tensile strength under uniaxial tensile load, which is more rational and reliable [29]. For FRP-concrete interface, to the author's best knowledge, there is few tests conducted for the tensile bond strength. Coronado and Lopez [7] conducted split tests for the cylinder and prismatic specimens with a layer of epoxy at the centre line, as shown in Figure 2.14. The sensitivity of tensile bond strength to the type of epoxy and specimen geometry was investigated. It was found that different kinds of epoxy did not affect the result. However, due to fabrication imperfection in cylinder specimens, prismatic specimens showed higher bond strengths than cylinder specimens. Therefore, the author's suggested prismatic specimens for determining tensile bond strength. The derived splitting tensile strength for the concrete-epoxy interface was close to that of plain concrete.

Figure 2.14 Schematic diagram of splitting tests (a) cylinder specimen and (b) prismatic specimens [7]

2.4.2 Fracture tests

The fracture energy is necessary in the analysis of crack softening behaviour. Therefore, a few fracture tests have been carried out to study the tensile fracture energy of the FRP-concrete interface bond [6, 7, 30-32]. According to the assumptions used for calculating the interfacial fracture energy, these studies could be divided into linear elastic (LEFM) and nonlinear fracture mechanics (NLFM) methods.

In the LEFM method, Karbhari and Engineer [30] proposed a peeling test to characterize the tensile fracture of FRP-concrete interface bond, as shown in Figure 2.15. This setup allowed to alter the peeling angle and vary the loading rate. In their analysis, FRP and concrete materials were assumed to be linear elastic and the stress field around the crack was calculated using elastic theory. The fracture process was assumed to be linear elastic and the fracture energy was determined from a global energy balance. With an increase of peeling angle from 15° to 120° , the total fracture energy of GFRP-concrete interface increased from 420.64 J/m^2 to 913.82 J/m^2 . In this process, fracture energy in mode I reduced from 413.60 J/m^2 to 228.35 J/m^2 , but fracture energy in mode II increased from 7.04 J/m^2 to 685.47 J/m^2 . To the author's best knowledge, fracture energy obtained from the peeling test have not been used to predict the fracture behaviour of FRP strengthened concrete structures. It was due to that linear elastic assumption was not met by concrete material which had nonlinear fracture response.

Figure 2.15 Schematic diagram of test setup proposed by Karbhari and Engineer [30]

Giurgiutiu et al. [32] and Wan et al. [31] used a modified double cantilever beam (DCB) test to determine the fracture energy of FRP-concrete interface bond, in which FRP laminate was assumed to work as a beam, as shown in Figure 2.16. To satisfy the requirement of study under mixed-mode loading, the basement could be adjusted to vary the loading angle. In pure mode I, Giurgiutiu et al. [32] reported a range of strain energy release rate from 228 J/m^2 to 332 J/m^2 and Wan et al. [31] reported fracture energy of 273 J/m^2 to 545 J/m^2 . Significant variation was found in their test results. Similar to the method used by Karbhari and Engineer [30], the researchers assumed that fracture process followed LEFM. However, as mentioned by Hillerborg [33] and Petersson [34], the nonlinear fracture zone behind the crack tip was relatively larger in the geometry of material tests than that of structure tests. Therefore, NLFM method was needed to determine the crack process.

Figure 2.16 Schematic diagram of the modified double cantilever beam test [32]

Since the fracture of the bond commonly occurred in the concrete layer, the nonlinear fracture behaviour could not be evaluated by the LEFM method [7]. For the NLFM method, the notched three-point bending tests have been used by Qiao and Xu [6] and their test setup is shown in Figure 2.17. This setup referred to the method for determining concrete fracture energy by Hillerborg [35] and Petersson [34]. A layer of FRP laminate was inserted at the mid span to produce FRP-concrete tensile fracture behaviour. For CFRP-concrete interface bond under loading rates from 0.12 to 3 mm/min, the fracture energy was varied from 81.6 N/m to 115.0 N/m. The variation of loading rate in this range did not have a significant effect on the fracture energy. Their test results indicated that the interfacial fracture energy was similar to that of plain concrete in magnitude. The results of GFRP-concrete interface bond were slightly higher than those of CFRP-concrete interface, which were reported between 83.6 N/m to 125.8 N/m.

Figure 2.17 Schematic diagram of the notched three-point bending test setup [6]

After their tests, Qiao and Chen [36] used cohesive zone model (CZM) to simulate the fracture behaviour of the notched three-point bending tests in FEA. The influence of material parameters on the failure modes and load-carrying capacity were evaluated, including interface bond strength, interface fracture energy, concrete strength and concrete fracture energy. The simulation results identified three failure modes: (1) total debonding in concrete-adhesive interface (a weak bond), (2) total

debonding in concrete layer (a strong bond), and (3) debonding in both concrete and adhesive. The failure modes could change when the concrete parameters or interface parameters were modified.

Coronado and Lopez [7] conducted similar three-point bending tests to determine the fracture energy of concrete-epoxy interface bond. The fracture energy G_{Ist} was reported in a range from 107.14 N/m to 130.36 N/m, which was close to the fracture energy of plain concrete of 117.04 N/m (the control specimen). The size effect fracture energy G_s of concrete-epoxy interface was varied between 34 N/m and 52.2 N/m. In their study, both size-effect fracture energy G_s and cohesive fracture energy G_{Ist} were derived and discussed, as shown in Figure 2.18. The test results indicated that concrete-epoxy interface and plain concrete had different size-effect fracture energy in magnitude but similar cohesive fracture energy. Finally, they suggested using size-effect fracture energy as the characteristic fracture energy of the interface bond.

Figure 2.18 (a) Softening curve and (b) approximated bilinear curve [7]

2.5 Numerical modelling of FRP debonding behaviour

2.5.1 Models for concrete

In analytical studies of FRP debonding behaviour, concrete was usually assumed to be rigid substrate, since the deformation in concrete substrate is minor [21, 37].

Moreover, to model the flexure behaviour of FRP strengthened RC structures, concrete was considered as an elastic material [38, 39].

In finite element (FE) simulations, concrete models were more complicated than those in analytical studies. Qiao and Chen [36] used tensile plastic damage model to simulate the tensile fracture of concrete material in the three-point bending tests. Fracture energy was used to determine the post-peak failure behaviour. As the compressive damage in concrete was ignored, there was no softening defined for compressive stress-strain relation. Coronado and Lopez [40], and Tao and Chen [41] used the concrete damage plasticity model proposed by Lubliner et al. [42] and modified by Lee and Fenves [43] to model the concrete plastic and damage behaviour in single-lap shear tests. Under uniaxial loading, the stress-strain relation of the damage model was expressed as

$$\sigma = (1 - D)E_c(\varepsilon - \varepsilon^P) \quad (2.9)$$

where σ , D , E_c , ε and ε^P represent stress, damage factor, initial stiffness, total strain and plastic strain, respectively. The damage behaviour only occurred in the softening range and the stiffness in this range was proportion to the material strength as follows.

$$(1 - D) = \frac{E}{E_c} = \frac{\sigma}{f} \quad (2.10)$$

where f is the tensile or compressive strength of concrete material.

The concrete plasticity models introduced by Qiao and Chen [36] and Coronado and Lopez [40] were applicable for numerical simulations in quasi-static scenarios. For FRP strengthened RC structures subjected to dynamic and blast loads, dynamic response of concrete material was important in the numerical simulations. The

concrete damage model proposed by Malvar et al. [44] was popular for the dynamic simulations and it was used by Nam et al. [45] and Mutalib and Hao [46] in the simulations of FRP strengthened RC slabs subjected to blast loads. It employed three independent strength surfaces, viz initial yield surface, maximum failure surface and residual surface. These three surfaces are sequentially determined by the following equations.

$$\Delta\sigma_y = a_{0y} + \frac{p}{a_{1y} + a_{2y}p} \quad (2.11)$$

$$\Delta\sigma_m = a_0 + \frac{p}{a_1 + a_2p} \quad (2.12)$$

$$\Delta\sigma_f = \frac{p}{a_{1f} + a_{2f}p} \quad (2.13)$$

where p is the pressure, $\Delta\sigma_y$, $\Delta\sigma_m$ and $\Delta\sigma_f$ are the deviatoric stresses of the yield, maximum and residual failure surfaces, respectively, and the other factors are material parameters to be determined from experimental tests. The dynamic concrete strength was determined by static concrete strength multiplying dynamic increase factor (DIF). In the commercial FEA software LS-DYNA [47], Malvar's model included an option to automatically generate material properties based on the inputted concrete strength. The DIF of concrete material was referred to the CEB FIP 1990 code [48].

2.5.2 Models for FRP

Due to elastic behaviour along the fibre direction, FRP was commonly treated as a linear elastic material in analytical studies [37-39, 49] and FE simulations [10, 40, 50]

of material tests and structure tests. Since some researchers found the importance of the anisotropic properties in FRP and intended to model its realistic behaviour in different directions, they used other materials models in the numerical simulations.

Qiao and Chen [36] considered anisotropic behaviour of FRP plate and used orthotropic linear elastic model to simulate its mechanical properties in FE models.

Its constitutive law is given as

$$\begin{bmatrix} \varepsilon_x \\ \varepsilon_y \\ \varepsilon_z \\ \varepsilon_{xy} \\ \varepsilon_{yz} \\ \varepsilon_{zx} \end{bmatrix} = \begin{bmatrix} \frac{1}{E_x} & -\frac{\nu_{yx}}{E_y} & -\frac{\nu_{zx}}{E_z} & 0 & 0 & 0 \\ -\frac{\nu_{xy}}{E_x} & \frac{1}{E_y} & -\frac{\nu_{zy}}{E_z} & 0 & 0 & 0 \\ -\frac{\nu_{xz}}{E_x} & -\frac{\nu_{yz}}{E_y} & \frac{1}{E_z} & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1}{G_{xy}} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1}{G_{yz}} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{1}{G_{zx}} \end{bmatrix} \begin{bmatrix} \sigma_x \\ \sigma_y \\ \sigma_z \\ \sigma_{xy} \\ \sigma_{yz} \\ \sigma_{zx} \end{bmatrix} \quad (2.14)$$

where E_x, E_y, E_z are Young's moduli at x, y, z directions respectively. $\nu_{xy}, \nu_{yz}, \nu_{zx}$ are the corresponding Poisson's ratios. Note that $\frac{\nu_{xy}}{E_x} = \frac{\nu_{yx}}{E_y}$, $\frac{\nu_{zx}}{E_z} = \frac{\nu_{xz}}{E_x}$, $\frac{\nu_{zy}}{E_z} = \frac{\nu_{yz}}{E_y}$. G_{xy}, G_{yz}, G_{zx} are the shear moduli at xy, yz, zx directions respectively.

In some simulations of FRP strengthened RC slabs subjected to blast loads, FRP was modelled as an orthotropic elastic material to represent its strengthening effect on cross directions [45, 46]. Moreover, composite failure criterion was defined in FRP material models to simulate the FRP rupture behaviour in the blast tests. The Chang-Chang failure criterion [47, 51] was a popular breakage criterion for composites and it was adopted by Nam et al. [45] and Mutalib and Hao [46]. The Chang-Chang criterion was given as follows.

For tensile matrix mode $\sigma_x > 0$,

$$e_t = \left(\frac{\sigma_x}{f_{tf}} \right)^2 + \left(\frac{\sigma_{xy}}{f_{sf}} \right)^2 - 1 \begin{cases} \geq 0 & \text{failed} \\ < 0 & \text{elastic} \end{cases} \quad (2.15)$$

where e_t is a failure index of tensile and shear stresses combined effect; σ_x is the normal stress; σ_{xy} is the shear stress; f_{tf} is the tensile strength; f_{sf} is the in-plane shear strength.

For compressive mode $\sigma_x < 0$,

$$e_c = \left(\frac{\sigma_x}{f_{cf}} \right)^2 - 1 \begin{cases} \geq 0 & \text{failed} \\ < 0 & \text{elastic} \end{cases} \quad (2.16)$$

where e_c is a failure index of compressive and shear stresses combined effect, f_{cf} is the compressive strength.

2.5.3 Models for the interface bond

Various mechanical methods were developed in numerical simulations to analyse material fracture, including cohesive zone model (CZM), extended finite element method (XFEM) and mesh free method. Among these methods, CZM is the simplest and most time efficiency which has been extensively used for simulating fracture issues of different materials, such as composite materials, metallic joints, concrete and masonry structures [52, 53]. CZM describes the traction-separation relationship in the fracture zone until the fracture occurs and new free surfaces appear [54]. Therefore, its performance is quite suitable to model the bond failure between FRP and concrete. To study FRP-concrete debonding behaviours, CZM is used to model the bond between FRP and concrete in quasi-static numerical simulations [6, 10, 55] and analytical methods [38, 39, 49].

Several shapes have been proposed in literature, such as bilinear and trapezoid models, exponential models and viscoplastic models. Qiao and Chen [36], De Lorenzis and

Zavarise [49], and De Lorenzis et al. [38] used bilinear models to simulate the fracture behaviour of FRP-concrete interface bond, including fractures in normal, shear and mixed-mode directions. Wang [39] applied a modified bilinear CZM to model the interface debonding under mixed-mode load. Bilinear models are frequently used due to their simplicity, as shown in Figure 2.19, which assume that both ascending and softening ranges are linear.

Figure 2.19 Stress-separation law of a typical bilinear model: (a) in normal direction and (b) in shear direction

Ouyang and Wan [55] used an exponential model proposed by Xu and Needleman [56] to simulate the FRP debonding under peeling load. The stress-separation curve of this exponential model is shown in Figure 2.20.

Figure 2.20 Stress-separation curve of the exponential model

The stress-separation equation in normal direction is given as Eq. (2.17). Since the

shear separation was minor in their study, compared to the normal separation, the shear separation in peel test was ignored.

$$\sigma = e\sigma_{\max} \frac{s_I}{s_{Ist}} e^{-\left(\frac{s_I}{s_{Ist}}\right)} \quad (2.17)$$

where e is a mathematical constant, s_I is normal separation and s_{Ist} is the separation corresponding to the normal strength.

Apart from bilinear and exponential models, viscoplastic models focus on physical description of dynamic behaviour by including rate effects into material parameters. Corigliano and Racci [57] and Lorefice et al. [58] gave their models for composite and concrete joints, respectively. Due to complicated definition of viscosity, coefficients of these models which control damage behaviour still need much study to correlate with mixed-mode crack of FRP-concrete interface.

2.6 Summary

This section reviewed existing studies on FRP-concrete debonding behaviour at quasi-static and dynamic regimes. The quasi-static debonding behaviour has been well-investigated via experiments, numerical simulations and theoretical studies. Based on the literature review, the author found that shear bond-slip behaviour of FRP-concrete interface with a much higher loading rate of up to 2000 mm/s has not been well studied. The tensile debonding behaviour under dynamic loads also needs to be determined through actual tests. The lack of knowledge on dynamic debonding behaviour would limit accuracy of high-fidelity numerical simulations and design of FRP strengthening system under dynamic and blast loading regime.

Apart from the actual tests to determine the dynamic debonding behaviour, the theoretical study and numerical simulations for modelling this response are required to understand the failure mechanism and provide reasonable predictions. In the

reviewed quasi-static studies, the failure behaviours of FRP-concrete interface in tension and shear show large difference. Qiao and Xu [6] and Coronado and Lopez [7] experimentally studied the bond failure in mode I and found that both the quasi-static bond strength and the fracture energy were close to those of concrete material. However, according to the shear debonding studies conducted by Lu et al. [3] and Dai et al. [5], the quasi-static shear bond strength could be a bit higher than that in normal direction and quasi-static fracture energy in shear separation would be 3 to 6 times larger than that in normal direction. Such significant differences between mode I and mode II need a flexible CZM to fit both separation modes as well as give realistic response under mixed mode.

To the author's best knowledge, existing numerical simulations for FRP-concrete mixed-mode debonding performance are only analysed with bilinear CZMs. Most existing exponential models were developed for metallic joint and composite laminates. In addition, the dynamic enhancing effect of these introduced bilinear models and exponential models mainly considers the dynamic response of adhesive joints and metallic joints, which are different from the FRP-concrete bond joint. For metallic joints and composite laminates, failure is noticed in adhesive. However, the debonding behaviour of FRP-concrete joint generally failed in the concrete layer [3] and its dynamic response is closely related to material properties of concrete and FRP [22]. For modelling the FRP-concrete bond subjected to dynamic loading, it is important to include material properties of both adherents.

Chapter 3 Experimental study on CFRP-concrete dynamic debonding behaviour

3.1 Introduction

External bonding of fibre reinforced polymer (FRP) has been widely used as an effective technique for strengthening reinforced concrete (RC) structures [1]. The effectiveness of this method primarily depends on the interfacial shear bond behaviour between the FRP and the concrete substrate. Shear bond behaviour in quasi-static scenarios has been extensively studied through single-lap and double-lap shear tests [1, 3-5]. Due to impulsive nature of impact or blast loading, the response of the FRP-concrete interface is not the same as that observed under quasi-static regime. Orton et al. [59] mentioned that the strain rate reached by the FRP material on strengthened RC structures in close-in blast loading was about 8 s^{-1} . To achieve this strain rate in dynamic shear tests, loading rates of over 800 mm/s need to be applied for a 100 mm FRP bond length. It should be noted that, the dynamic bond-slip behaviour has not been as well investigated as the quasi-static loading cases. This lack of knowledge would limit the accuracy of high-fidelity numerical simulations and design of FRP strengthening under impact or blast loading regime.

Only a few experiments have been carried out to study the dynamic shear debonding failure of FRP-concrete interface under relatively low loading rates. As mentioned in Section 2.3.2, Shi et al. [22] investigated the effects of FRP material and concrete strength for loading rates below 20 mm/s with a servo-hydraulic test machine. However, FRP geometric parameters such as b_f and L were not included in their studies. Shen et al. [23] increased the loading rate to 70 mm/s but no material and geometric parameter was considered. Pereira and Lourenco [25] used a drop weight to apply impact loads on glass fibre reinforced polymer (GFRP)-masonry specimens. However, they did not use a concrete substrate in their study. In addition, majority of

tests achieved were lower than 800 mm/s. To the author's best knowledge, shear bond-slip behaviour of FRP-concrete interface with a much higher loading rate of up to 2000 mm/s has not been reported. Therefore, in this work a novel experimental method for high loading-rate tests using a modified Split Hopkinson Pressure Bar (SHPB) set-up was carried out. Additionally, CFRP was selected among the different types of FRP for its light weight, large stiffness and high strength, as well as its wide applications in FRP strengthened RC structures [17, 27].

To comprehensively study the CFRP dynamic debonding behaviour, the present programme investigated the dynamic enhancing effect of CFRP-concrete bond ranging from quasi-static to high loading rates (0.02 mm/s to about 2000 mm/s). It is noteworthy that when considering the physical debonding process, slip rate is the practical debonding velocity. This is different from strain rate, which is dependent on a specific element size [10, 52]. Therefore, slip rate has been used in dynamic studies of Pereira and Lourenco [25] and Li [10]. Following these studies, the author also adopted slip rate in the current study to describe the dynamic bond-slip behaviour. The mechanical behaviour associated with the four parameters ($E_f t_f$, b_f , L and f'_c) in the dynamic tests was analysed through experimental tests. Finally, for applications in finite element analysis (FEA), constitutive equations of the dynamic bond-slip behaviour were proposed.

3.2 Experimental programme

A total of 75 single-lap shear tests have been conducted to investigate the dynamic bond-slip behaviour with loading rates ranging from 0.02 mm/s to about 2000 mm/s. The quasi-static tests with a loading rate of 0.02 mm/s served as the baseline for the analysis of dynamic tests. Based on different material and geometric features, the specimens were divided into six groups.

In accordance with the loading rate, these tests were classified as quasi-static (0.02 mm/s), low loading-rate (0.8-80 mm/s) and high loading-rate tests (800-2000 mm/s), respectively. The quasi-static and low loading-rate tests were conducted using an MTS servo-hydraulic test machine with displacement control, while the high loading-rate tests were performed with a modified SHPB set-up. In the following sections, the specimen preparation and the respective test set-up for the low and the high loading rates will be described.

3.2.1 Specimen properties

As shown in Figure 3.1, the specimen consisted of a CFRP sheet bonded to a concrete block. The specimens were prepared by (a) grinding the concrete surface to reveal the aggregates, (b) cleaning the exposed substrate using compressed air, and (c) attaching the CFRP sheet to the concrete block using the epoxy provided by Nitowrap FRC reinforcement system [60].

Figure 3.1 Dimensions of a typical specimen: (a) top view, (b) front view, (c) side view and (d) actual specimen

The concrete block was cast in the same dimensions (100 mm × 100 mm × 190 mm) for all 75 specimens. Two concrete grades (C40 and C60) were used to study the influence of concrete strength. Table 3.1 shows the mix proportions of the two concrete grades and respective strength of three cylinder samples (Φ150 mm by 300 mm long). The 28-day mean cylinder strengths f'_c were measured as 32.0 MPa and 53.8 MPa for the two concrete grades. To suit local practice which adopts cube strength, they were designated as C40 and C60 to indicate equivalent cube strength, respectively.

Table 3.1 Mix proportions of concrete (kg/m³) and corresponding strengths

Grade	Cement	Sand	Water	Coarse aggregate	Measured strengths (MPa)	Average cylinder strength (MPa)	SD ¹ (MPa)
C40	350	720	168	1070	32.5/31.8/31.7	32.0	0.4
C60	535	676	162	930	54.7/53.0/53.6	53.8	0.9

¹means standard deviation (SD)

The CFRP sheet used in this study was made with a unidirectional fabric of carbon fibres and an epoxy which were included in the Nitowrap FRC 300 reinforcement system. The product brochure [60] indicated that the dry fibres and the epoxy had tensile strength of 3400 MPa and 50 MPa, respectively. The single and double layers of CFRP were produced with a thickness t_f of 0.5 mm and 1.0 mm, respectively. After fabricating a single layer of CFRP sheet, tensile strength f_{tf} and elastic modulus E_f were measured as 750 MPa and 82.7 GPa, respectively, using the test method described in ASTM D3039/D3039M-00 [61]. Since the CFRP stiffness is equal to its elastic modulus multiplied by its thickness ($E_f t_f$), the stiffness values of single and double layers were 41.4 and 82.7 GPa·mm, respectively.

To investigate the effect of CFRP parameters on the specimens, CFRP sheets were varied including the stiffness ($E_f t_f=41.4$ or 82.7 GPa·mm), bond width ($b_f=26$ or 33

mm) and bond length ($L=75, 100$ or 150 mm). For specimens with three different lengths, four, five and six strain gauges were arranged on the CFRP surface within the bonded part at equal spacing, respectively. Through varying the concrete and CFRP parameters, these specimens could be used to investigate the influence of the four parameters ($E_f t_f$, b_f , L and f'_c) in the dynamic tests.

3.2.2 Quasi-static and low loading-rate tests

Figure 3.2 shows the test set-up for the quasi-static and low loading-rate tests (0.02-80 mm/s). The specimen was placed vertically in an MTS servo-hydraulic test machine and its CFRP sheet was clamped by the lower grip and loaded with displacement control. The load cell in the actuator was used to record the applied force, and strain gauges on the CFRP surface were arranged to measure axial strain of the bonded area. A high-frequency data logger was used to collect the strain measurements. To minimise eccentric loading, a fixture was manufactured to align the bonded interface of the specimen with the loading direction, as shown in Figure 3.2.

Figure 3.2 Bond-slip test set-up using MTS servo-hydraulic test machine: (a) side view, (b) front view

Information on the loading, material and geometric parameters of these tests is given in Table 3.2. It includes the loading rate, concrete strength f'_c , stiffness and thickness of the CFRP ($E_f t_f$ and t_f), width and length of the CFRP-concrete bonding interface (b_f and L). The nomenclature of the specimens contained information on the loading rate, indicated by M1, M2, M3 or M4 prefix (0.02, 0.8, 8.0 and 80 mm/s), followed by the concrete grade as C40 or C60. The final number, ranging from 1 to 5, was used to distinguish different sizes of FRP sheets. Three samples (A, B, C) were prepared for each testing configuration to evaluate repeatability. For specimens tested under low loading rates (M1 and M2), the loads were imposed with a ramp mode which incorporated the acceleration range from 0 to the target rate in the entire loading process. However, to maintain a stable loading rate in the dynamic regime (M3 and M4), the actuator was accelerated to the loading rate first and then the sample was loaded.

Table 3.2 Test items on the servo-hydraulic test machine

	Test	Loading rate (mm/s)	f'_c (MPa)	$E_f t_f$ (GPa·mm)	t_f (mm)	b_f (mm)	L (mm)	Number
C40-1	M1-C40-1	0.02	32.0	41.4	0.5	33	150	A, B, C (3)
	M2-C40-1	0.8	32.0	41.4	0.5	33	150	A, B, C (3)
	M3-C40-1	8.0	32.0	41.4	0.5	33	150	A, B, C (3)
	M4-C40-1	80	32.0	41.4	0.5	33	150	A, B, C (3)
C40-2	M1-C40-2	0.02	32.0	82.7	1.0	33	150	A, B, C (3)
	M4-C40-2	80	32.0	82.7	1.0	33	150	A, B, C (3)
C40-3	M1-C40-3	0.02	32.0	41.4	0.5	26	150	A, B, C (3)
	M4-C40-3	80	32.0	41.4	0.5	26	150	A, B, C (3)
C40-4	M1-C40-4	0.02	32.0	41.4	0.5	33	100	A, B, C (3)
	M4-C40-4	80	32.0	41.4	0.5	33	100	A, B, C (3)
C40-5	M1-C40-5	0.02	32.0	41.4	0.5	33	75	A, B, C (3)
	M4-C40-5	80	32.0	41.4	0.5	33	75	A, B, C (3)
C60-1	M1-C60-1	0.02	53.8	41.4	0.5	33	150	A, B, C (3)
	M3-C60-1	8.0	53.8	41.4	0.5	33	150	A, B, C (3)
	M4-C60-1	80	53.8	41.4	0.5	33	150	A, B, C (3)

3.2.3 High loading-rate tests

As the loading rate of the servo-hydraulic test machine was limited by the capacity of its hydraulic oil pump, the set-up could not be used for high loading-rate tests. A modified SHPB set-up could be used for this purpose. A normal SHPB set-up consists of a striker bar, an incident bar and a transmission bar [62], all of which are made of high-strength steel. The specimen is placed between the incident bar and the transmission bar and is loaded by a compressive impulse wave from the incident bar.

Since it was not possible to test CFRP-concrete specimens using the normal SHPB machine, the set-up had to be modified to conduct high loading-rate shear tests. As depicted in Figures 3.3(a) and 3.3(b), the transmission bar was removed and replaced by a grip, which was constrained by a roller guide to have horizontal movement only. The upper part of the grip was in contact with the incident bar to transfer the impact load so that the whole grip would move along the roller guide. Below the incident bar (Figure 3.3(a)), the CFRP sheet was clamped in the grip and was pulled in the horizontal direction upon impact. Both the grip and the roller guide were fabricated using DF3 steel, a tool steel with a high yield strength of 1800 MPa. During the impact test, the concrete block was restrained from movement by a fixture on each side.

(a) Test set-up

(b) Grip and roller guide

(c) Photograph of the modified SHPB test

Figure 3.3 Bond-slip test set-up using modified SHPB

The measurement system is shown in Figures 3.3(a) and 3.3(c). Six equally-spaced strain gauges were placed on the bonded region to measure CFRP strain distribution along the loading direction. Since no load cell was available for this test set-up, one additional strain gauge was placed on the CFRP sheet ahead of the bonding area to measure the impact force applied onto the specimen. A high-frequency data logger was used to collect strain measurements at a frequency of 1 MHz.

Table 3.3 summarises the tests conducted with the modified SHPB set-up. The loading rate of the tests was controlled by air pressure in the gas gun. Since there was no direct measurement for loading rate in this set-up, the S1 and S2 designation

represent an applied air pressure of 1.5 bar and 3.5 bar in the gas gun, respectively. The meanings of the notations in Table 3.3 are the same as those for the quasi-static and low loading-rate tests as described for Table 3.2.

Prior to the impact tests, two specimens were tested under the air pressure of 4.0 bar and 1.0 bar to find an appropriate range for the impact load. The test under 4.0 bar caused visible bending of one roller indicating that the applied load had exceeded the capacity of the roller guide. On the other hand, the loading produced with a pressure of 1.0 bar was too low to entirely separate CFRP sheet from the concrete surface, indicating an unsuccessful test. Therefore, to avoid both situations, the specimens were tested under an air pressure of 1.5 bar or 3.5 bar. Similar to the tests conducted on the servo-hydraulic machine, three samples (i.e. A, B, C) were prepared for each configuration.

Table 3.3 Test items on the modified SHPB

Test		Air pressure (bar)	f'_c (MPa)	$E_f t_f$ (GPa·mm)	t_f (mm)	b_f (mm)	L (mm)	Number
C40-1	S1-C40-1	1.5	32.0	41.4	0.5	33	150	A, B, C (3)
	S2-C40-1	3.5	32.0	41.4	0.5	33	150	A, B, C (3)
C40-2	S1-C40-2	1.5	32.0	82.7	1.0	33	150	A, B, C (3)
	S2-C40-2	3.5	32.0	82.7	1.0	33	150	A, B, C (3)
C40-3	S1-C40-3	1.5	32.0	41.4	0.5	26	150	A, B, C (3)
C40-4	S1-C40-4	1.5	32.0	41.4	0.5	33	100	A, B, C (3)
	S2-C40-4	3.5	32.0	41.4	0.5	33	100	A, B, C (3)
C40-5	S1-C40-5	1.5	32.0	41.4	0.5	33	75	A, B, C (3)
C60-1	S1-C60-1	1.5	53.8	41.4	0.5	33	150	A, B, C (3)
	S2-C60-1	3.5	53.8	41.4	0.5	33	150	A, B, C (3)

3.3 Test Results

3.3.1 Quasi-static behaviour

Quasi-static test results were assessed by comparing with model predictions with other researchers. Based on finite element simulations and regression analysis of existing quasi-static tests, Lu et al. [3] derived a bond-slip model to predict peak bond

stress σ_{IIst} and interface fracture energy G_{IIst} of FRP-concrete bond interface, as follows.

$$\sigma_{IIst} = \alpha_1 \beta_w f_t \quad (3.1)$$

$$G_{IIst} = 0.308 \beta_w^2 \sqrt{f_t} \quad (3.2)$$

where α_1 is a coefficient determined from regression and is given by 1.5, β_w is the bond width coefficient and f_t is the concrete tensile strength.

With interface fracture energy, based on literature, the ultimate load P_u of single-lap shear test can be predicted by [3, 22, 37, 63]:

$$P_u = \beta_L b_f \sqrt{2 E_f t_f G_{IIst}} \quad (3.3)$$

where β_L is the bond length coefficient, b_f is the bond width of CFRP sheet and $E_f t_f$ is the CFRP stiffness.

The bond width coefficient β_w is to account for different widths of the FRP (b_f) and the concrete substrate (b_c) [3, 4].

$$\beta_w = \sqrt{\frac{2 - b_f/b_c}{1 + b_f/b_c}} \quad (3.4)$$

The bond length coefficient is a piecewise function related to effective bond length L_e , as shown in Eq. (3.5) [3, 4]. When the bond length L is smaller than L_e , an increase of L would change β_L . However, any increase in bond length over L_e would not improve ultimate load.

$$\beta_L = \begin{cases} 1 & \text{if } L \geq L_e \\ \sin \frac{\pi L}{2 L_e} & \text{if } L < L_e \end{cases} \quad (3.5)$$

where effective bond length L_e is defined as $L_e = \sqrt{\frac{E_f t_f}{f'_c}}$.

To assess the accuracy of model predictions, the author's results of the quasi-static

tests were compared with the model predictions by Lu et al. [3]. The results are shown in Table 3.4. The coefficients of variation (CoV) of ultimate load P_u and peak bond stress τ_{st} were in the range of 1.0% to 16.2%. This showed that the repeated tests produced consistent results. The ratio between the test results and the predicted values were reasonable, varying between 0.81 and 1.05, indicating that the model by Lu et al. [3] could properly predict these quasi-static tests. The overestimation of peak bond stress in M1-C60-1 specimens showed that Lu's model might overstate this property for samples with high strength concrete. As Lu's model [3] has been widely used in analyses of FRP strengthened concrete structures, the predictions of peak bond stress and interface fracture energy were used as the baseline for dynamic increase factor (DIF) values of dynamic test results.

Table 3.4 Quasi-static test results and comparison with predictions from Lu's model [3]

Item	P_u in tests (kN)	Average (kN)/ CoV	P_u by model (MPa) [3]	Ratio of test/ model	σ_{lst} in tests (MPa)	Average (MPa)/ CoV	σ_{lst} by model (MPa) [3]	Ratio of test/ model
M1-C40-1A	8.22				5.05			
M1-C40-1B	8.51	8.16/	7.77	1.05	5.15	5.10/	5.06	1.01
M1-C40-1C	7.74	4.8%			5.09	1.0%		
M1-C40-2A	9.96				5.02			
M1-C40-2B	9.07	9.86/	10.99	0.90	4.14	4.97/	5.06	0.98
M1-C40-2C	10.56	7.6%			5.75	16.2%		
M1-C40-3A	6.02				4.88			
M1-C40-3B	7.73	6.68/	6.48	1.03	5.37	5.27	5.35	0.99
M1-C40-3C	6.29	13.8%			5.55	6.6%		
M1-C40-4A	8.73				4.68			
M1-C40-4B	8.81	8.19/	7.77	1.05	5.62	5.10/	5.06	1.01
M1-C40-4C	7.03	12.3%			5.00	9.4%		
M1-C40-5A	7.08				5.00			
M1-C40-5B	7.70	7.18/	7.61	0.94	4.85	5.26/	5.06	1.04
M1-C40-5C	6.77	6.5%			5.93	11.1%		
M1-C60-1A	8.35				6.28			
M1-C60-1B	9.32	9.02/	8.51	1.04	5.81	5.82/	7.15	0.81
M1-C60-1C	9.37	6.4%			5.38	7.7%		

3.3.2 Test results and failure modes

Figure 3.4 presents typical layers in CFRP strengthened RC structures and locations of possible debonding failure as reported in literature [2]. For classification purpose, failure modes of the specimens in this study were recorded as debonding in concrete (DB-C), debonding in concrete and epoxy (DB-CE), debonding and CFRP failure at the edge (DB-CFE), CFRP failure at middle (CFM) and CFRP not debonding but failure at the edge (NDB-CF), as shown in Figure 3.5. The debonded CFRP sheet was placed next to the concrete substrate to show the failure mode and its extent. The failure mode of each test is indicated in Table 3.5.

Figure 3.4 Bond failure at different layers

(c) Debonding and CFRP failure at edge (DB-CFE) (d) CFRP failure at middle (CFM)

(e) Not debonding and CFRP failure at edge (NDB-CF)

Figure 3.5 Different failure modes of testing specimens

In these tests, sixty out of the seventy-five specimens failed by debonding in the concrete layer (DB-C), which left some mortar and sands attached to the CFRP sheet, as shown in Figure 3.5(a). The thickness of this separated concrete layer below the bond interface varied from about 0.5 mm to 4 mm. This result was commonly observed in the published literature on CFRP-concrete debonding tests [1, 3]. Since tensile strength and shear strength of concrete are always weaker than epoxy strength of 50 MPa, failure process started from cracking of concrete at the loaded end of the specimen (Figure 3.1(a)). With an increase of applied load, more cracks appeared in the concrete layer below the bond interface and failure propagated to the free end of the bonded area.

Ten specimens under dynamic load showed a second failure mode in which cracks formed in both the concrete and the epoxy (DB-CE), as shown in Figure 3.5(b).

Similar to the first failure mode, a lump of concrete spalled off from the loaded end, which meant that cracks also started from the concrete layer. After separation from the concrete substrate, less concrete was attached to the CFRP sheet and some epoxy lumps were left on the concrete block. This failure mode could be attributed to dynamic load causing greater stress concentration at different locations between the two materials. Specifically, after concrete spalling at the loaded end in Figure 3.5(b), the U-shaped corner experienced stress concentration in the epoxy layer and resulted in debonding of the epoxy layer around the corner.

In the dynamic tests using the modified SHPB, four specimens failed due to CFRP failure along the fibre direction, as shown in Figures 3.5(c) and 3.5(d). Majority of the CFRP sheet of specimen S1-C40-3A debonded from the concrete. However, it split longitudinally and a 4 mm wide strip at the left edge remained affixed on the concrete surface, as shown in Figure 3.5(c). Specimen S2-C40-4C showed more severe longitudinal damage, with a 10 mm wide of CFRP remaining on the concrete in Figure 3.5(d). Specimens S2-C60-1B and S2-C60-1C also failed in the same way. To investigate the reason for the CFRP fracture, tensile strength of the composite sheet was obtained as 750 MPa. The sectional dimensions of the sheets used for S1-C40-3 and S2-C40-4 were 26 mm $\times$ 0.5 mm and 33 mm $\times$ 0.5 mm, respectively. Multiplying these areas by its tensile strength, the estimated ultimate loads were 9.75 kN and 12.38 kN, respectively. The measured ultimate loads for the three tests of S1-C40-3A, B and C were between 9.30 kN and 10.04 kN, close to the estimated value of 9.75 kN. In the same way, the ultimate loads measured in the tests of S2-C40-4 specimens were between 12.10 kN and 14.42 kN, compared to the ultimate load of 12.38 kN. Therefore, this simple calculation shows that CFRP fracture in these specimens was caused by ultimate loads exceeding the respective CFRP fracture capacity. Since strain gauges were attached on the CFRP surface in these shear tests, the CFRP fracture behaviour made tests of S2-C40-4C, S2-C60-1B and S2-C60-1C

have no valid data for the interface debonding study.

As shown in Figure 3.5(e), the specimen S1-C40-2B failed with only a narrow part of the CFRP debonding, while a large portion of the sheet remained intact. In this case, unlike the other CFRP fracture scenarios, the load was significantly lower than the composite capacity. Therefore, this result could have been due to poor workmanship of the CFRP sheet itself. Though this particular sample failed to provide useful data, the repeated tests S1-C40-2A and S1-C40-2C gave reliable results and were used in the statistical analysis.

All the results of seventy-five specimens are presented in Table 3.5. Six types of specimens with different material and geometric properties were tested for parametric study. Specimens C40-1 were a control group with a single-layer CFRP sheet. A double-layer CFRP sheet and a narrower CFRP sheet were used in specimens C40-2 and C40-3 to investigate the effect of CFRP stiffness and width, respectively. Specimens C40-4 and C40-5 were used to study the effect of bond length under dynamic loading. To study the effect of different concrete strengths, higher concrete strength was adopted for C60-1 specimens. The DIF was calculated as the ratio of each dynamic test result divided by the corresponding quasi-static predictions by Lu's model [3].

Table 3.5 Summary of all test results

Specimens		Slip rate	d_u	P_u	σ_{IIa}	DIF	Stiffness	G_{IIa}	DIF	Failure
		(mm/s)	(mm)	(kN)	(MPa)	(σ_{IIa})	(MPa/mm)	(N/mm)	(G_{IIa})	mode
C40-1	M1-C40-1A	3.90e-3	0.78	8.22	5.05	1.00	65.5	0.86	1.31	DB-C
	M1-C40-1B	4.50e-3	0.74	8.51	5.15	1.02	84.9	0.76	1.13	DB-C
	M1-C40-1C	4.70e-3	0.70	7.74	5.09	1.01	90.8	0.63	0.94	DB-C
	M2-C40-1A	2.37e-1	0.83	9.84	5.83	1.15	92.3	1.04	1.55	DB-C
	M2-C40-1B	2.28e-1	0.75	8.17	4.90	0.97	89.9	0.81	1.21	DB-C
	M2-C40-1C	2.26e-1	0.71	8.07	5.88	1.16	96.1	0.81	1.21	DB-C
	M3-C40-1A	2.46	0.83	9.38	5.57	1.10	85.5	1.15	1.72	DB-CE
	M3-C40-1B	2.16	0.71	8.92	5.75	1.14	86.0	0.88	1.31	DB-C
	M3-C40-1C	2.38	0.79	8.70	6.09	1.20	97.3	0.90	1.34	DB-C
	M4-C40-1A	31.83	0.88	10.27	5.93	1.17	84.5	1.21	1.81	DB-C
	M4-C40-1B	25.12	0.85	9.34	6.04	1.19	88.6	0.82	1.22	DB-C
	M4-C40-1C	26.85	0.84	10.09	6.46	1.28	92.0	1.12	1.67	DB-C
	S1-C40-1A	1541	0.94	12.45	6.88	1.36	58.3	1.47	2.19	DB-C
	S1-C40-1B	1786	0.88	11.74	9.85	1.94	98.0	1.79	2.67	DB-CE
	S1-C40-1C	1127	0.83	11.39	7.51	1.48	84.7	1.14	1.70	DB-C
	S2-C40-1A	1509	1.07	14.71	8.71	1.72	96.3	1.98	2.96	DB-CE
	S2-C40-1B	2150	1.16	11.79	8.77	1.73	95.8	1.81	2.70	DB-CE
	S2-C40-1C	1927	1.22	14.12	9.81	1.94	89.4	2.05	3.06	DB-CE
C40-2	M1-C40-2A	4.60e-3	0.48	9.96	5.02	0.99	114.6	0.64	0.96	DB-C
	M1-C40-2B	4.10e-3	0.44	9.07	4.14	0.82	116.2	0.47	0.70	DB-C
	M1-C40-2C	3.50e-3	0.41	10.56	5.75	1.14	125.0	0.81	1.21	DB-C
	M4-C40-2A	22.08	0.54	14.44	6.83	1.35	129.0	0.96	1.43	DB-C
	M4-C40-2B	25.06	0.61	14.00	7.30	1.44	123.0	1.12	1.67	DB-C
	M4-C40-2C	24.49	0.72	14.11	8.88	1.75	138.3	1.60	2.39	DB-C
	S1-C40-2A	807	0.76	19.46	9.76	1.93	169.1	1.98	2.96	DB-C
	S1-C40-2B	—	—	—	—	—	—	—	—	NDB-CF
	S1-C40-2C	849	0.76	21.59	10.07	1.99	153.9	1.90	2.84	DB-C
	S2-C40-2A	1371	0.74	18.29	9.25	1.83	172.1	1.90	2.84	DB-C
	S2-C40-2B	1176	0.79	19.32	11.99	2.37	172.2	1.86	2.78	DB-C
	S2-C40-2C	1676	0.88	21.39	9.87	1.95	166.9	2.21	3.30	DB-C
C40-3	M1-C40-3A	4.10e-3	0.61	6.02	4.88	0.91	96.3	0.59	0.79	DB-C
	M1-C40-3B	4.10e-3	0.87	7.73	5.37	1.00	86.8	0.78	1.04	DB-C
	M1-C40-3C	4.70e-3	0.74	6.29	5.55	1.04	94.0	0.65	0.87	DB-C
	M4-C40-3A	43.30	1.10	9.47	7.21	1.35	91.1	1.55	2.07	DB-C
	M4-C40-3B	41.47	0.91	8.68	6.89	1.29	87.8	1.08	1.44	DB-C
	M4-C40-3C	37.09	0.96	9.94	6.97	1.30	88.5	1.28	1.71	DB-C
	S1-C40-3A	2077	1.05	10.04	8.16	1.52	102.7	1.63	2.17	DB-CFE

(Continued on next page)

Table 5. (continued)

Specimens		Slip rate	d_u	P_u	$\sigma_{II d}$	DIF	Stiffness	$G_{II d}$	DIF	Failure
		(mm/s)	(mm)	(kN)	(MPa)	($\sigma_{II d}$)	(MPa/mm)	(N/mm)	($G_{II d}$)	mode
C40-3	S1-C40-3B	2017	1.00	9.30	7.84	1.46	97.7	1.12	1.49	DB-CE
	S1-C40-3C	1873	1.10	9.46	7.95	1.48	97.1	1.49	1.99	DB-C
C40-4	M1-C40-4A	4.60e-3	0.39	8.73	4.68	0.92	122.5	0.50	0.75	DB-C
	M1-C40-4B	4.20e-3	0.53	8.81	5.62	1.11	119.5	0.68	1.01	DB-C
	M1-C40-4C	4.30e-3	0.37	7.03	5.00	0.99	107.2	0.50	0.75	DB-C
	M4-C40-4A	24.31	0.63	9.89	8.30	1.64	143.7	1.25	1.87	DB-CE
	M4-C40-4B	21.66	0.50	8.76	6.69	1.32	125.0	0.90	1.34	DB-C
	M4-C40-4C	23.82	0.53	9.10	7.31	1.44	140.7	0.94	1.40	DB-C
	S1-C40-4A	845	0.71	11.50	9.83	1.94	144.0	1.56	2.32	DB-C
	S1-C40-4B	841	0.74	13.01	7.83	1.55	134.2	1.59	2.37	DB-CE
	S1-C40-4C	743	0.70	12.38	9.02	1.78	145.3	1.47	2.19	DB-C
	S2-C40-4A	1521	0.90	14.42	8.91	1.76	96.4	1.73	2.58	DB-C
	S2-C40-4B	1858	0.80	12.10	10.07	1.99	137.1	1.61	2.40	DB-C
	S2-C40-4C	—	—	13.60	—	—	—	—	—	CFM
C40-5	M1-C40-5A	4.00e-3	0.35	7.08	5.00	0.99	105.4	0.53	0.79	DB-C
	M1-C40-5B	3.70e-3	0.31	7.70	4.85	0.96	112.1	0.52	0.78	DB-C
	M1-C40-5C	3.60e-3	0.30	6.77	5.93	1.17	138.8	0.53	0.79	DB-C
	M4-C40-5A	15.33	0.39	9.46	6.94	1.37	116.2	0.90	1.34	DB-C
	M4-C40-5B	16.24	0.36	8.40	6.77	1.34	148.9	0.77	1.15	DB-C
	M4-C40-5C	16.85	0.39	7.41	8.23	1.63	154.5	1.13	1.69	DB-C
	S1-C40-5A	595	0.52	11.50	7.99	1.58	103.3	1.30	1.94	DB-CE
	S1-C40-5B	655	0.54	12.73	8.35	1.65	150.3	1.36	2.03	DB-C
	S1-C40-5C	818	0.52	11.00	9.93	1.96	159.8	1.27	1.90	DB-C
C60-1	M1-C60-1A	6.10e-3	0.88	8.35	6.28	0.88	96.2	0.93	1.17	DB-C
	M1-C60-1B	5.10e-3	0.83	9.32	5.81	0.81	91.5	1.00	1.26	DB-C
	M1-C60-1C	4.60e-3	0.76	9.37	5.38	0.75	96.3	0.89	1.12	DB-C
	M3-C60-1A	2.60	0.91	10.58	6.27	0.88	83.2	1.26	1.58	DB-C
	M3-C60-1B	2.44	0.77	9.84	5.93	0.83	91.9	0.93	1.17	DB-C
	M3-C60-1C	2.36	0.79	8.82	6.26	0.88	96.1	0.92	1.15	DB-C
	M4-C60-1A	31.43	0.90	9.83	8.02	1.12	96.9	1.32	1.66	DB-C
	M4-C60-1B	29.17	0.91	9.79	7.34	1.03	98.9	1.11	1.39	DB-C
	M4-C60-1C	29.32	0.89	10.25	7.61	1.06	93.9	1.23	1.54	DB-C
	S1-C60-1A	1206	1.16	12.92	8.56	1.20	101.7	1.75	2.20	DB-C
	S1-C60-1B	1211	1.10	12.87	8.23	1.15	98.5	1.53	1.92	DB-CE
	S1-C60-1C	1262	1.15	12.38	9.22	1.29	99.7	1.76	2.21	DB-C
	S2-C60-1A	2031	1.11	13.85	8.21	1.15	99.2	1.64	2.06	DB-C
	S2-C60-1B	—	—	14.23	—	—	—	—	—	CFM
	S2-C60-1C	—	—	13.19	—	—	—	—	—	CFM

Note: DB-C, debonding in concrete; DB-CE, debonding in concrete and epoxy; NDB-CF, not debonding and CFRP failure at edge; DB-CFE, debonding and CFRP failure at edge; CFM, CFRP failure at middle.

3.3.3 Rate effect on load-displacement behaviour

The general failure process of the specimens could be studied using the load-displacement curves. The load values were recorded by the built-in load cell in the test machine and the high-frequency data logger. However, the displacements recorded by the test machine always included relative slip between the grip and the CFRP sheet and deformations of the supporting fixture [1]. To obtain more accurate load-displacement curves, relative slip measurements at the loaded end of the bonded area were used to represent the displacement values. Dai et al. [5] proposed Eq. (3.6) to calculate the relative slip. Concrete was assumed to be rigid and has no movement. The slip at an arbitrary position i along the CFRP bonding length was calculated by integrating the CFRP strains starting from the free end, where strain was assumed to be 0, up to location i .

$$s_i = \frac{\Delta x}{2} (\varepsilon_{f0} + 2 \sum_{j=1}^{i-1} \varepsilon_{fj} + \varepsilon_{fi}) \quad (3.6)$$

where s_i is the slip at position i , Δx is the distance between the strain gauges, ε_{f0} is the strain measured by the strain gauge near the free end (position 0), ε_{fj} is the strain measured between positions 0 and j , and ε_{fi} is the strain value at position i . Thus, the slip at the loaded end can be calculated by integrating the strain along the whole bonded length of CFRP. This method was also adopted in the study of Yuan et al. [37].

Along the CFRP bond length of 150 mm, the loaded end was defined at the position of 150 mm and the free end was defined to be at 0 mm. Variables t_1 to t_5 in Figure 3.6(a) denote the time required for each bonding point to undergo debonding process. In accordance with Eq. (3.6), strain-time curves of 7 mm and 35 mm positions in

Figure 3.6(a) were numerically integrated to produce the slip-time curve at 35 mm position in Figure 3.6(b). Similarly, the slip-time curve at 63mm positions was derived from integration of strain-time curves at 7 mm, 35 mm and 63 mm positions, and so on. Parameters s_1 to s_5 in Figure 3.6(b) are the respective slip values of each point at the instant of debonding. It is observed that the gradients of the slip-time curves in Figure 3.6(b) were not stable. As the slip values were the greatest at the loaded end and the smallest near the free end, it was useful to obtain an average slip rate value to represent the dynamic properties of each test. The author decided to use slip-time average equation to obtain an average slip rate from these slip-time curves for each specimen. In this study, the equation to generate the constant slip rate is given by:

$$\dot{s} = \frac{\sum_{i=1}^z \frac{s_i}{t_i}}{z} \quad (3.7)$$

where z is the total number of slip-time curves for a tested specimen. Figure 3.6(b) shows an example of this calculation including the slip-time history of each strain gauge position and the constant slip rate. The calculated average slip rates are given in the second column of Table 3.5 and are used in subsequent data analyses.

(b)

Figure 3.6 (a) Strain-time history and (b) slip-time history at each position and average slip rate (M4-C40-1A, average slip rate of 31.83 mm/s)

Figure 3.7 presents the load-displacement curves of specimens in the C40-1 group. To highlight the slip-rate effects, curves obtained from the quasi-static tests to the impact tests were included. The slip rates of these tests were between $3.90\text{e-}3$ mm/s and 1927 mm/s.

Figure 3.7 Load-displacement curves of C40-1 specimens for different slip rates

In the quasi-static test (M1-C40-1A), the load-displacement curve was close to linear at initial displacements. After the displacement reached 0.1 mm, the gradient gradually decreased to zero due to softening caused by local cracks along the bond length. The specimen had reached its ultimate strength at this stage. This value was maintained and bond failure propagated towards the free end until the CFRP sheet was totally separated from the concrete substrate. Specimens tested under higher slip rates generally showed a similar behaviour but attained higher ultimate loads and larger ultimate displacements. These phenomena indicated that both the load-carrying capacity and the ultimate displacement of the bond are sensitive to slip rate effect.

3.3.4 Rate effect on bond stress-slip behaviour

Following the analysis of the load-displacement curves, the differences in bond strength between static and dynamic tests were evaluated. Existing studies proposed a method to calculate shear bond stress τ for these tests [22], as shown in Figure 3.8. The relevant equations are reproduced in Eq. (3.8) and Eq. (3.9) by Lu et al. and Dai et al. [3, 5].

Figure 3.8 Static equilibrium sketch for bond stress τ

From Figure 3.8, the stress equilibrium equation can be written as:

$$\sigma_f t_f + \tau dx = \left(\sigma_f + \frac{d\sigma_f}{dx} dx \right) t_f \quad (3.8)$$

This equation can be simplified to $\tau = \frac{d\sigma_f}{dx} t_f$. Substituting $\sigma_f = E_f \varepsilon_f$ into Eq. (3.8),

shear bond stress τ can be calculated as:

$$\tau = \frac{E_f d\varepsilon_f}{dx} t_f \quad (3.9)$$

where σ_f , t_f and E_f are respectively the axial stress, the thickness and Young's modulus along the loading direction of the CFRP sheet. To generate shear bond stress τ at an arbitrary position i , the difference in axial strain $d\varepsilon_f$ is obtained using the difference in values of two strain gauges adjacent to position i . The length dx denotes the physical distance between two strain gauges. Since the first strain gauge at the free end was the starting point and had zero strain gradient ($d\varepsilon_f$), there were five computing points for the six strain gauges on the CFRP (Figure 3.1(a)). Following this method, as an example, the five bond stress-slip curves of specimen S1-C40-1C are calculated and shown in Figure 3.9.

As the aggregates and mortar were not uniformly distributed in concrete along the bond length, the bond-slip behaviour of S1-C40-1C at the five points shown in Figure 3.9 displayed some differences. The local interfacial bond behaviour is governed by the average interfacial bond strength and fracture energy [5]. To obtain a characteristic curve representing an average response along the bonding length, the curves at all five points were used to generate an average bond-slip curve for each specimen. The average line is shown as the solid line in Figure 3.9.

Figure 3.9 Bond stress-slip curves at each position of S1-C40-1C

Figure 3.10 shows the slip-rate effect on bond stress-slip curves of the same six specimens in Figure 3.7, ranging from quasi-static to impact loading regimes. The general shape of these curves was similar, with a linear ascending range and a gradual softening range. The results showed the dynamic enhancing effect, with higher peak bond stresses and larger areas under the graph for specimens tested at higher slip rates. However, except for the one tested at the lowest rate (M1-C40-1A), the initial slopes of all the curves were similar. This indicated that the initial stiffness values of these bond-slip curves were insensitive to slip rate.

Figure 3.10 Bond stress-slip curves of C40-1 specimens under different slip rates

The initial stiffness value K_{II} of the bond is determined from the equation below [64]:

$$K_{II} = \frac{\Delta\tau}{\Delta s} \quad (3.10)$$

where Δs is the selected slip range in the initial part of the curve in mm and $\Delta\tau$ is the corresponding increase of bond stress in MPa. In this study, Δs was chosen to be between 0.01 mm and 0.04 mm. To shed light on the slip-rate effect on initial stiffness with material and geometric parameters, Figure 3.11 shows the results for specimens in groups C40-1, C40-2, C40-3 and C60-1. Specimens C40-1 were a control group with CFRP stiffness of 41.4 GPa·mm and bond width of 33 mm.

Specimens C40-2 had a higher CFRP stiffness at 82.7 GPa·mm and specimens C40-3 reduced CFRP bond width at 26 mm. Compared with C40-1 specimens, C60-1 specimens adopted higher concrete strength. As the bond lengths of C40-4 and C40-5 specimens were too short to generate complete bond-slip curves, these two groups were excluded from the study of local interface property.

Figure 3.11 Dynamic response of the initial stiffnesses of different specimens (ave* means an average of 3 tests)

Among the specimens in groups C40-1, C40-3 and C60-1, there was no clear dynamic enhancing effect, and most of them had similar initial modulus magnitude in the range of 83.2-102.7 MPa/mm. However, the C40-2 specimens, which had a higher CFRP stiffness, showed a higher average quasi-static initial stiffness of 118.8 MPa/mm. The value increased significantly beyond a slip rate of 23.87 mm/s and reached 170.4 MPa/mm at the highest rate. The significant rate-dependent response of C40-2 specimens might be due to that CFRP sheets in these specimens have the highest stiffness and the dynamic response of the bond is more dependent on the concrete material. Concrete is much more sensitive to the rate effect than CFRP [46, 65]. Therefore, the data showed that higher composite stiffness ($E_f t_f$) could have rendered the initial stiffness higher and more sensitive to slip rate.

3.3.5 Rate effect on fracture energy

Apart from load-displacement curves and bond stress-slip curves, fracture energy is an essential property of the interfacial bond. It is defined as the area under bond stress-slip curves [3]. Figure 3.12 shows the fracture energy of specimens C40-1 to assess the dynamic effect on this property. The results show a nonlinear dynamic enhancement with the increase of slip rate. The initial range, between a slip rate of 0.004 mm/s and 50 mm/s, is approximately linear. After this, the gradient increases and the fracture energy increases significantly at a slip rate of about 2000 mm/s, indicating that the interface bond is more sensitive to high slip rates. As the dynamic behaviour of the interface bond largely depends on concrete dynamic response [22], this phenomenon might be attributed to the fact that, at impact regime, concrete material properties are significantly sensitive to rate effect [48, 66].

Figure 3.12 Rate effect on fracture energy of C40-1 specimens

3.4 Parametric effect

3.4.1 Effect of CFRP stiffness

To study the effect of CFRP stiffness on dynamic debonding properties, specimens of group C40-2 were fabricated with double-layer CFRP sheets, which doubled the stiffness ($E_f t_f$) of the composite. Figure 3.13 shows a comparison of the global

performance and local bond properties of specimens with different CFRP stiffnesses. The global performance of these shear tests includes ultimate load and ultimate displacement. The local bond properties are the key parameters which dominate local debonding behaviour, such as peak bond stress and fracture energy.

Figure 3.13 Rate effect comparison between specimens of C40-1 and C40-2 for (a) ultimate load, (b) ultimate displacement, (c) peak bond stress and (d) fracture energy

Compared to C40-1 specimens, specimens of C40-2 group were generally more sensitive to slip rate, in particular, the ultimate loads of C40-2 group in Figure 3.13(a) increased with a larger gradient and showed a more significant dynamic enhancing effect. Two reasons could contribute to this phenomenon. The first reason is similar to that explained for the initial stiffness shown in Figure 3.11. The stiffer composite sheet could increase the initial stiffness of the bond and reduce the relative slip

between the two materials. Concrete dynamic response is more sensitive to the rate effect than CFRP response [48, 65]. With a stiffer bond and less relative slip, the debonding process in C40-2 specimen is more closely linked to concrete response. Secondly, specimens in C40-2 group used a double-layer CFRP sheet which doubled the CFRP load capacity. Accordingly, specimens of C40-2 group could sustain significantly higher loading at dynamic regime. The increase of stiffness had less effect on improving dynamic peak bond stress, as shown in Figure 3.13(c). Both the average curves of C40-1 and C40-2 started from about 5 MPa under quasi-static load and respectively reached 9.10 MPa and 10.37 MPa at the highest rates. This phenomenon could be explained through Eq. (3.9). The bond stress is determined by the product of CFRP stiffness $E_f t_f$ and strain gradient $d\varepsilon_f$. As the strain gradients of C40-2 specimens were smaller under higher stiffness, the final bond stress did not increase significantly.

Unlike the higher values of ultimate load and peak bond stress in C40-2 group, the ultimate displacements presented in Figure 3.13(b) are lower than those of the control group. Such performance indicated that, for the specimens with a greater CFRP stiffness, concrete had more effect on the bond and the relative slip was reduced.

Among the four charts of Figure 3.13, the mean ultimate displacement, peak bond stress and fracture energy of samples C40-2 increased moderately at a slip rate above 1000 mm/s. The average ultimate load even decreased slightly in this range. Such a phenomenon could be due to scattering of test results at high slip rates.

3.4.2 Effect of CFRP-concrete bond width ratio

Specimens of C40-3 used CFRP sheets with a width of 26 mm, narrower than the 33 mm sheets used for other samples. Figure 3.14 presents a comparison of slip-rate effect on these narrower samples and on control group C40-1.

Figure 3.14 Rate effect comparison between specimens of C40-1 and C40-3 on (a) ultimate load, (b) ultimate displacement, (c) peak bond stress and (d) fracture energy

The dynamic increase in mechanical properties of samples C40-3 is seen in Figure 3.14. Due to the narrower bond width of CFRP, the ultimate loads were lower than those of C40-1. However, the mean ultimate displacement values were greater than those of the control group, except for the final set at the highest slip rate. Since the CFRP stiffness of the two sample types was the same, the larger deformations in specimens with the narrower CFRP sheets indicated higher axial stresses than those in the control group at equal slip rates. Beyond a slip rate of 40 mm/s, unlike the remarkable increases of C40-1, the ultimate load and the ultimate displacement of samples C40-3 increased moderately. This difference could be caused by the dynamic loads in the high slip rate region approaching the CFRP maximum load of 9.75 kN,

as explained in Section 3.2.

In Figures 3.14(c) and 3.14(d), under quasi-static load, the peak bond stress and fracture energy of the two groups were similar. At a slip rate of 40 mm/s, the bond properties of C40-3 specimens were slightly higher than those of control group. Observing the failure behaviour of the specimens after the tests, it was found that composite debonding failure was not only located in the concrete below the bonding area, but also extended to nearby concrete. Therefore, the narrow sheets could reach a higher mobilised concrete-to-CFRP width ratios, which meant more concrete contributing to the debonding capacity for a unit width of CFRP sheet. This finding could explain the slightly higher peak bond stresses and fracture energies in group C40-3. At a slip rate of around 2000 mm/s, both values of peak bond stress and fracture energy of group C40-3 were lower than those of C40-1. As mentioned in Section 3.2, the limitation of CFRP maximum load in C40-3 group restricted further development of dynamic enhancing effect on this group. The specimen S1-C40-3A even failed by CFRP fracture.

3.4.3 Effect of bond length

The effect of bond length was studied with specimens C40-4 and C40-5, which used CFRP sheets with lengths of 100 mm and 75 mm, respectively. Figure 3.15 presents a comparison of the slip-rate effect on ultimate load and ultimate displacement between these specimens and the control group of C40-1. Due to a shorter bond length of 100 mm or 75 mm compared to 150 mm, it was not able to obtain complete bond stress-slip curves for analysis of local properties from the results of C40-4 and C40-5. Therefore, the effect of bond length on the peak bond stress and fracture energy is not presented in this section.

Figure 3.15 Rate effect comparison between specimens of C40-1, C40-4 and C40-5 on (a) ultimate load and (b) ultimate displacement

In Figure 3.15(a), the average values of the ultimate load of samples C40-4 were close to those of group C40-1 under all slip rates. Instead, due to their shorter bond length, the average values of C40-5 specimens were lower under low slip rates. As measured in the quasi-static tests, the effective bond lengths of samples C40-1 and C40-4 were around 90 mm. The actual CFRP bond lengths of these samples were longer than the effective bond length and this guaranteed that their ultimate loads would be fully developed. However, the shorter length of C40-5 specimens restricted the full development of ultimate load. According to the results, the effect of bond length under low slip rate was similar to the quasi-static scenario. However, at a high slip rate of 689 mm/s, the ultimate load for the C40-5 group significantly increased and attained the values of the other two groups. This increase in the ultimate load could be attributed to an increase of concrete strength under a high slip rate. Based on Eq. (3.5), the effective bond length is inversely proportional to concrete strength. Due to the dynamic increase of concrete strength at a high slip rate, the effective bond length should be shorter than that for a lower slip rate. The equal load-carrying capacity of samples of C40-5 and C40-1 at high slip rates verified this trend.

In Figure 3.15(b), the general behaviour of ultimate displacement among all three sets

was an increase with growing slip rate. The ultimate displacement values consistently reduced with shorter bond length.

3.4.4 Effect of concrete strength

The specimens of group C60-1 had concrete strength of 53.8 MPa. Figure 3.16 presents a comparison of the results for C60-1 specimens and control group C40-1.

Figure 3.16 Rate effect comparison between specimens of groups C40-1 and C60-1 on (a) ultimate load, (b) ultimate displacement, (c) peak bond stress and (d) fracture energy

Among the four charts of Figure 3.16, due to higher concrete strength, the results for specimens of C60-1 group were all higher in the quasi-static tests. However, as slip rate increased to 30 mm/s, the gap between the two groups generally reduced, as shown in Figures 3.16(a), 3.16(b) and 3.16(d). At the high slip rates above 1000 mm/s,

the test results of both groups grew rapidly in the four charts, but the test results C60-1 were caught up by the control group of C40-1. In Figure 3.16(c), the behaviour of the peak bond stress of C60-1 was similar to its behaviour in the other three figures, except for the significant increase at a rate of 30 mm/s. Therefore, the general dynamic enhancement of the C60-1 specimens was lower than that seen for control group. Due to higher ultimate loads of C60-1 specimens under low slip rates, the corresponding axial loads in CFRP sheets were higher than those of C40-1 specimens, which resulted in a lower residual capacity for dynamic response before reaching the CFRP maximum load. This limit could also be seen in tests at slip rates above 1000 mm/s. The dynamic load values reached the CFRP maximum load and resulted in fracture failure of CFRP sheets of specimens S2-C60-1B and C, identical to the scenario shown in Figure 3.5(d).

From the experimental study, parametric effect on the bond interface is understood as follows. CFRP sheet with high stiffness could result in a stiffer bond and reduce the relative slip between CFRP and concrete. The CFRP sheet with a lower load capacity was observed by fracture failure in the sheet itself in the modified SHPB tests. However, a larger CFRP load capacity in C40-2 group could make the bond sustain a significant dynamic loading at the impact regime. In the comparison of specimens with different bond lengths, effective bond length was found to decrease due to the increase of concrete strength at a high slip rate. From the comparison of different concrete strengths, concrete of higher strength could make the bond less sensitive to dynamic enhancing effect.

3.5 Constitutive equations for dynamic bond-slip behaviour

The bond-slip behaviour is essential in studies of FRP-strengthened concrete structures [3, 23]. Considering the application in the structural analysis with dynamic loading, constitutive equations are proposed here to describe the dynamic bond-slip

behaviour. As the bond-slip models proposed by Lu et al. [3] have been validated in quasi-static analysis in Section 3.1, the constitutive equations in the current study are based on their quasi-static models and dynamic increase factor (DIF) equations proposed in this chapter. Two types of models are proposed for the dynamic bond-slip behaviour, including one model with exponential softening law for accurate simulation and the other bilinear model for simplified modelling.

The dynamic bond-slip model with exponential softening law in this study is shown as:

$$\tau = \begin{cases} ks, & \text{if } s \leq s_{II d} \\ \sigma_{II d} \cdot e^{-\alpha(\frac{s}{s_{II d}}-1)}, & \text{if } s > s_{II d} \end{cases} \quad (3.11)$$

where initial stiffness k is given by $k = \frac{\sigma_{II st}}{s_{II st}}$, quasi-static peak bond stress $\sigma_{II st}$ is given by Eq. (3.1), slip corresponding to quasi-static peak bond stress $s_{II st} = 0.0195\beta_w f_t$, $\alpha = \frac{1}{\frac{G_{II d}}{\sigma_{II d} s_{II d}} - 2}$. Dynamic peak bond stress $\sigma_{II d}$ and its corresponding slip $s_{II d}$ are determined by

$$\sigma_{II d} = DIF_{\tau} \cdot \sigma_{II st} \quad (3.12)$$

$$s_{II d} = \frac{\sigma_{II d}}{k} \quad (3.13)$$

where DIF_{τ} is DIF of peak bond stress under different slip rate.

Dynamic fracture energy $G_{II d}$ is determined by

$$G_{II d} = DIF_{GII} \cdot G_{II st} \quad (3.14)$$

where quasi-static fracture energy $G_{II st}$ is given by Eq. (3.2), DIF_{GII} is DIF of fracture energy under different slip rate.

The bilinear bond-slip model is a simplified model which is widely used in structural analysis of FRP strengthened concrete structures [3, 37]. The dynamic bilinear model is written as:

$$\tau = \begin{cases} ks, & \text{if } s \leq s_{II d} \\ \sigma_{II d} \frac{s_{II f} - s}{s_{II f} - s_{II d}}, & \text{if } s_{II d} < s \leq s_{II f} \end{cases} \quad (3.15)$$

where ultimate slip $s_{II f} = 2G_{II d}/\sigma_{II d}$.

DIF is the ratio of a dynamic property over its static state and is used to represent the dynamic enhancing effect of a material under a dynamic load. According to the analyses of test results presented in Sections 3.3 and 3.4 of this chapter, the dynamic bond behaviour of the FRP-concrete interface is close to the dynamic response of concrete material and is further affected by CFRP material parameters. The power function given by CEB-FIP Model Code 1990 [48] is used to determine the DIF of concrete material. As the dynamic response of the bond is close to that of concrete material, analogous power functions are used for the DIF of the peak bond stress ($\sigma_{II d}$) and the fracture energy ($G_{II d}$), as shown in Eq. (3.16a) and Eq. (3.17a), respectively. In the analysis of the test results in this chapter, the best-fit coefficients a and b in these equations are determined by concrete cylinder strength (f'_c) and CFRP load capacity (F_f), as shown in Eq. (3.16b) and Eq. (3.17b). The prediction curves of both DIF equations are presented in Figure 3.17.

$$DIF_{\tau} = \left(\frac{\dot{s}}{\dot{s}_0}\right)^a \quad (16a)$$

where the best-fit coefficient a is

$$a = 0.0334 * \ln \left(\frac{F_f/F_0}{f'_c/f_0} \right) - 0.0135 \quad (16b)$$

where $\dot{s}_0 = 0.004$ mm/s is regarded as the slip rate for quasi-static load, F_f is the load capacity of CFRP sheet, $F_0 = 1$ kN, f'_c is the cylinder concrete strength, $f_0 = 10$ MPa. As the current study tested specimens with 100 mm wide concrete substrate, the predictions with other widths of concrete substrate could be assumed to follow the equivalent width ratio as $\frac{b_{f0}}{b_{c0}} = \frac{b_{f-real}}{b_{c-real}}$, where b_{f-real} and b_{c-real} are real width values of CFRP and concrete, $b_{c0} = 100$ mm, b_{f0} is an equivalent CFRP bond width. Therefore, for those cases with different widths of concrete and CFRP

sheet, the CFRP load capacity can be determined by an equivalent maximum load of $F_f = f_{tf} t_f b_{f0}$, where f_{tf} is CFRP tensile strength.

The DIF equation for dynamic fracture energy is shown as

$$DIF_{GI} = \left(\frac{\dot{s}}{\dot{s}_0}\right)^b \quad (3.17a)$$

where the best-fit coefficient b is

$$b = 0.0264 * \ln \left(\frac{F_f/F_0}{f_c^t/f_0}\right) + 0.0295 \quad (3.17b)$$

(a)

(b)

Figure 3.17 DIF of (a) the peak bond stress and (b) the fracture energy

The dynamic bond-slip curves predicted by the exponential law and the bilinear law are presented in Figure 3.18(a) and 3.18(b), respectively. Compared with test results of C40-1 specimens, both models reasonably predicted the ascending and descending ranges of the bond-slip curves in the slip rate range from $4.70\text{e-}3 \text{ mm/s}$ to 2150 mm/s . Therefore, both models could be further used in structural analysis of CFRP strengthened RC structures subjected to blast loading.

(a)

(b)

Figure 3.18 Predicted dynamic bond-slip curves of C40-1 specimens by (a) the exponential model and (b) the bilinear model

To further verify the applicability of these DIF equations with different types of FRP, these equations were used to predict results of dynamic double-lap shear tests conducted by Shen et al. [23]. These tests used basalt fibre reinforced polymer (BFRP) and a higher FRP-concrete width ratio of 0.5. The material and geometric properties of their specimens were as follows: $f'_c=29.6$ MPa, $b_c=100$ mm, $f_{tf}=2300$ MPa, $E_f=105$ GPa, $t_f=0.121$ mm and $b_f=50$ mm. Figure 3.19 shows that the proposed DIF equations accurately predicted the fifty-one test results of this study and twelve experiments by Shen et al. [23], with acceptable mean prediction-to-test ratios of peak bond stress, fracture energy and ultimate load. Corresponding coefficients of variations (CoV) vary between 9.0% and 17.2%. R^2 falling between 0.77 and 0.89 is considered reasonable taking into account the nature of these materials and tests [25].

(a) peak bond stress

(b) fracture energy

(c) ultimate load

Figure 3.19 Comparison of DIF predicted values with test results for the (a) peak bond stress, (b) fracture energy and (c) ultimate load

3.6 Conclusion

In this study, 75 specimens were tested to investigate the dynamic enhancing effect on the interfacial bond-slip behaviour from quasi-static to high slip rates (0.004 mm/s to about 2000 mm/s). To achieve the high slip rate of 2000 mm/s, the author proposed a novel experimental method with a modified SHPB set-up. A gripping system was

added to clamp the CFRP sheet and spread the impact load from the incident bar. A high-frequency data logger was used to record strain data along the CFRP bond length. The test results and failure modes were analysed to gain understanding of the bond behaviour between CFRP and concrete structures. Debonding failure in concrete layer was observed for all quasi-static tests and most of dynamic tests. Additionally, debonding failure could also occur in both concrete and epoxy due to stress concentration under dynamic loading. However, at a significant high slip rate, the dynamic ultimate load could reach CFRP load capacity and the failure mode would switch from CFRP debonding from concrete substrate to CFRP fracture. Such a failure mode under high slip rate indicated that the bond strength could be limited by the CFRP load capacity.

In the analysis of interface bond behaviour, it was clearly found that, with an increase of slip rate, the bond properties including the ultimate loads, the ultimate displacement, the peak bond stress and the fracture energy showed dynamic enhancement. Since the relative slip between the two materials spread the load from CFRP to concrete, the dynamic behaviour of the interface bond is closely related to concrete dynamic response.

Through a detailed experimental parametric study, better understanding of the parametric effect on bond behaviour was achieved. Increasing the CFRP stiffness could result in a stiffer bond and reduce the relative slip between CFRP and concrete. When the effect of CFRP bond width was studied, it was found that CFRP width directly affected CFRP load capacity. At the impact regime, a lower CFRP load capacity could lead to the failure mode of CFRP fracture rather than debonding from concrete substrate. In this way, the interface bond could not withstand violent impact and CFRP sheet could not provide further protection for concrete structures. However, in the same regime, the CFRP-concrete bond joint with a higher CFRP load capacity

could sustain a significant dynamic loading. In terms of CFRP bond length, reducing the length below the effective bond length could decrease the ultimate load and ultimate displacement of the specimens at a low slip rate. However, at a high slip rate, specimens with a much shorter bond length could attain equal ultimate loads as those specimens with longer bond length. This phenomenon indicated that effective bond length would reduce due to dynamic increase of concrete strength. From the comparison of specimens with different concrete strengths, higher strength concrete could make the ultimate loads of the single-lap shear tests approach CFRP load capacity and result in the bond-slip behaviour becoming less sensitive to dynamic enhancing effect.

The constitutive equations were proposed for the dynamic bond-slip behaviour. Their accuracy was also validated through comparison with test results in the current study and in the literature. These equations can be used to define the mechanical behaviour for the CFRP-concrete interface in dynamic structural analysis. It should be noted that the scope of the DIF equations is limited to a concrete strength below 55 MPa and a CFRP load capacity below 25 kN. Further study of the dynamic interfacial properties is recommended for specimens with a higher concrete strength and a higher CFRP load capacity.

Chapter 4 Theoretical study of FRP-concrete dynamic shear debonding behaviour

4.1 Introduction

To understand the failure mechanism of this interface bond, shear debonding behaviour is usually examined through single-lap shear tests, in which an FRP sheet is bonded to a fixed concrete block and is subjected to tension [1], as shown in Figure 4.1.

Figure 4.1 Schematic diagram of a single-lap shear test

Based on numerous test data, several models [3, 4, 67] have been developed to describe the local bond behaviour and to predict the ultimate load values in the quasi-static regime. Different from well-established quasi-static studies, only a limited number of dynamic tests were conducted to study the effect of material and geometric parameters. Accordingly, some empirical models based on regression analyses were derived. Shi et al. [22] studied the effects of FRP type and concrete strength under a slip rate below 20 mm/s. Shen et al. [23] increased the slip rate to 70 mm/s but no material and geometric parameter was investigated. In Chapter 3, the author investigated parameters including carbon fibre reinforced polymer (CFRP) stiffness, CFRP bonding width and length and concrete strength under slip rates ranging from 0.004 mm/s to around 2000 mm/s and proposed dynamic constitutive equations for the interfacial shear debonding behaviour. Although these researchers have done dynamic tests and provided some empirical models, the analytical continuum model is still lacking. For realistic FRP strengthening cases, the effects of various material

and geometric characteristics are not yet well understood. To gain insight into the dynamic fracture mechanics of CFRP-concrete interface, a rate-dependent continuum damage model was proposed in this chapter to describe the nonlinear bond stress-slip performance for different CFRP bond widths, CFRP stiffnesses and concrete strengths.

The continuum damage model is derived from continuum mechanics and damage mechanics. For a material body idealised as a continuum, every point of the body can be represented by an element of an appropriate size and its material performance can be described by continuum mechanics [68]. Extending this approach to the bond between CFRP and concrete, the bond interface is also considered as a continuum and represented by a layer of interfacial bonding elements. As shown in Figure 4.2, a typical bond stress-slip curve of the interface element includes a stress ascending range and a softening range [3]. The O-A ascending range at the initial stage indicates linear elastic performance without damage, followed by stiffness degradation. The curve reaches its peak bond stress at B and shows softening performance in the B-C range. To describe such a nonlinear performance shown in Figure 4.2, a progressive damage function is used for the continuum damage model in both the ascending and the softening ranges.

Figure 4.2 Typical sketch of the bond stress-slip curve

Damage function is a practical tool to describe the stiffness deterioration of materials

and structures in fracture problems. Accordingly, damage functions have been developed for creep, fatigue, ductile and brittle damage scenarios [69]. In this study, the damage equation needs to be slip-rate dependent and considers the various effects of material parameters. Among existing damage equations, the creep damage equation developed by Kachanov L.M. [70] and Rabotnov Y.N. [71] has the capability to consider various crack scenarios and strain-rate effect. However, the creep damage equation is used to derive creep strain and rupture time of metal materials under a constant stress, which is not used to control stress softening behaviour under a constant strain rate. To apply it in this study, the author modify the damage equation by introducing a shape-control parameter to determine the specific shape of bond stress-slip response. Meanwhile, coefficients in the modified damage equation have new meaning and are determined through a semi-analytical method.

The overall framework of the present paper is illustrated in Figure 4.3, in which the development of the methodology is presented sequentially. In Section 4.2.1, the author proposed a novel continuum damage model for the dynamic bond stress-slip curves of CFRP-concrete interface. To identify dominant interfacial bond properties, equations of dynamic bond strength and dynamic fracture energy were obtained. Bond stiffness and static bond strength were quantified in Section 4.2.2, while the relations between model coefficients and material parameters were addressed in Section 4.2.3. To predict the CFRP debonding process, the proposed model was incorporated into interfacial stress equilibrium equations in Section 4.3. In Section 4.4, the numerical predictions for load-displacement curves, strain distributions in FRP laminate and specimens with various types of FRP were compared with test results in literature. Finally, the continuum damage model was converted into three-dimensional scalar formulations so that it could be applied in finite element analysis (FEA) of CFRP-concrete debonding behaviour.

Figure 4.3 Overall framework of present study

4.2 Development of the continuum damage model

4.2.1 Constitutive equations

This section aims to propose a continuum damage model for the dynamic debonding behaviour of CFRP-concrete interface. Some dominant interfacial properties, such as dynamic bond strength and dynamic fracture energy, could be derived from the equations of the continuum damage model. To describe the nonlinear performance of

interfacial bond stress-slip curve, the proposed continuum damage model includes a viscous damage parameter D into an elastic model, as expressed in Eqs. (4.1).

$$\sigma_{II} = (1 - D)K_{II} \cdot s_{II} \quad (4.1)$$

where damage parameter D is used to evaluate the magnitude of stiffness deterioration in the debonding process, which ranges from 0 when the bond is undamaged to 1 at fully damaged, σ_{II} is the bond stress (MPa), K_{II} is the initial stiffness (MPa/mm) and s_{II} is the relative slip (mm) between the CFRP and the concrete substrate.

Damage parameter D is determined by a damage rate equation, which is modified from an equation originally used in creep rupture [70, 71]. The damage equation in creep study is used to derive creep strain and rupture time under a constant stress. Related coefficients are obtained by fitting rupture time-stress data [72]. However, in this study, the damage equation needs to determine the stress softening behaviour under a constant slip rate. To control the specific shape of bond stress-slip curves, the author introduces static bond strength σ_{IIst} in this equation as a shape-control parameter. Its control effect is presented in Section 4.2.2. The modified damage rate equation is a power function of current state of bond stress τ and damage coefficient D as follows:

$$\dot{D} = B \frac{(\sigma_{II}/\sigma_{IIst})^n}{(1-D)^m} \quad (4.2)$$

where coefficients n , m and B , which control the dynamic bond strength and damage process, are obtained from static material parameters and will be discussed in detail in Section 4.2.3.

The analytical solution for $(1 - D)$ in Eq. (4.1) is required for obtaining a general expression for bond stress σ_{II} . To obtain an analytical expression for $(1 - D)$, Eq. (4.1) for σ_{II} is substituted into Eq. (4.2) and the damage rate equation can be expressed as:

$$(1 - D)^{m-n} \frac{dD}{dt} = B \left(\frac{K_{II} \cdot s_{II}}{\sigma_{IIst}} \right)^n \quad (4.3)$$

where t is time.

Integrating Eq. (4.3) with a constant slip rate $s_{II} = \dot{s}_{II} \cdot t$, the following expression can be obtained.

$$\frac{-(1-D)^{m-n+1}}{m-n+1} = B \left(\frac{K_{II} \cdot \dot{s}_{II}}{\sigma_{IIst}} \right)^n \frac{t^{n+1}}{n+1} + C_1 \quad (4.4)$$

where $\dot{s}_{II}$ is slip rate. At the starting point $t=0$, there is no damage and $D=0$. Incorporating the initial condition into Eq. (4), the solution for C_1 is found, i.e. $C_1 = -\frac{1}{m-n+1}$. Take $t = \frac{s_{II}}{\dot{s}_{II}}$ into Eq. (4) and then the analytical solution for $(1-D)$ is expressed as:

$$(1 - D) = \left(1 - \frac{m-n+1}{n+1} \left(\frac{K_{II}}{\sigma_{IIst}} \right)^n B \frac{s_{II}^{n+1}}{\dot{s}_{II}} \right)^{1/(m-n+1)} \quad (4.5)$$

Substituting Eq. (4.5) into Eq. (4.1), the rate-dependent constitutive equation for bond stress-slip becomes:

$$\sigma_{II} = \left(1 - \frac{m-n+1}{n+1} \left(\frac{K_{II}}{\sigma_{IIst}} \right)^n B \frac{s_{II}^{n+1}}{\dot{s}_{II}} \right)^{1/(m-n+1)} \cdot K_{II} s_{II} \quad (4.6)$$

Following this constitutive equation, some dominant interfacial properties of the bond stress-slip curve is concerned in the debonding process, such as dynamic bond strength $\sigma_{II d}$ and fracture energy $G_{II d}$. To derive the dynamic bond strength in Eq. (4.6), this equation is differentiated and is then set as $\frac{\partial \sigma_{II}}{\partial s} = 0$. The slip corresponding to the dynamic bond strength is obtained and defined as $s_{II d}$, as shown in Eq. (4.7). The peak bond stress under different slip rates is indicated in Eq. (4.8).

$$s_{II d} = \left(\left(\frac{\sigma_{IIst}}{K_{II}} \right)^n \frac{\dot{s}_{II}}{B} \left(\frac{n+1}{m+2} \right) \right)^{1/(n+1)} \quad (4.7)$$

$$\sigma_{II d} = K_{II} \left(\frac{n+1}{m+2} \right)^{1/(m-n+1)} \left(\left(\frac{\sigma_{IIst}}{K_{II}} \right)^n \frac{\dot{s}_{II}}{B} \left(\frac{n+1}{m+2} \right) \right)^{1/(n+1)} \quad (4.8)$$

Fracture energy $G_{II d}$ is defined as the area under the corresponding bond-slip curve [3] and it can be expressed as:

$$G_{II d} = \int \sigma_{II} ds_{II} \quad (4.9)$$

However, based on Eq. (4.6), it is not amenable to find an analytical expression for $G_{II d}$. A numerical integration is introduced here to generate the fracture energy. The slip rate $\dot{s}_{II}$ is a constant and the time step Δt is given, the slip increment Δs_{II} is determined through the following equation.

$$\Delta s_{II} = \Delta t \cdot \dot{s}_{II} \quad (4.10)$$

The slip at step i is equal to the accumulated slip increment and is derived as follows. In the calculations, i starts from 1 and $s_{II0}=0$.

$$s_{II i} = s_{II i-1} + \Delta s_{II} \quad (4.11)$$

The corresponding stress and damage parameter at step i can be generated from the stress-slip relation, as shown in Eqs. (4.12) and (4.13).

$$\sigma_{II i} = (1 - D_i) K_{II} \cdot s_{II i} \quad (4.12)$$

$$1 - D_i = \left(1 - \frac{m-n+1}{n+1} \left(\frac{K_{II}}{\sigma_{II st}}\right)^n B \frac{s_{II i}^{n+1}}{\dot{s}_{II}}\right)^{1/(m-n+1)} \quad (4.13)$$

Fracture energy $G_{II d}$ and released energy at step i (G_i) are expressed by Eqs. (4.14) and (4.15), respectively.

$$G_{II d} = \int_0^{s_{II f}} \sigma_{II} ds_{II} = \sum \frac{\sigma_{II i-1} + \sigma_{II i}}{2} \cdot \Delta s_{II} \quad (4.14)$$

$$G_i = \sum_{i=1}^i \frac{\sigma_{II i-1} + \sigma_{II i}}{2} \cdot \Delta s_{II} \quad (4.15)$$

where $s_{II f}$ is the ultimate slip.

4.2.2 Bond parameters

To apply the continuum damage model, interfacial bond parameters including bond stiffness K_{II} and static bond strength $\sigma_{II st}$ must be determined, as shown in Figure 4.3(Section 4.2.2). When the bond between the CFRP and the concrete is perfect, bond stiffness can be defined as follows [3].

$$K_{II} = \frac{K_{se} K_{sc}}{K_{se} + K_{sc}} \quad (4.16a)$$

where the epoxy shear stiffness is $K_{se} = \frac{G_e}{t_e}$, G_e is the epoxy shear modulus and t_e

is the thickness of epoxy layer. $K_{sc} = \frac{G_c}{t_c}$ is the concrete shear stiffness, determined by concrete shear modulus G_c and effective thickness t_c . Based on the meso-scale FE analysis by Lu et al. [3], effective thickness $t_c = 5$ mm is adopted here. However, the real bond is much softer with the relative slip between the two materials. Dai and Ueda [67] experimentally studied the local bond response and proposed a fitting coefficient γ for the stiffness value. With the fitting coefficient, the modified bond stiffness is shown as

$$K_{II} = \gamma \frac{K_{se}K_{sc}}{K_{se}+K_{sc}} \quad (4.16b)$$

where γ is given by $\gamma = 0.028(E_f t_f)^{0.254}$, $E_f t_f$ (GPa-mm) is the CFRP stiffness.

The quasi-static bond strength σ_{IIst} can significantly affect the general shape of the dynamic bond stress-slip curve. Figure 4.4 shows the bond stress-slip curve with varying σ_{IIst} values between 2 and 6 MPa, which are common bond strengths found in the published test results [3, 23, 67]. In this figure, bond stiffness and slip rate are given as $K_{II}=78.87$ MPa/mm and $\dot{s}_{II}=0.004$ mm/s (quasi-static slip rate), respectively. Coefficients in damage equation are tentatively given as $n=24.90$, $m=12.57$ and $B=0.022$. Under the quasi-static slip rate, parameter σ_{IIst} controls the peak bond stress and damage evolution. A lower value of σ_{IIst} reduces the peak bond stress of the curve and leads to greater brittle damage behaviour, as shown by the curve of $\sigma_{IIst}=2$ MPa. Therefore, an accurate value of σ_{IIst} is essential for realistic predictions of damage state under both quasi-static and dynamic regimes.

Figure 4.4 Bond stress-slip curves of continuum damage model with different values of σ_{IIst}

Since CFRP-concrete debonding commonly occurs in the concrete layer, quasi-static peak bond stress σ_{IIst} can be derived from concrete shear strength τ_{con} and CFRP bond width b_f . The bond width effect with different values of the CFRP sheet width b_f and the concrete substrate b_c was investigated by Chen and Teng [4]. Based on regression analysis, the width calibration factor was given as $\beta_w = \sqrt{\frac{2-b_f/b_c}{1+b_f/b_c}}$. The purpose of this parameter is to cater for specimens with different values of b_f and b_c . For the special condition that $b_f = b_c$, $\beta_w = \beta_0 = \sqrt{\frac{1}{2}}$. Incorporating width calibration factor β_w , static bond strength in this study is defined as a function of concrete shear strength τ_{con} as follows.

$$\sigma_{IIst} = \frac{\beta_w}{\beta_0} \tau_{con} \quad (4.17)$$

In accordance with the investigation by Guo and Shi [73], concrete shear strength was identical to its tensile strength, i.e. $\tau_{con} \approx f_t$. It is noteworthy that concrete tensile strength given by the fib code 2010 [74] is a function of cylinder compressive strength: $f_t = 0.3(f'_c)^{2/3}$. Through these equations, bond stiffness K_{II} and quasi-static bond strength σ_{IIst} can be determined using known material and geometric parameters of concrete and CFRP.

4.2.3 Coefficients of damage rate equation

The three coefficients in Eq. (4.2), namely n , m and B , determine the dynamic bond strength and rate-dependent damage process. In their original applications for creep rupture behaviour, coefficient n was the power controlling viscous performance; m was an additional parameter allowing the equation to fit various crack conditions and B was a constant determined from material properties. These coefficients were typically derived through fitting the stress-rupture time curves and the creep time-strain curves of creep tests [72]. However, such methods were not available for the bond stress-slip curves of CFRP-concrete interface with dynamic effect. In the current study, these coefficients are determined through a semi-analytical approach as follows.

In Eq. (4.8), the bond strength $\sigma_{II d}$ is a power function related to slip rate $\dot{s}_{II}$. To understand the effect of coefficient n , Eq. (4.8) is taken natural logarithm on both sides and Eq. (4.18) is obtained as follow.

$$\ln(\sigma_{II d}) = \frac{1}{n+1} \ln(\dot{s}_{II}) + \ln \left(K_{II} \left(\frac{n+1}{m+2} \right)^{\frac{1}{m-n+1}} \left(\left(\frac{\sigma_{II st}}{K_{II}} \right)^n \frac{1}{B} \left(\frac{n+1}{m+2} \right) \right)^{\frac{1}{n+1}} \right) \quad (4.18)$$

It is found that parameter $\ln(\sigma_{II d})$ is linearly related to $\ln(\dot{s}_{II})$ once n is assumed to be a constant. Accordingly, Eq. (4.18) can be further simplified by replacing the intercept with coefficient H as follows.

$$\ln(\sigma_{II d}) = \frac{1}{n+1} \ln(\dot{s}_{II}) + H \quad (4.19)$$

By now, it is clear that coefficient n relates dynamic bond strength $\sigma_{II d}$ with slip rate $\dot{s}_{II}$. Coefficient H represents the value of $\ln(\sigma_{II d})$ at slip rate $\dot{s}_{II}=1$ mm/s. To determine the values of n and H , the test results in Chapter 3 are used for data analysis, as shown in Figure 4.5. The four groups of tests conducted in Chapter 3 were named C40-1, C40-2, C40-3 and C60-1 depending on the different material and geometric properties, as shown in Table 4.1. Parameters such as f'_c , $E_f t_f$ and b_f/b_c

represent the cylinder concrete strength, the CFRP stiffness and the CFRP-concrete bonding width ratio, respectively. The CFRP sheets in their experiments had a tensile strength of 750 MPa and a bond length of 150 mm. Based on regression analysis through Eq. (4.19), the four fitting lines in Figure 4.5 are derived. The values of $\frac{1}{n+1}$ and H are determined by the gradients of $\ln(\sigma_{II,d})$ under different slip rates and the intercepts of these lines on the y axis, respectively. As shown in Figure 4.5 and Table 4.1, a lower value of n indicates a response more sensitive to the slip rate, e.g. the line for group C40-2. The best-fit values of n and H are given in Table 4.1.

Figure 4.5 Natural logarithm plot of bond strength and slip rate

Table 4.1 Fit constants for test groups in Chapter 3

Test	Material Properties	n	H	Δ	m	B
C40-1	$f'_c=32.0$ MPa, $E_f t_f=41.4$ GPa·mm, $b_f/b_c=0.33$	24.90	1.78	0.278	12.57	0.022
C40-2	$f'_c=32.0$ MPa, $E_f t_f=82.7$ GPa·mm, $b_f/b_c=0.33$	16.76	1.89	0.419	8.52	0.026
C40-3	$f'_c=32.0$ MPa, $E_f t_f=41.4$ GPa·mm, $b_f/b_c=0.26$	30.30	1.83	0.262	14.12	0.010
C60-1	$f'_c=53.8$ MPa, $E_f t_f=41.4$ GPa·mm, $b_f/b_c=0.33$	32.60	1.90	0.072	16.80	3.504

According to the findings in Section 3.4, the dynamic enhancing effect of the CFRP-concrete interface bond is determined by CFRP load capacity and concrete strength. Since the relative slip between the two materials transfers the load from CFRP to concrete, the mechanical relation between CFRP material and concrete material in such bonding interface is concerned. For CFRP sheet with a low load capacity, its

maximum load was easily reached in the dynamic test at a high slip rate and the CFRP sheet would fail due to fracture initiation instead of debonding from the concrete substrate. Accordingly, the interfacial bonding properties would not show further dynamic enhancing effects. The CFRP sheet with a higher load capacity could sustain a significant dynamic load and was unlikely to fail in the dynamic debonding process. The interface bond could then transfer a higher dynamic load from the CFRP to the concrete and present obvious dynamic enhancing effects. However, using concrete with a higher strength could make the ultimate load of the single-lap shear test approach CFRP load capacity and resulted in the bond-slip behaviour becoming less sensitive to dynamic enhancing effect. Therefore, the ultimate load ratio (μ) between the CFRP sheet and the concrete in tension is introduced here to indicate the relative difference in the loading capacities between the two materials and investigate its effect on the model coefficients.

The ultimate load ratio is calculated as a ratio of tensile capacity of the CFRP sheet and the concrete layer, as shown in Eq. (4.20) and Table 4.2. The concrete layer consists of cement matrix and coarse aggregates close to the surface [40]. In terms of cracking behaviour of concrete, Bazant and Oh [75] suggested that damage localisation occurred in a crack band width $w_c = 3d$, where d is the maximum aggregate size. As the maximum aggregate size of 10 mm is widely used in concrete material and there is usually a cement cover of 5 mm around the aggregates, a nominal depth of 35 mm is assumed for concrete layer in Table 4.2.

$$\mu = \frac{F_f}{F_{con}} = \frac{f_{tf}A_f}{f_tA_{con}} \quad (4.20)$$

where F_f is CFRP load capacity (N) determined by $F_f = f_{tf}A_f$, f_{tf} is CFRP tensile strength (MPa), A_f is CFRP cross-sectional interface area (mm²) determined by $A_f = b_f t_f$, F_{con} is concrete load capacity (N) given by $F_{con} = f_t A_{con}$, f_t is concrete tensile strength (MPa), A_{con} is concrete interface area (mm²) given by $A_{con} = 35b_c$.

Table 4.2 Ultimate load ratio (μ)

Test	Concrete tensile strength f_t (MPa)	Concrete interface area A_{con} (mm ²)	Concrete ultimate load F_{con} (N)	CFRP tensile strength f_{tf} (MPa)	CFRP interface area A_f (mm ²)	CFRP ultimate load F_f (N)	Ultimate load ratio (μ)
C40-1	3.00	100*35	10500	750	33*0.5	12375	1.18
C40-2	3.00	100*35	10500	750	33*1.0	24750	2.36
C40-3	3.00	100*35	10500	750	26*0.5	9750	0.93
C60-1	4.16	100*35	14560	750	33*0.5	12375	0.85

According to Eq. (4.2), Eq. (4.19) and Figure 4.5, a lower value of n indicates a larger dynamic enhancing effect on bond strength σ_{IId} . This trend is the same with the finding in Chapter 3 that an increase of CFRP load capacity or a decrease of concrete strength will lead to a more significant dynamic enhancing effect on interfacial properties. Therefore, coefficients n in Eq. (4.19) is found related to ultimate load ratio μ . Based on regression study (see details in Appendix A), the relation between ultimate load ratio μ and damage function power n is shown as follows.

$$n = 28.75\mu^{-0.64} \quad (4.21)$$

According to Eqs. (4.19) and Figure 4.5, coefficient H represents the value of $\ln(\sigma_{IId})$ at slip rate $\dot{s}_{II}=1$ mm/s. Its expression as Eq. (4.22) consists of the component of static bond strength $\frac{n}{n+1} \ln(\sigma_{IIst})$ and the increment component from quasi-static to dynamic state at a slip rate of 1 mm/s ($H - \frac{n}{n+1} \ln(\sigma_{IIst})$). The increment component is indicated by Δ in Eq. (4.22b).

$$H = \frac{n}{n+1} \ln(\sigma_{IIst}) + \ln \left(K_{II}^{\frac{1}{n+1}} \left(\frac{n+1}{m+2} \right)^{\frac{m+2}{(m-n+1)(n+1)}} \left(\frac{1}{B} \right)^{\frac{1}{n+1}} \right) \quad (4.22a)$$

$$H = \frac{n}{n+1} \ln(\sigma_{IIst}) + \Delta \quad (4.22b)$$

Based on regression analyses between increment Δ and ultimate load ratio μ , an

equation for Δ is obtained (see Appendix A for details). Substituting Δ into Eq. (4.22b), the expression for H is shown as follows.

$$H = \begin{cases} \frac{n}{n+1} \ln(\sigma_{IIst}) + 2.42\mu - 1.99, & 0.85 < \mu \leq 0.93 \\ \frac{n}{n+1} \ln(\sigma_{IIst}) + 0.11\mu + 0.16, & 0.93 < \mu \end{cases} \quad (4.23)$$

Following the discussion on n and H , the influence of coefficients m and B on the bond stress-slip curve is examined. According to the bond stress-slip relation in Eq. (4.6), the values of m obtained from regression analysis of the bond-slip results in Chapter 3 are shown in Table 4.1. It is found that a smaller value of m can mitigate the damage extent in the softening range and improve fracture energy. Therefore, coefficient m is related to fracture energy G_{IId} . The relationship between coefficient m and ratio μ is similar to that between n and μ , and the best-fit function between the two coefficients is given as follows.

$$m = 14.24\mu^{-0.61} \quad (4.24)$$

Finally, after some manipulation of Eq. (19a), the final coefficient B can be obtained from the known coefficients n , m and H .

$$B = e^{(n+1)(\ln\left(K_{II}^{\frac{1}{n+1}}\left(\frac{n+1}{m+2}\right)^{\frac{m+2}{(m-n+1)(n+1)}}(\sigma_{IIst})^{\frac{n}{n+1}}\right) - H)} \quad (4.25)$$

4.3 Numerical method to predict debonding process

To verify the continuum damage model in CFRP-concrete dynamic debonding process, the model is used to obtain load-displacement histories of the CFRP-concrete shear tests and to simulate the debonding process. Taljsten, Yuan et al. and Cornetti and Carpinteri [37, 76, 77] used fracture mechanics to study the relation between fracture energy of local bond and external load sustained by the CFRP sheet. According to their studies, in the single-lap shear test shown in Figure 4.1, the relation between released energy G and external load P at the loaded end is expressed as:

$$P^2 = \frac{2Gb_f^2E_ft_f}{1+\rho} \quad (4.26)$$

where $\rho = \frac{E_ft_fb_f}{E_ct_cb_c}$.

Since the released energy at time step i (G_i) can be obtained by numerical integration via Eq. (4.15), corresponding external load P_i can be predicted via Eq. (4.27).

$$P_i = b_f \sqrt{\frac{2G_iE_ft_f}{1+\rho}} \quad (4.27)$$

For the load-displacement curve, the displacement for load P_i at the loaded end is represented by the slip value at the loaded end.

When the released energy attains fracture energy G_{IIa} , local bond near the loaded end loses its capacity and fracture extends further in the bonded region. Eq. (4.27) can be rewritten as Eq. (4.28), indicating that the maximum load P_u is attained at this instant and will be maintained in the following crack process. This fracture process continues until the interfacial shear stress propagates to the bonded end of the sheet. Thus, the process is terminated, and the load value drops to zero.

$$P_u = b_f \sqrt{\frac{2G_{IIa}E_ft_f}{1+\rho}} \quad (4.28)$$

To verify the continuum damage model in detailed debonding process, the distributions of CFRP axial strain and interfacial shear stress along the bond length are analysed with the model proposed in this chapter. Yuan et al. [37] investigated the stress equilibrium condition and gave corresponding equations between the plate and the concrete, as shown in Figure 4.6 and Eqs. (4.29) and (4.30).

Figure 4.6 Sketch of stress equilibrium

$$\frac{d\sigma_f}{dx} - \frac{\tau}{t_f} = 0 \quad (4.29)$$

$$\sigma_f t_f b_f + \sigma_c t_c b_c = 0 \quad (4.30)$$

where σ_f and σ_c are CFRP axial stress and concrete tensile stress in Figure 4.6, respectively. According to Hook's law, the respective stress equation for CFRP and concrete substrate are given as:

$$\sigma_f = E_f \frac{du_f}{dx}, \quad \sigma_c = E_c \frac{du_{con}}{dx} \quad (4.31)$$

where u_f and u_c are the longitudinal displacements of the composite and the concrete, respectively. The relative displacement between both adherents s_{II} is given by:

$$s_{II} = u_f - u_{con} \quad (4.32)$$

Substituting Eqs. (4.30) and (4.32) into Eq. (4.31), axial stress σ_f become a function of the term $\frac{ds_{II}}{dx}$, as shown below.

$$\sigma_f = \frac{E_f}{1+\rho} \frac{ds_{II}}{dx} \quad (4.33)$$

where $\rho = \frac{E_f t_f b_f}{E_c t_c b_c}$, the solution for $\frac{ds_{II}}{dx}$ is expressed as $\frac{ds_{II}}{dx} = \frac{\sigma_f(1+\rho)}{E_f}$.

Based on the study of interfacial stress equilibrium equations, the distribution of axial stress is derived using decreasing gradient equations, as shown in Eqs. (4.34) and

(4.35). Since CFRP is elastic, the distribution of axial strain in this study can be obtained via the corresponding axial stress over Young's modulus E_f .

$$\sigma_{ij-1} = \sigma_{ij} - d\sigma_{ij} \quad (4.34)$$

$$d\sigma_{ij} = \frac{\tau_{ij}}{t_f} dx \quad (4.35)$$

where i and j mean time step and position on the bond length, respectively, the shear stress τ_{ij} is derived from the $\sigma_{II} - s$ function described by Eq. (4.6).

The shear stress distribution is related to the slip value of each point along the bond length. Iterative Eqs. (4.36) and (4.37) are given to calculate slip values of each point. The shear stress of each point is then determined through the bond stress-slip function (Eq. (4.6), $\sigma_{II} = \tau$). At a constant slip rate $\dot{s}_{II}$, the numerical solutions of the shear stress distribution are then generated by iteration with Eqs. (4.6) and (4.34-4.37).

$$s_{ij-1} = s_{ij} - ds_{ij} \quad (4.36)$$

$$ds_{ij} = \frac{\sigma_{ij}(1+\rho)}{E_f} dx \quad (4.37)$$

To present the proposed computing procedure, a flow chart for numerical predictions is given in Figure 4.7. In the first step, the quasi-static peak stress σ_{IIst} , bond stiffness K_{II} and ultimate load ratio μ of the interface bond are required to determine the model coefficients n , m and B . With these determined coefficients, the bond stress-slip curve under a target slip rate $\dot{s}_{II}$ could be generated and be used in the following computation. Fracture energy G_{IIId} can be obtained by numerically integrating the bond stress-slip curve. Followed by the third step, the external load P could also be derived through the equation which defines the relation between energy release rate and external load. To obtain the detailed debonding process in Step 4, including load-displacement curve, CFRP strain and interfacial stress distributions along the bond length, the bond stress-slip curve is substituted into interfacial stress equilibrium equations (Eqs. (4.34) to (4.37)) to do iterative computation.

Figure 4.7 Flow chart for the numerical method

4.4 Comparison of numerical predictions and experimental results

To verify its applicability, the continuum damage model is combined with the numerical equations introduced in Section 4.3 to predict the test results of four groups of specimens in Chapter 3 at slip rates ranging from 0.004 mm/s to around 2000 mm/s. The material parameters are given in Table 4.1. The coefficients needed for the numerical solutions, including n , m and B were derived from Eqs. (4.21), (4.24) and (4.25), respectively. Using the material and geometric parameters given in Tables 4.1 and 4.2, the predicted bond stress-slip curves, fracture energy and ultimate loads are calculated via Eqs. (4.6), (4.14) and (4.28), respectively. The verification work starts from the bond stress-slip curve for applicability of local bond. After obtaining the load-displacement curve and CFRP axial strain distribution, the general load process and corresponding CFRP axial response are evaluated. In this way, local bond behaviour is correlated with the general load capacity. Further verifications on specimens with other types of FRP are conducted by a comparison of ultimate load P_u between model predictions and test results from Shen et al. [23], Huo et al. [78] and Salimian and Mostofinejad [24].

4.4.1 Behaviour of the bond stress-slip curve

A comparison of the predicted and experimental bond stress-slip curves under a quasi-static load is shown in Figure 4.8(a). In the ascending phase, the numerical prediction curve matches the test result very well, including the stiffness and the peak bond stress. In the softening phase, the numerical prediction curve decreased smoothly but the test result deviated somewhat. It is common that numerical result is ideal but test result presents practical crack process and has some irregular behaviours. Integrating both curves along the slip, as shown in Figure 4.8(b), the values of local fracture energy were close, 0.663 N/mm for the numerical prediction and 0.631 N/mm for the test result. Therefore, the continuum damage model accurately predicted the bond-slip curve of M1-C40-1C.

Figure 4.8 Comparison of (a) bond stress-slip curves and (b) fracture energies for M1-C40-1C (quasi-static test)

Figure 4.9 presents a general comparison of bond stress-slip curves between the numerical predictions and the test results in Chapter 3 from quasi-static to impact regimes. It is noticed that the dynamic enhancing effect on test results is well predicted by the numerical model. Among the six specimens, the numerical model accurately predicts the peak bond stress for most specimens, with deviations between 3% and 9% from the test results. Only for the curves at slip rates of 0.23 mm/s and 2.46 mm/s, their numerical predictions overestimate the peak bond stresses by 15% and 11%, respectively. When the local fracture energy is calculated, apart from the prediction at 1127 mm/s, the numerical solutions properly predict the test results with differences ranging from 4% to 12%. However, a difference of 46% is noticed for the discrepancy between the numerical prediction and the selected test result at 1127 mm/s. As indicated in the test programme in Section 3.2, three tests are conducted under the same loading condition to confirm their repeatability. Different from the large difference seen in the case of specimen S1-C40-1C (1127 mm/s), the numerical predicted G_{IIa} is close to the other two results under the same loading condition, viz S1-C40-1A and S1-C40-1B, with prediction-to-test ratios of 1.16 and 0.96. Therefore, the remarkable difference in specimen S1-C40-1C could be attributed to an irregular

test result, which was highlighted by the data from the other two test results.

Figure 4.9 Comparison of bond stress-slip curves at different slip rates

(The selected specimens for these slip rates: M1-C40-1C (4.70e-3 mm/s), M2-C40-1B (0.23 mm/s), M3-C40-1A (2.46 mm/s), M4-C40-1C (26.85 mm/s), S1-C40-1C (1127 mm/s) and S2-C40-1B (2150 mm/s))

Figure 4.10 shows the damage evolution of the numerical solutions at various slip rates. With increasing slip rates, the damage accumulation is delayed and the index increases more slowly. Due to this, a larger softening range is presented for a curve at a higher slip rate in Figure 4.9 and a higher fracture energy is achieved. Meanwhile, in Figure 4.10, a larger ultimate slip is required for the damage index to approximate 1 and then the specimen totally loses its loading capacity.

Figure 4.10 Damage response at different slip rates

4.4.2 Predictions of the debonding process

The detailed CFRP-concrete debonding process is investigated using the load-displacement curve, the interfacial shear stress and CFRP axial strain distributions along the bond length. Therefore, the values produced with numerical solutions are compared with test results to verify the predicted debonding process.

The load-displacement curves shown in Figure 4.11(a) compare the numerical prediction and the test result for the quasi-static condition. Identical to the test curve, the prediction curve showed a linear increase at initial displacements, indicating no softening behaviour in the interfacial bond. After the displacement reached 0.1 mm, the gradient gradually decreased to zero due to the softening behaviour of the bond around the loaded end. The prediction reaches its ultimate strength as it flattens. This loading level is maintained until debonding happens at a displacement of 0.61 mm. Compared with the numerical prediction, the test result shows more oscillation between the displacements of 0.2 mm to 0.4 mm, which is attributed to surface roughness and non-uniformly distributed aggregates in the concrete.

Figure 4.11 Debonding process of M1-C40-1C, (a) load-displacement curve; (b) shear stress distribution at different loading stages

To illustrate shear stress distribution at different moments, four different loading stages (A, B, C and D) along the prediction curve are selected in Figure 4.11(a) and the corresponding shear stress distributions are presented in Figure 4.11(b). Point A represents the moment at which the bond stress at the loaded end reaches its peak value. Following this, at point B the bond stress at the loaded end is in its softening range and the peak bond stress spreads to a further position in the bond area. When the load reaches its ultimate value at point C, the bond stress at the loaded end decreases to a low magnitude. Finally, at point D the shear stress distribution shape is similar to that of point C and moves to the bond end.

Figure 4.12 presents the load-displacement curves of the same specimens in Figure 4.9. The numerical solutions accurately predict the ultimate loads from quasi-static to high slip-rate scenarios with differences lower than 10%. Compared with the variations seen in the test results, the initial gradients of numerical prediction curves are similar. It is due to that the estimate of the initial stiffness K_{II} does not incorporate dynamic enhancing effect and is maintained a constant for all slip rates.

Figure 4.12 Load-displacement curves at different slip rates

The final displacement values between the tests and the numerical predictions are then analysed. As shown in Table 4.3, the numerical results are generally in good agreement with the test results and predicted larger plastic deformation capacities under higher slip rates. Compared with the prediction of 0.61 mm for the M1-C40-1C (slip rate at 4.70e-3 mm/s), a higher displacement value of 0.7 mm was reached by the test result. This could be due to stress concentration at the bond end of the test specimen caused by the CFRP plate bending [1]. Since such stress concentration just occurs at the bond end and does not affect the interface analysis very much, this effect is not discussed in this chapter. Except specimen M1-C40-1C, the numerical results were quite close to the test results with differences between 2% and 8%.

Table 4.3 Comparison of final displacement between test results and numerical predictions

Specimen (slip rate, mm/s)	Test (mm)	Numerical results (mm)	Ratio
M1-C40-1C (4.70e-3)	0.70	0.61	0.87
M2-C40-1B (0.23)	0.75	0.70	0.93
M3-C40-1A (2.46)	0.83	0.76	0.92
M4-C40-1C (26.85)	0.87	0.84	0.97
S1-C40-1C (1127)	0.94	0.96	1.02
S2-C40-1B (2150)	1.07	1.00	0.93

Detailed debonding process at different loading stages can be studied by observing CFRP strain distributions along the bond length. Strain distribution plots are obtained from raw data measured by the strain gauges located on the CFRP and it is useful to understand the loading process along the bond length. These distributions are also numerically predicted for the same six specimens, as shown in Figures 4.13(a) to 4.13(f) for tests at slip rates of 4.70e-3 mm/s, 0.23 mm/s, 2.46 mm/s, 26.85 mm/s, 1127 mm/s and 2150 mm/s, respectively. With the increase of the load from $0.25P_u$ to P_u , all test specimens showed a similar evolution, with the CFRP strains extending from the loaded end to the bond end. As the slip rates increase from 4.70e-3 mm/s to 2150 mm/s, higher strain values at the loaded end are noticed at all the four loading stages, viz $0.25P_u$ to P_u .

Figure 4.13 Strain distribution along the bond length

The strain distribution curves for the lowest loading level of $0.25 P_u$ are well predicted for the six specimens. This indicates that the initial range of the bond stress-slip response is properly determined in the numerical method. However, the predictions became less accurate for higher loads. In Figures 4.13(b) and 4.13(d), the experimental points are scattered around the prediction curves for $0.75 P_u$ in the distance range from 30 mm to 60 mm. A similar scenario is also noticed that test points are around the predictions for the loading stage P_u in Figures 4.13(a), (c) and (d). The difference is caused by the non-uniformly distributed aggregates in the concrete substrate along the CFRP bond length. Large aggregates can cause a higher local stiffness which reduces the deformation of the CFRP sheet. Accordingly, the

strains decrease rapidly when the local concrete zone is more rigid and vice versa. However, model predictions could not consider such non-uniform distribution of aggregates and present smooth curves as usual. When the ultimate strains at the loading stage P_u are concerned, the numerical method accurately predicts the strain values in Figures. 4.13(a) to (d). In Figures. 13(e) and (f), the numerical method slightly overestimates the values by 7.6% and 6.1%, respectively. According to these data analyses, the numerical method can reasonably predict the strain distributions along the CFRP bond length under various slip rates.

4.4.3 General comparisons

To fully evaluate the numerical method proposed in this chapter, the model predictions are compared with the test results of the four test groups in Chapter 3 in Figure 4.14, viz. C40-1, C40-2, C40-3 and C60-1. Properties for comparison include bond strength $\sigma_{II,d}$, fracture energy $G_{II,d}$ and ultimate load P_u . The prediction-to-test ratios are also given to assess the accuracy of numerical predictions. The average ratio of bond strength is 1.00 with a coefficient of variation (CoV) of 0.09 in Figure 4.14(a), indicating that the numerical method could accurately predict dynamic bond strength of these shear tests. However, a larger CoV of 0.18 is noted for the average ratio of fracture energy. Reviewing this scatter, it is found that large variations of the test results in Figure 4.14(b) contribute to this larger CoV. The average ratio of ultimate load of 1.00 with a CoV of 0.10 in Figure 4.14(c) is also considered to be precise. In summary, the proposed model could reasonably predict the CFRP-concrete debonding behaviour with various material properties from quasi-static to impact regimes.

(a) bond strength

(b) fracture energy

(c) ultimate load

Figure 4.14 Comparison of predictions with test results

4.4.4 Comparison with other dynamic tests

The continuum model is also applied to predict ultimate load of dynamic shear tests in three other studies [23, 24, 78] to enhance its applicability with different types of FRP. Shen et al. [23] used a single-layer basalt fibre reinforced polymer (BFRP) to investigate the dynamic effect on ultimate load of BFRP-concrete interface. Both single-layer and double-layer glass fibre reinforced polymer (GFRP) sheets were studied by Huo et al. [78] to investigate the effect of number of layers in dynamic

shear tests. Similar to the study in Chapter 3, Salimian and Mostofinejad [24] used single-layer CFRP laminate in the shear tests but with a lower concrete cylinder strength of 22 MPa. Specimens in the three studies had larger FRP bond widths than those in Chapter 3, namely, 48 mm, 50 mm and 80 mm, which could also be used to evaluate the applicability of the proposed model with larger bond widths. The detailed material and geometric properties of the three studies are given in Table 4.4.

Table 4.4 Detailed material and geometric properties

Properties	Shen [23]	Huo [78]	Salimian [24]
Number of tests	12	27	12
f'_c (MPa)	29.6	28	22, 48
b_c (mm)	100	100	150
FRP type	BFRP	GFRP	CFRP
f_{tf} (MPa)	2300	1434	350
E_f (GPa)	105	78	28
t_f (mm)	0.121	0.169, 0.338	1.0
b_f (mm)	50	50, 80	48
μ	1.52	1.25, 2.56, 4.10	1.36, 1.05
Slip rate ^a (mm/s)	0.07, 0.7, 7, 70	0.002, 216-907 ^b	0.03, 0.67, 1.0

^aShen and Salimian used loading rate to evaluate the dynamic effect on their tests and the loading rates are assumed equal to the slip rate;

^bHuo used strain rate $\dot{\epsilon}$ to evaluate the dynamic effect on their tests. According to Bazant et al. [79], the slip rates in dynamic tests are assumed to be $\dot{s} = L \cdot \dot{\epsilon}$, where L is cohesive crack length and is assumed to 100 mm.

The comparison of ultimate load P_u between the test results and numerical predictions is presented in Figure 4.15. The numerical method predicts the ultimate load with an average prediction-to-test ratio of 1.06 and a CoV of 0.18. It can be seen that the predicted load values match those of Shen [23] and Salimian [24] very well, and the continuum damage model can be applied on specimens with wider FRP laminate and lower strength concrete. In Figure 4.15, the scatter between predictions and test results is observed in some specimens of Huo's study [78]. This discrepancy is due to the discrete test results in dynamic configurations. Moreover, these specimens used a double-layer GFRP sheet and the GFRP load capacity was higher

than those single-layer composites. In this way, a higher ultimate load ratio μ of above 2.5 is predicted by the numerical method in Table 4.4, which indicates that the bond is very sensitive to dynamic effect. As a result, the numerical method overestimated the ultimate loads for some specimens. Since the numerical method precisely predicts test configurations of Chapter 3, Shen and Salimian [23, 24] with ultimate load ratios in a range of 0.85 to 2.36, such overestimation in some of Huo's specimens [78] indicates that the numerical method may be less accurate when the ultimate load ratio is over 2.5. In general, the comparison presented in Figure 4.15 is accurate with a R^2 of 0.84.

Figure 4.15 Comparison of numerical predictions with test results

4.5 Finite Element Analysis

4.5.1 The three-dimension constitutive equations

To apply the continuum damage model into FEA, the model developed in Section 4.4.2 is converted into three-dimension scalar formulations and is used as the interface element between CFRP and concrete. A total of 51 tests in Figure 4.14 are simulated by the FE model with the interface element described in this section. Three-dimension (3D) constitutive equations are given as follow with subscripts I for normal direction, II for the in-plane shear directions. Since normal debonding is not

investigated in this study, the function in normal direction is assumed to be the same as that in the shear direction.

The total mixed mode slip is given by:

$$s_m = \sqrt{(s_I)^2 + (s_{II})^2} \quad (4.38)$$

Slip rate in mixed mode is shown in function below.

$$\dot{s}_m = \sqrt{(\dot{s}_I)^2 + (\dot{s}_{II})^2} \quad (4.39)$$

Substituting Equations (4.38) and (4.39) into Eq. (4.5), the damage parameter $(1 - D)$ in mixed mode is written as below.

$$(1 - D) = \left(1 - \frac{m-n+1}{n+1} \left(\frac{K}{\sigma_{mst}}\right)^n B \frac{(s_m)^{n+1}}{\dot{s}_m}\right)^{1/(m-n+1)} \quad (4.40)$$

where σ_{IIst} is the quasi-static peak bond stress in mixed mode and is assumed to be $\sigma_{mst} = \sigma_{IIst}$, stiffness K is assumed to be $K = K_{II}$. Coefficients n , m and B are given by Eqs. (4.21), (4.24) and (4.25), respectively. The stress components in each direction are shown in Eqs. (4.41) and (4.42). To prevent penetration between CFRP and concrete material through interface element, the penalty function is used in Eq. (4.41) when $s_I < 0$. Corresponding 3D domain for the constitutive equations are shown in Figure 4.16.

$$\sigma_I = \begin{cases} K s_I, & s_I < 0 \\ (1 - D) K s_I, & s_I \geq 0 \end{cases} \quad (4.41)$$

$$\sigma_{II} = (1 - D) K s_{II} \quad (4.42)$$

Figure 4.16 3D domain for the constitutive equations

4.5.2 FE model

The LS-DYNA commercial software is used for the FE simulations in this study due to its common application in explicit dynamic simulations. The 3D constitutive equations are then programmed into a user-defined interface element in LSDYNA. The material properties of the interface element, including the stiffness K , the quasi-static peak bond stress σ_{mst} and the ultimate load ratio μ , can be defined in the card UMAT in the software. Element deletion is defined for the interface elements to simulate the debonding process. Due to the linear elastic behaviour along the fibre direction, the CFRP sheet is simulated using an elastic model. The concrete component is then simulated with K&C concrete model [44] which is a common plastic concrete material model for dynamic simulations [46]. As a convenient model in LS-DYNA, the K&C model can automatically generate model parameters based on the concrete strength. The input material parameters for the CFRP, the concrete and the interface element are given in Tables 4.1 and 4.2. Since concrete material is quite sensitive to the dynamic enhancing effect, dynamic increase factors (DIFs) are also defined in the concrete model according to the LS-DYNA material manual [47]. To improve the computational efficiency, the three-dimensional CFRP-concrete specimen is simplified into a plane stress problem and simulated by a single layer of

solid elements.

Since Lu et al. [18] validated their meso-scale simulation method in the comparison with various FRP-concrete lap shear tests, this study follows their method to model the layout of CFRP and concrete. A sketch of the CFRP-concrete simulation and the corresponding FE model are shown in Figure 4.17. The length and height of concrete block are 180 mm and 50 mm, respectively. The thickness values of CFRP and the interface layer are set to 1 mm to avoid size problem by modelling the real thickness of CFRP in solid element, viz 0.17 mm. Accordingly, CFRP elastic modulus is adjusted to maintain the same CFRP stiffness $E_f t_f$. The model is restrained along the bottom and the front edge h_2 (30 mm) to model the boundary condition of the tests in Chapter 3. The CFRP is in the top of the FE model with bonding length of 150 mm.

(a)

(b)

Figure 4.17 (a) Geometry sketch and (b) FE model

Apart from the development of the interface element, perfect bond is also frequently used to simulate the FRP-concrete debonding performance [18, 80]. The perfect bond simulations are conducted by removing the interface elements in Figure 4.17(b) and directly sharing nodes between the CFRP and the concrete. Therefore, the debonding failure in these simulations is shown as damage of concrete material at the zone close to CFRP material. To reasonably present the damage of concrete material, fine mesh (0.5 to 2 mm) is always required for such FE models. The difference on dynamic debonding simulations between the interface element method and the perfect bond method is concerned in this study. The proposed FE model in Figure 4.17 is compared with that with perfect bond on the simulations of single-lap shear tests of all specimens in Figure 4.14. Mesh size of 2 mm is used in both kinds of FE models.

4.5.3 Result discussion

Since a detailed parametric study is done by using the numerical solutions in Section 4.4, the FE model with interface elements studies the CFRP-concrete debonding process and compares predicted results with those of the FE model with perfect bond. The results of the FE simulation of specimen M4-C40-1C, including the load-displacement curve and the strain contours before loading and at the moment just after reaching the ultimate load, are shown in Figures 4.18(a), (b) and (c), respectively. At the loading stage A in the load-displacement curve, the interface elements are intact, as shown in Figure 4.18(b). At the loading stage B, as shown in Figure 4.18(c), the load reaches the ultimate load value and the interface element failure occurs at the loaded end. Subsequently, the ultimate load is maintained as shown in Figure 4.18(a) and the interface elements fail one by one from the loaded end. Therefore, the FE model with interface element could properly simulate the real debonding process.

(a) FE model predicted load-displacement curve

(b) CFRP-concrete debonding at loading stage A

(c) CFRP-concrete debonding at loading stage B

Figure 4.18 FE simulations of specimen M4-C40-1C

Figures 4.19(a) and (b) show the comparisons between test results and FE simulations with interface elements and perfect bond, respectively. In Figure 4.19(a), the FE models with interface elements predicted the ultimate loads well for all rates, ranging from quasi-static to impact scenarios with various material properties. One exception is that the FE model overestimates ultimate load of C40-3 group under slip rates above 1000 mm/s. This is because CFRP is assumed to be a pure linear elastic material without plastic and fracture behaviours in the FE model, which leads to overestimating the capacity of the specimen in an extreme scenario. As a comparison

in Figure 4.19(b), the FE model with perfect bond predicts ultimate loads well below a slip rate of 50 mm/s. However, unlike the FE model with interface elements, the capacities of all specimens tested at high slip rates are dramatically overestimated. These invalid predictions could be due to the perfect bond ignoring the relative slip between the two materials and therefore making the concrete component directly resist the dynamic loading. Therefore, the perfect bond model is not appropriate to predict the debonding performance of the system under high slip rate.

(a) Test results and FE model with interface element

(b) Test results and FE model with perfect bond

Figure 4.19 Comparison between test results in Chapter 3 and FE model

4.6 Conclusion

To better understand parametric effects and to provide a useful interface model for the bond between CFRP and concrete, this study proposed a continuum damage model to describe rate-dependent bond stress-slip curves. Model analyses and regression test data were conducted to study the physical relationships between the model coefficients and the characteristic parameters of the test specimens. It was found that the dynamic enhancing effect becomes more significant due to a larger ultimate load ratio of the CFRP to the concrete. A larger ultimate load ratio indicates a relatively higher CFRP load-carrying capacity than that of concrete, which causes the interface bond to transfer a larger dynamic load from the CFRP to the concrete at the impact regime. The effect of this ratio is verified in the range of 0.85 to 2.36 and under the slip rates from $3.5\text{e-}3$ mm/s to 2150 mm/s. Its applications in a higher ratio of over 2.5 and an extreme high slip rate of over 2500 mm/s still need further investigation. After developing the model, stress equilibrium equations are added to generate functions for the CFRP-concrete load-displacement curve, shear stress and CFRP strain distributions along the bond length. In this way, local bond behaviour was related to load response and debonding process of dynamic shear tests.

Following the development of the numerical method, it was validated by comparing its predictions with test results in detail. These included the bond stress-slip curves, the load-displacement curves and the strain distribution along the bond length. The predictions are generally in good agreement with test results. Some of the predictions are different from the test data to a large degree. However, given that the results of repeated tests showed closer agreement with the numerical results, the differences seen in these isolated cases are deemed acceptable.

Additionally, to investigate the applicability of the proposed model to different types

of FRP, the continuum damage model was used to predict the dynamic shear test using BFRP and GFRP. Comparison results showed that the proposed model was able to predict the ultimate loads accurately for specimens with different FRP materials and larger FRP bond widths.

Apart from numerical solutions, the continuum damage model was converted into three-dimension scalar formulae and was applied in FE simulations of CFRP-concrete debonding tests through a LS-DYNA user-defined material model. The predictions of FE model were compared with those of numerical method. The comparison results indicated that both approaches could accurately predict dynamic shear tests and the application of the continuum damage model into FEA is proved to be available. In future, the proposed continuum damage model can be used to simulate dynamic bond performance in analytical studies and FE models of FRP strengthened RC structures under impact and blast loads.

Chapter 5 Experimental study of FRP-concrete dynamic tensile fracture behaviour

5.1 Introduction

Since the shear debonding is the primary failure mode in quasi-static scenarios [3], the shear bond behaviour has been extensively investigated through single-lap and double-lap shear tests [1, 3-5]. However, the tensile fracture failure, which was found in dynamic tests [12-14, 17, 27], was investigated by few researchers. The FRP delamination from the concrete substrate can spoil the bond at a lower loading level [6]. Therefore, there is a need to characterize the mode I dynamic fracture of the interface bond. It is found that two key properties dominate the nonlinear fracture process of the bond, viz tensile bond strength σ_{ld} and the fracture energy G_{ld} [7].

As the characteristics of the mode I fracture are hard to be derived from a simple test, different test procedures have been proposed to determine the dominant properties. To clarify their difference, these test procedures are referred to tensile tests and fracture tests for determining tensile bond strength σ_{ld} and the fracture energy G_{ld} , respectively. For concrete material, there are three test methods to measure the tensile strength, viz split cylinder test, beam bending test and direct tension test [28]. The tensile stress in the first two methods is imposed indirectly on the test specimens. In split test, concrete cylinder is placed horizontally and is subject to compression load at the two edge along the diametrical direction. Beam bending test works by adding three-point or four-point bending load on the concrete beam. However, direct tension test can measure the concrete tensile strength under uniaxial tensile load, which is more rational and reliable [29]. For FRP-concrete interface, to the author's knowledge, there is few tests conducted for the tensile bond strength. Coronado and Lopez [7] conducted split tests for the cylinder and prismatic specimens with a layer

of epoxy at the centre line. The derived splitting tensile strength for the bonded interface was close to that of plain concrete. Compared with the split test, direct tension test can give results closer to the true tensile strength [28] and was used in this study to measure the tensile bond strength of FRP-concrete interface. Moreover, the dynamic response of this property was also investigated with dynamic tests.

The fracture energy is necessary in the analysis of crack softening behaviour. Therefore, a few fracture tests have been carried out to study the mode I fracture energy of the FRP-concrete interface [6, 7, 30-32]. In these studies, the ways to determine mode I fracture energy of the bond could be divided into linear elastic (LEFM) and nonlinear fracture mechanics (NLFM) methods. In the LEFM method, Karbhari and Engineer [30] proposed a peel test which could alter the peel angle and vary the loading rate. In their analysis, FRP and concrete materials were assumed to be linear elastic and the stress field around the crack was calculated using elastic theory. Giurgiutiu et al. [32] and Wan et al. [31] used a modified double cantilever beam test in which FRP laminate was assumed to work as a beam. However, since the fracture of the bond commonly occurred in the concrete layer, its nonlinear fracture behaviour could not be properly evaluated by the LEFM method [7, 34]. For the NLFM method, the notched three point bending tests have been used by Qiao and Xu [6] and Coronado and Lopez [7]. Both quasi-static studies reported similar values for the mode I fracture energy of the FRP-concrete interface. However, it is found that dynamic tests are still lacked for determining the dynamic effect on fracture energy.

To bridge this gap, the current study proposed two dynamic test procedures. Dynamic direct tension test aimed to provide the tensile bond strength from quasi-static to dynamic load. The results of this test setup and the split tests conducted by Coronado and Lopez [7] were compared to gain insight into the interfacial bond strength.

Following the study of the bond strength, the dynamic behaviour of the mode I fracture energy was investigated using the notched three-point bending tests. Considering the possible material effect of concrete strength, concrete with two different strengths were used in specimen preparation. Similarly, to study the effect of different kinds of FRP, carbon and glass fibre reinforced polymer (CFRP and GFRP) were bonded to concrete substrate for the investigations of CFRP-concrete interface and GFRP-concrete interface, respectively. Finally, an application example of test results in an interface model between FRP and concrete. The interface model is used in the numerical simulations of the three-point bending tests.

The measurements of deformations of the specimens were obtained using an optical method, viz. digital image correlation (DIC). This is a data analysis procedure which uses digital cameras to record continuous deformations of a specimen, which has been prepared with a speckle pattern. The images are then processed using mathematical correlation techniques to obtain the deformation data. This method is very popular due to its ability to analyse the deformation vectors at all points on the surface covered with the speckle pattern.

5.2 Direct tension test

Coronado and Lopez [7] had used split tests to measure the tensile bond strength of concrete-epoxy interface bond. The tensile stress in this method was imposed indirectly on the specimens. It seemed that their study did not derive the true tensile strength. On the other hand, direct tension test is a direct method to obtain the tensile strength of the FRP-concrete interface bond. When the specimen was tested by ordinary clamping grip, a necked specimen was suggested by Petersson [34]. Since it had a small sectional area at the middle, the specimen could achieve a uniform tensile stress at this cross-sectional area. Accordingly, the curved edge of the dog-bone shape was designed for the specimens to achieve uniaxial tensile stress and reduce stress

concentration along the edge.

5.2.1 Sample preparation

The specimens for the direct tensile test were fabricated as shown in Figure 5.1. The concrete components were cast in the dog-bone shape and, after 28 days, they were cut into half along the transverse direction by an electric saw. The cut surfaces were cleaned using compressed air. After surface treatment, an FRP sheet was attached to the cut surface on one-half of the concrete components. The two halved components were then bonded together using epoxy. Plastic sheets which would not adhere to the epoxy were used to cover the concrete edges on both sides to produce the notches shown in Figure 5.1. The interface bond had an area of $50 \times 50 \text{ mm}^2$. The bonded specimens were then cured for 7 days. Subsequently, the front surface of the specimens was sprayed to create a randomly distributed speckle pattern for the DIC measurements. The front surface was initially sprayed with white paint as the base coat, followed by black paint to develop the speckle pattern.

Figure 5.1 Dimensions of a dog bone specimen: (a) sketch and (b) actual specimen

The specimens were divided into three groups, T1, T2 and T3, as shown in Table 5.1. Through altering concrete strength and FRP type, these specimens could be used to investigate the influence of these material parameters. Two concrete strengths were

used in fabrication of specimens T1 and T2, respectively, to study the material influence from concrete strength. The 28-day mean cylinder strengths f'_c were measured as 28.9 MPa and 45.3 MPa for the two concrete strengths.

To investigate the effect of different kinds of FRP, CFRP and GFRP sheets were bonded to the concrete substrate to produce T1 specimens with a CFRP-concrete interface and T3 specimens with a GFRP-concrete interface, respectively. The CFRP and GFRP laminates used in these specimens were fabricated using the wet lay-up method, i.e. using wet epoxy to saturate the unidirectional fabrics of carbon or glass fibres. The static material properties are given in Table 5.1. A single layer of the CFRP laminate had a thickness t_f of 0.5 mm. Its tensile strength f_{tf} and elastic modulus E_f were measured as 750 MPa and 82.7 GPa, respectively, using the testing method proposed in ASTM D3039/3039M-00 [61]. As the FRP stiffness in the fibre direction $E_f t_f$ is equal to its elastic modulus multiplied by its thickness, the stiffness value of the single-layer CFRP was 41.4 GPa·mm. Moreover, in the same method, the tensile strength and elastic modulus of the GFRP laminate were measured as 300 MPa and 28.6 GPa, respectively. The stiffness value in the fibre direction was 28.6 GPa·mm.

Table 5.1 Material parameters of the direct tension tests

Test	Loading rate (mm/s)	Concrete strength (MPa)	FRP type	FRP stiffness (GPa·mm)	Number
T1-1	0.003	28.9	CFRP	41.4	A, B, C (3)
T1-2	2	28.9	CFRP	41.4	A, B, C (3)
T1-3	20	28.9	CFRP	41.4	A, B, C (3)
T2-1	0.003	45.3	CFRP	41.4	A, B, C (3)
T2-2	2	45.3	CFRP	41.4	A, B, C (3)
T2-3	20	45.3	CFRP	41.4	A, B, C (3)
T3-1	0.003	28.9	GFRP	28.6	A, B, C (3)
T3-2	2	28.9	GFRP	28.6	A, B, C (3)
T3-3	20	28.9	GFRP	28.6	A, B, C (3)

5.2.2 Test setup

The test set up is shown as Figure 5.2. A stiff steel frame was designed to assure that, in the testing process, the frame had less deformation and maintain a pure tensile load for the interface bond. The testing specimen was placed in the frame, which was clamped by the grip of an MTS servo-hydraulic test machine. To maintain a stable test process, the specimen was loaded with displacement control.

Figure 5.2 Test setup of the direct tension test

The applied force was measured by the load cell in the test machine. The deformation and displacement of a testing specimen were recorded by DIC, which comprised a digit camera and a computer. A part of ruler was glued on the testing specimen to provide a reference for the measurement of displacement. The camera was placed perpendicular to the specimen to take photos for deformation and displacements at regular intervals. The images recorded by the camera were stored in the computer and were analysed by a digital image correlation (DIC) software, named GOM correlate 2018. Accordingly, the load-time history and displacement-time history were documented in these tests. To study the dynamic rate effect on the fracture behaviour of FRP-concrete interface, the direct tension tests were conducted from 0.003 mm/s to 20 mm/s.

The material parameters and loading rates of the direct tension tests are shown in Table 5.1. The nomenclature of these specimens contained information on the material features and loading rate. At the beginning, T1, T2 and T3 represented control group with CFRP laminate, specimens with a higher concrete strength and specimens with GFRP laminate, followed by the number 1, 2 and 3 representing loading rates of 0.003 mm/s, 2 mm/s and 20 mm/s, respectively. According to the loading rate, these tests were classified as quasi-static (0.003 mm/s) and dynamic tests (2 mm/s and 20 mm/s). The results of quasi-static tests served as the baseline for the analysis of dynamic tests. Three samples (A, B, C) were tested for each configuration to evaluate repeatability.

5.2.3 Results and discussion

5.2.3.1 Failure modes and test results

Typical fracture patterns of the direct tension test are shown in Figure 5.3, including fracture in concrete and fracture in both concrete and epoxy. The cracked two half parts are shown in Figures 5.3(b) and (c). Most specimens in the direct tension tests failed by fracture in the concrete, in which a layer of mortar and sands were peeled together with the FRP sheet. It was due to that the epoxy has a tensile strength of 50 MPa which is much higher than that of concrete material. The crack zone of the concrete layer was close to the bonded interface with a depth in the range of 0.5 mm to 11.9 mm. Considering the maximum aggregate size of 10 mm used in these specimens, the crack was varied between the cement and the first layer aggregates near to the bonded interface. Such crack zone has been usually found in direct tensile tests of pure concrete material [34], which meant that, in the test process, uniform deformation gradually became local crack and limited crack zone, followed by complete separation at this zone.

(a) bond failure at different layers

(b) fracture in concrete

(c) fracture in concrete and epoxy

Figure 5.3 Failure modes of direct tension tests

Six specimens showed a second failure mode in which fracture existed in both concrete and epoxy, as shown in Figure 5.3(c). The fracture in concrete layer was same with the first failure mode that some mortar and aggregates attached on the FRP sheet. However, the fracture failure in epoxy should be attributed to the condition that

some rigid aggregates at local bond area had a higher strength than the epoxy at the same position. In these local parts, the strength of epoxy prepared via wet lay-up method might be lower than the expected strength and even weaker than the rigid aggregates. When the tensile bond strengths of these specimens were reviewed, it was not found that this failure mode has any significant effect on the test results.

Since the loading rate set in the test machine was not actual separation velocity of the bond during the fracture process, the separation rate was re-evaluated to represent the crack velocity and to conduct the analysis of dynamic enhancing effect on the bond strength. The separation rate of direct tension test was derived through separation-time history recorded by DIC, as shown in Figure 5.4. It was observed that the gradient of the separation-time curve was not a constant, which was relatively lower in the elastic range and increased significantly in the softening stage. Therefore, an average separation rate was used to represent the dynamic properties of each test in the dynamic analysis.

Figure 5.4 Separation-time history of specimen T1-2B

Table 5.2 presents the results obtained from the direct tension tests. Three kinds of specimens were tested to investigate the material effect from concrete strength and different FRP material. Specimens T1 were a control group with a CFRP-concrete

interface bond. A higher concrete strength and a GFRP-concrete interface bond were used in specimens T2 and T3, respectively. The results of quasi-static tests were analysed and are presented in Section 5.2.3.2. The DIF values in Table 5.2 were calculated as the ratio of each dynamic test result to the quasi-static strength based on a best-fitting equation in Section 5.2.3.2.

Table 5.2 Summary of all test results of the direct tension test

Test	Loading rate (mm/s)	Separation rate (mm/s)	Ultimate load (kN)	Bond strength (MPa)	DIF for bond strength	Failure mode
T1-1A	0.003	0.001	5.48	2.19	0.90	C
T1-1B	0.003	0.001	5.63	2.25	0.93	C
T1-1C	0.003	0.002	7.21	2.88	1.19	C
T1-2A	2	0.518	6.21	2.49	1.03	C
T1-2B	2	0.989	5.89	2.36	0.98	C
T1-2C	2	0.679	5.97	2.39	0.99	CE
T1-3A	20	3.42	7.33	2.93	1.21	C
T1-3B	20	3.75	8.21	3.28	1.36	C
T1-3C	20	9.51	10.04	4.02	1.66	C
T2-1A	0.003	0.001	8.42	3.37	1.03	C
T2-1B	0.003	0.002	7.11	2.85	0.87	C
T2-1C	0.003	0.001	9.02	3.61	1.10	CE
T2-2A	2	0.313	9.13	3.65	1.12	C
T2-2B	2	0.562	8.69	3.48	1.06	C
T2-2C	2	0.580	11.05	4.42	1.35	C
T2-3A	20	4.11	11.02	4.41	1.35	C
T2-3B	20	4.69	10.30	4.12	1.26	C
T2-3C	20	5.52	11.65	4.66	1.43	CE
T3-1A	0.003	0.001	5.78	2.31	0.95	CE
T3-1B	0.003	0.001	5.66	2.26	0.93	C
T3-1C	0.003	0.001	5.90	2.36	0.98	CE
T3-2A	2	0.649	7.12	2.85	1.18	C
T3-2B	2	0.515	6.23	2.49	1.03	C
T3-2C	2	0.505	6.79	2.72	1.12	C
T3-3A	20	5.31	7.23	2.89	1.19	C
T3-3B	20	7.10	7.58	3.03	1.25	C
T3-3C	20	3.01	7.87	3.15	1.30	CE

Note: C, fracture in concrete; CE, fracture in concrete and epoxy.

After the comparison among specimens T1, T2 and T3 in Table 5.2, it was not found

that the material parameters were related to the different failure modes. Both failure modes could happen in the three kinds of specimens. The results of quasi-static tests were analysed and regressed. A best-fitting equation was proposed later to predict the bond strength under quasi-static scenario.

5.2.3.2 Quasi-static tests

The quasi-static test results were evaluated here. Figure 5.5 presents the load-time history of specimen T2-1B. At the beginning, due to the relative slip between the specimen and the grip, the gradient of the curve was a little lower. After that, the load-time history could be divided into three ranges: elastic range at point A, nonlinear pre-peak range at point B and post-peak range at points C and D. In the first range, elastic behaviour was characterized by uniform deformation of the interface bond, as shown in Figure 5.6(a). Followed by the second range, the damage initiation around the bond area reduced the stiffness and caused the nonlinear pre-peak behaviour. Some minor cracks developed since this stage, as shown in Figure 5.6(b). After the peak load, the significant growth of crack around the bond area became visible and greatly reduced the stiffness and the load capacity. The fracture process is shown in Figure 5.b(b). As recorded by the DIC, the crack zone was limited in the first layer of aggregates close to the interface bond.

Figure 5.5 Load-time history of specimen T2-1B

(a)

(b)

Figure 5.6 DIC strain plots for the tests of T2-1B: (a) general plot and (b) fracture process

Figure 5.7 shows the quasi-static tensile strength plotted against the concrete cylinder

strength, including specimens T1, T2, T3 and the split test specimens (CEI-1) in Coronado's study [7]. As their aim was identical to the current study, their test results were included here as a comparison with the present direct tension tests.

Figure 5.7 Quasi-static tensile strength against concrete cylinder strength

(T1: control group, T2: a higher concrete strength, T3: GFRP-concrete interface)

Based on the analysis in Figure 5.7, the quasi-static bond strength σ_{lst} of the interface was positively correlated to concrete cylinder strength f'_c and could be predicted using the author's proposed equation as follows.

$$\sigma_{lst} = 0.257(f'_c)^{2/3} \quad (5.1)$$

The prediction line is provided in Figure 5.7. The prediction-to-test ratio of 1.02 and coefficient of variation (CoV) of 0.09 present that the equation can properly predict the quasi-static tensile strength of FRP-concrete interface. As shown in Figure 5.7, the consistent data of the tensile strength in both direct tension test and split tests indicates that both methods can provide reliable tensile strength for the interface bond.

5.2.3.3 Dynamic tests

Different from the stable loading process in quasi-static tests, natural period and inertia force of the test setup usually influence the test results under dynamic load. Once the nature period of the test setup is close to the testing duration, the load cell

may be bothered by the interaction between machine and specimen, and fail to track the applied force accurately [65]. Similarly, inertia force is always measured by the load cell together with material response in dynamic tests. For analysing material dynamic behaviour, inertia force needs to be deducted from general load. Therefore, to validate the dynamic tension tests, the natural period and the inertia force were examined.

Figure 5.8 shows the load-time history of specimen T1-3C after its fracture. The nature period of the test setup estimated from the oscillation curve was about 0.004 s, which was much shorter than the testing duration of about 0.04 s for the specimen under a loading rate of 20 mm/s. Therefore, it was believed that the load cell was not disturbed by the nature period of the test setup.

Figure 5.8 Vibration of testing machine after specimen fracture (specimen T1-3C, loading rate 20 mm/s)

To evaluate inertia effect in the dynamic tests, the load-time curves of two selected specimens are shown in Figure 5.9. Specimens T1-2B and T1-3C were tested under loading rates of 2 mm/s and 20 mm/s, respectively. The general load recorded by the load cell included both the material response and the inertia force. The inertia force-time histories were derived by multiplying the acceleration-time history from DIC by the mass of the testing specimen and the steel frame. As shown in Figure 5.9(a), the

inertia force was minor in the testing process at a rate of 2 mm/s. In Figure 5.9(b), the inertia force was also minor in pre-peak range but became significant in the post-peak range. As the loading rate of 20 mm/s was not an extreme velocity, the inertia force was not great. For the dynamic analysis, the inertia force was deducted from the general load to derive actual load, as shown in Figure 5.9.

Figure 5.9 Load-time histories of specimens (a) T1-2B and (b) T1-3C

Figure 5.10 presents the stress-separation curves of specimens cracked with separation rates varying between 0.001 mm/s and 9.51 mm/s. Bond stress was derived from the actual load over the interfacial bonding area of 2500 mm². All the three specimens showed a sudden jump at the beginning. This behaviour was identical to the tensile response of concrete material, which indicated a high stiffness for the interface bond. With the increase of separation rates, dynamic enhancing effect on bond strength was noticed for specimens T1-2B and T1-3C. Considering the fracture path in concrete layer, the reason of dynamic enhancing effect should be similar to that of concrete material. There was free water inside the concrete material which had viscosity. The viscosity could improve the dynamic tensile strength [81]. After the peak bond stresses, the curves presented the softening behaviour with the growth of separation. However, like many other quasi-brittle materials, the FRP-concrete interface bond lost its capacity quickly after reaching the bond strength, the softening

range was not measured in a stable state. The softening range was found in the direct tension test, but it would not be used in the analysis due to its low accuracy in the measurement.

Figure 5.10 Stress-separation curves of specimens under different separation rates

The dynamic enhancing effect is presented by the bond strength against the separation rate, as shown in Figure 5.11. The bond strength in three kinds of specimens clearly presented increase trend with the growth of separation rate. The specimens T1 and T3 had same concrete strength but different FRP materials. There was no obvious difference on the bond strength between the two specimens. Only one specimen T1-3C which had a significant higher dynamic bond strength of 4.02 MPa at the separation rate of 9.51 mm/s. This result might be a scatter when compared with its repeating tests results of 2.93 MPa and 3.28 MPa. Compared with the specimens T1 and T3, specimens T2 had a higher concrete compression strength and present higher bond strength in all the separation rates. Accordingly, the material effect due to concrete strength was dominant and significant but the effect of different types of FRP was relatively negligible. This was expected since the failure mainly took place in the concrete rather than within the FRP laminate.

Figure 5.11 Rate effect comparison on bond strength

Based on the activation energy theory, Bazant et al. [79] derived an inverse sinh function to indicate the relation between concrete opening rate and concrete tensile strength. As the dynamic response of the FRP-concrete interface bond is highly dependent on the concrete material, an analogous inverse sinh function is proposed in this study to predict the dynamic bond strength σ_{ld} as follows.

$$\sigma_{ld} = \sigma_{lst} \left(1 + a_l \cdot a \sinh\left(\frac{\dot{s}_l}{\dot{s}_{l0}}\right) \right) \quad (5.2a)$$

$$a_l = 0.0082 \left(\frac{f'_c}{f_0} \right) + 0.0008 \quad (5.2b)$$

where coefficient a_l is a dynamic coefficient and its value is determined through regression of test results in this chapter, coefficient f_0 is defined as 10.0 MPa and the reference separation rate $\dot{s}_{l0}$ is fitted as 0.004 mm/s. The effect of concrete strength is considered in Eq. (5.2b). The prediction lines are shown in Figure 5.11, with a prediction-to-test ratio of 1.01 and a CoV of 0.11.

Considering the close relation between the interface bond and concrete strength, the

strain rates in the critical areas of concrete in these tests were assessed, which varied from $2\text{e-}6 \text{ s}^{-1}$ to 0.12 s^{-1} . According to prediction equation in CEB code (2010), corresponding dynamic tensile strength of concrete material varied from 2.82 to 4.55 MPa, as shown in Figure 5.12. Compared with the predicted dynamic concrete strength in these tests, the values of corresponding interface bond strength are slightly lower. In this figure, it could be found that concrete tensile strength takes a dominant effect on the interface bond strength in dynamic scenarios.

Figure 5.12 Comparison between predicted concrete tensile strength and the interfacial bond strength

5.3 Three-point bending test

The notched three-point bending tests were commonly used to study the mode I fracture energy of concrete material [6]. RILEM(1985) [82] also recommended the notched three-point bending tests to determine the fracture energy. Hereby, the notched three-point bending test was adopted here to determine mode I fracture energy G_{Id} of FRP-concrete interface.

5.3.1 Specimen preparation

The geometry of the beam specimens is $100 \text{ mm} \times 100 \text{ mm} \times 800 \text{ mm}$, as shown in Figure 5.13. The geometry of the specimens had been used in the investigation of fracture energy of plain concrete in the report of Hillerborg [35]. The preparation

procedures of these beam specimens were identical to those of specimens for direct tension tests. The concrete components were cast in rectangular shape with dimensions of $100 \text{ mm} \times 100 \text{ mm} \times 400 \text{ mm}$. After 28 days, the concrete surfaces for the interface bond were ground to reveal the aggregates and then they were cleaned using compressed air. After surface preparation, FRP laminates were bonded to the cleaned surfaces. The two concrete components were bonded together using epoxy and then were left to cure for 7 days. Same with the specimens reported by Qiao and Xu [6], the notch depth was 50 mm and the interface bond area was $100 \times 50 \text{ mm}^2$. Similar with the specimens in direct tension tests, the front surface around the bond area was treated to be a random distributed speckle pattern for the measurement by digit camera.

(a)

(b)

Figure 5.13 A specimen of notched three point bending test: (a) specimen sketch, (b) actual specimens

Based on different material features, the specimens in the three-points bending beam tests were divided into three groups, B1, B2 and B3, as shown in Table 5.3. Group B1 was a control group has a 28-day concrete strength of 29.3 MPa and a layer of

CFRP laminate. To study the effect from concrete strength, Group B2 had a higher 28-day concrete strength of 52.7 MPa. The GFRP laminate was used for group B3 to investigate the effect of different FRP materials. The material parameters of CFRP and GFRP laminates used in these specimens were same with those in the specimens of direct tension tests.

Table 5.3 Material parameters of the three-point bending beam tests

Test	Loading rate (mm/s)	Concrete strength (MPa)	FRP material	FRP stiffness (GPa·mm)	Number
B1-1	0.02	29.3	CFRP	41.4	A, B, C (3)
B1-2	2	29.3	CFRP	41.4	A, B, C (3)
B1-3	20	29.3	CFRP	41.4	A, B, C (3)
B2-1	0.02	52.7	CFRP	41.4	A, B, C (3)
B2-2	2	52.7	CFRP	41.4	A, B, C (3)
B2-3	20	52.7	CFRP	41.4	A, B, C (3)
B3-1	0.02	29.3	GFRP	28.6	A, B, C (3)
B3-2	2	29.3	GFRP	28.6	A, B, C (3)
B3-3	20	29.3	GFRP	28.6	A, B, C (3)

5.3.2 Test setup

The test setup is shown as Figure 5.14. In the tests, the weight of the beam contributed to the additional vertical load, which could accelerate the crack process and obtain a unreliable result [34]. To eliminate this effect, as shown in Figure 5.13(a), a 200 mm long section of the specimen was left to be a cantilever on each side of the roller support. The 200 mm long sections on both sides of the supports balanced the weight of 400 mm length of central span.

(a)

(b)

Figure 5.14 Setup of notched three point bending test: (a) actual test and (b) view path of digit camera

During the tests, the beam specimen was placed on the supporting rollers and was subjected to the vertical loading at the mid span. Same as the direct tension tests, displacement control was used in the loading to maintain a stable fracture process. The applied force was measured by the load cell of the testing machine. The fracture process and the displacements were recorded by the DIC. Since the space between the testing beam and the supporting column of the testing machine was quite narrow, the camera for recording loading point displacement could not be placed behind the measuring zone. Instead, a mirror was fixed at a 45° angle behind the testing specimen and then the sight of camera was placed at 45° angle to the mirror, as shown in Figure 5.14(b). Photos recorded by the camera were also analysed by the DIC software, GOM correlate 2018. Accordingly, the load-time history and displacement-time history were recorded in the test process. The three-point bending tests were conducted from 0.02 mm/s to 20 mm/s to investigate the dynamic rate effect on the fracture behaviour of FRP-concrete interface.

Table 5.3 shows the loading rates and characteristic parameters of the notched three-point bending tests. Similar to those of direct tension tests, B1, B2 and B3 in the nomenclature represent control group, specimens with a higher concrete strength and specimens with GFRP laminate, followed by the number 1, 2 and 3 for loading rates of 0.02 mm/s, 2 mm/s and 20 mm/s, respectively. Three samples (A, B, C) were prepared for each testing configuration to evaluate repeatability.

5.3.3 Results and discussion

5.3.3.1 Determination of interface fracture energy

Hillerborg [35] and RILEM [82] proposed a work-to-fracture method to determine the fracture energy of concrete material. Such method was also used in the current study for the FRP-concrete interface. According to the proposed method, the fracture energy means the external work required to create unit area of fracture and its function is given as follows.

$$G_{Id} = \frac{W_F}{t_b(h-h_n)} \quad (5.3)$$

where W_F is external work, t_b is specimen thickness, h and h_n are height of the beam and the notch depth, respectively. Therefore, the value of $t_b(h-h_n)$ is the area of the interface bond.

Due to the counter weight offered by the cantilever on each side of roller support, the load-displacement curve has a very long tail before reaching ultimate failure [34], as shown in Figure 5.15. Therefore, the tests were commonly stopped before complete failure of the specimen occurred.

Figure 5.15 Typical load-displacement curve for the three-point bending test

The work to create the fracture, W_F , is the area bounded by the load-displacement curve and the line B'T, as shown in Figure 5.15. According to investigations by Petersson [34], Qiao and Xu [6] and Coronado and Lopez [7], the value of W_F can be calculated using Eq. (5.4).

$$W_F = W_m + \frac{2A}{u_C - u_B} \quad (5.4)$$

where u_B and u_C are the displacements corresponding to points B and C, respectively. At point T, the specimen is assumed to have fully failed. However, the tests are commonly stopped at point C before complete failure occurs. A line BC was drawn parallel to the deflection axis and the area enclosed in BMC is represented by W_m in Eq. (5.4). After the peak load, the load value P and corresponding displacement u are found following the law in Eq. (5.5) [34].

$$P - P_C = A \left[\frac{1}{(u - u_B)^2} - \frac{1}{(u_C - u_B)^2} \right] \quad (5.5)$$

where P_C is the load value at point C. Coefficient A is the gradient of the plot $(P - P_C)$ in Newton versus $(u - u_B)^{-2} - (u_C - u_B)^{-2}$ in mm^{-2} , which is determined for every specimen using a least-squares straight line.

5.3.3.2 Failure modes and test results

Typical fracture patterns of the interface bond after the bending tests are shown in Figure 5.16. The fracture behaviours were similar to those in direct tension tests, as most specimens failed by cracks within concrete, as shown in Figure 5.16(a). The crack zone of the concrete substrate had a depth in the range of 0.5 mm to 6.6 mm, which meant that the cracks propagated in the first layer of aggregates. Moreover, three specimens failed by fracture in concrete and epoxy, as shown in Figure 5.16(b). As suggested by Coronado and Lopez [7], due to local crack oscillations around the bond line, some specimens failed by local cracks in concrete and epoxy. When the peak load and fracture energy of these specimens were reviewed, it was found that different failure modes did not have any significant effect on the result data.

Figure 5.16 Failure modes of the three-points bending tests

Similar to the direction tension tests, loading rates in the three-point bending tests could not directly indicate the fracture velocity of the interface bond. According to the interface crack process recorded by DIC system, the fracture velocity of a testing specimen is determined as shown in Figure 5.17. Five pairs of points at both sides of the interface bond are selected from the notch end to the loading position in Figure 5.17(a) and the separation-time histories in each pair of points are shown in Figure 5.17(b). Since the crack velocities of the five positions were different, an average

separation rate among the five positions was calculated to represent the dynamic properties of each test in the dynamic analysis.

(a) crack positions in DIC analysis

(b) separation-time history

Figure 5.17 Crack behaviour of specimen B1-2A

Table 5.4 presents the results obtained from the three-point bending tests, including three kinds of specimens with different material parameters. Specimens B1 were a control group with a CFRP-concrete interface bond. Followed by specimens B2 and B3, a higher concrete strength and a GFRP-concrete interface bond were used in the

fabrication, respectively. The mean value of the three quasi-static tests was taken as the final result under quasi-static scenario. The DIF was calculated as the ratio of each dynamic test results divided by the mean value of corresponding quasi-static test results.

Table 5.4 Summary of test results of the notched three-point bending tests

Test	Loading rate (mm/s)	Separation rate (mm/s)	Peak load (kN)	G_{ld} (N/mm)	DIF of fracture energy	Failure mode
B1-1A	0.02	0.008	2.08	0.109	0.93	C
B1-1B	0.02	0.007	1.98	0.121	1.04	C
B1-1C	0.02	0.008	2.21	0.120	1.03	C
B1-2A	2	0.818	2.75	0.127	1.09	C
B1-2B	2	0.684	1.87	0.101	0.87	C
B1-2C	2	0.748	2.36	0.125	1.07	C
B1-3A	20	13.4	3.11	0.230	1.97	C
B1-3B	20	15.2	2.78	0.204	1.75	C
B1-3C	20	12.8	2.97	0.219	1.88	C
B2-1A	0.02	0.009	2.17	0.080	0.72	C
B2-1B	0.02	0.007	2.44	0.122	1.09	C
B2-1C	0.02	0.007	2.30	0.133	1.19	C
B2-2A	2	0.595	3.17	0.149	1.33	C
B2-2B	2	0.589	2.26	0.124	1.11	CE
B2-2C ¹	2	0.287	2.97	0.228	2.04	C
B2-3A	20	6.86	3.44	0.246	2.20	C
B2-3B	20	14.8	3.00	0.194	1.74	CE
B2-3C	20	14.8	3.13	0.178	1.59	C
B3-1A	0.02	0.006	1.86	0.114	0.93	C
B3-1B	0.02	0.008	2.18	0.120	0.98	C
B3-1C	0.02	0.007	2.29	0.135	1.10	C
B3-2A	2	0.739	2.91	0.155	1.26	C
B3-2B	2	0.723	3.12	0.132	1.07	C
B3-2C	2	0.818	2.48	0.161	1.31	C
B3-3A	20	15.4	2.95	0.182	1.48	C
B3-3B	20	13.8	3.42	0.233	1.89	C
B3-3C	20	16.5	3.77	0.235	1.91	CE

Note: C, fracture in concrete; CE, fracture in concrete and epoxy

¹the fracture energy of specimen B2-2C was significantly higher than its two repeated results, due to multiple cracks forming around the principal crack and consuming additional energy. Therefore, B2-2C was not included in the analysis of the dynamic fracture energy due to its abnormal fracture behaviour.

5.3.3.3 Static tests

For the analysis of quasi-static tests, the fracture energy of plain concrete was introduced here as reference. The test results from Qiao and Xu [6] and Coronado and Lopez [7] were also compared with the current study to gain insight for this subject. A comparison of fracture energy between FRP-concrete interfaces and plain concrete is presented in Table 5.5.

Table 5.5 Fracture energy (N/mm) under quasi-static load

Specimen	Plain concrete ^a	Qiao[6] CFRP ^b	Qiao[6] GFRP ^c	Coronado [7]	B1-1	B2-1	B3-1
1	0.109	0.115	0.123	0.130	0.109	0.80	0.114
2	0.113	0.087	0.084	0.129	0.121	0.122	0.120
3		0.094	0.107	0.126	0.120	0.133	0.135
4		0.092	0.126				
5		0.097	0.113				
6		0.096	0.092				
Ave	0.111	0.097	0.109	0.128	0.117	0.112	0.123
CoV		0.099	0.157	0.020	0.057	0.250	0.011

^aFrom Hillerborg (1985) [35].

^bTests of CFRP-concrete interface bond reported by Qiao and Xu [6].

^cTests of GFRP-concrete interface bond reported by Qiao and Xu [6].

The test results in this study showed that the fracture energy of CFRP-concrete interface (B1-1 and B2-1) was close to that of plain concrete. The fracture energy of GFRP-concrete interface (B3-1) is slightly higher than that of CFRP-concrete interface. It should benefit from the larger deformation capacity of GFRP laminate, which make the specimens withstand larger cracks than those with CFRP laminate. The higher fracture energy of GFRP-concrete interface was also reported by Qiao and Xu [6]. Coronado and Lopez [7] presented the highest average fracture energy in Table 5.5. This behaviour might be attributed to a smaller notch-to-depth ratio used in their specimens. Generally, the difference among the results from Qiao and Xu [6], Coronado and Lopez [7] and the current study was not significant. Considering the

different sizes used in these studies, closed fracture energy values under quasi-static load proved that the fracture energy was not sensitive to variation of specimen's size. These results were still within the range of fracture energy reported for plain concrete, viz 0.070 to 0.140 N/mm [34], which implied that in quasi-static tests, the fracture behaviour of the FRP-concrete interface was similar to that of plain concrete.

The load-displacement curve of a quasi-static test is shown in Figure 5.18. It has a linear ascending range before the peak, followed by softening behaviour. Stages I, II, III and IV are marked in the load-displacement curve and corresponding fracture plot are shown in Figure 5.19. In the stage I, elastic behaviour was characterized by uniform deformation of the interface bond. The initiation of internal cracks was negligible at this stage [7]. Followed by the stage II at the peak load, micro crack developed to macro crack and damage localization started from the crack tip. After the peak load, the significant growth of crack around the bond area became visible at the Stage III. The macro crack greatly reduced the stiffness and the load capacity of the specimen. As recorded by the DIC, a principal crack propagated from the notch towards the mid-span loading point in the plot of stage IV.

Figure 5.18 Load-displacement curve of specimen B2-1C (loading rate: 0.02 mm/s)

Figure 5.19 DIC plots for the loading centre of B2-1C

5.3.3.4 Dynamic tests

Dynamic tests were usually affected by the interaction between specimen and test machine. For the validation of the dynamic tests, the natural period of the test setup and the inertia force in the process were evaluated. As the bending tests were conducted on the same machine with the direct tension tests, the natural period of the testing system was same as 0.004 s. The testing durations under loading rates of 0.02 mm/s, 2 mm/s and 20 mm/s were 100 s, 1.0 s and 0.1 s, respectively. All the durations were much longer than the natural period and made the load cell to work in a reliable

condition.

The inertia effect was investigated for tests under a loading rate of 20 mm/s. Same with those direct tension tests, the acceleration-time histories were measured by the DIC and then they were multiplied by the mass of specimen to generate the inertia force-time histories. As shown in Figure 5.20, the general load measured by the load cell contained the specimen's response and the inertia force. The inertia force was tracked and was deducted from the general load for the actual load. Compared with the general load, the inertia force was not significant.

Figure 5.20 Load-time histories of specimen B1-3C (loading rate 20 mm/s)

After the validation of dynamic tests, the fracture energies were calculated and analysed. As shown in Figure 5.21, a dynamic increase trend is found for fracture energy. The reason for the dynamic enhancing effect should be the same as that explained for the dynamic tensile tests. Free water in the void and porous structure of concrete material had viscosity which could improve dynamic bond strength and fracture energy. In the separation rate range between 0.006 mm/s and 0.818 mm/s, specimens B2 and B3, which use a higher concrete strength and GFRP laminate, respectively, showed slightly higher fracture energies than that of the control group. However, at a separation rate of about 15 mm/s, such an increasing trend for specimens B2 and B3 was masked by the variation of the results under dynamic loads.

As the increasing trend was minor and the variation in the repeating tests was significant, there was no distinctive effect from the material parameters including concrete strength and type of FRP materials.

Figure 5.21 Fracture energy versus separation rate

To predict the dynamic enhancing effect, a dynamic increase factor (DIF) equation for the fracture energy was derived from regression analysis based on the test data. The DIF for the dynamic fracture energy is given as follows.

$$DIF_{GI} = \frac{G_{Id}}{G_{Ist}} = 0.26 \operatorname{arc} \sinh\left(\frac{\dot{s}}{1.53}\right) + 1.0 \quad (5.6)$$

where G_{Ist} is the quasi-static fracture energy. The prediction line is shown in Figure 22, with a prediction-to-test ratio of 1.00 and a CoV of 0.14. Since the material parameters did not influence the dynamic fracture energy significantly, they were not considered in the DIF equation.

Figure 5.22 DIF of fracture energy against separation rate

5.4 Application of test results

This section provides application examples of interface bond properties in finite element analysis (FEA) models of the three-point bending tests with specimens B1. The interface bond properties obtained in Sections 5.2 and 5.3 were used to define the bond stress-separation law of interface element in FEA. The interface bond properties derived from test results were validated through these numerical simulations.

5.4.1 Bilinear stress-separation law

The mode I fracture behaviour of the FRP-concrete interface has been usually modelled by the bilinear stress-separation law [36, 38, 49], which describes the fracture process with linear ascending and linear softening ranges. The constitutive equations are shown in Eq. (5.7). In the stress-separation law, the tensile bond strength and the fracture energy are dominant properties, which have been successfully obtained from the direct tension tests and the notched three-point bending tests in this study, respectively.

$$\sigma_I = \begin{cases} K_I \cdot s_I, 0 \leq s_I \leq s_{Id} \\ \sigma_{Id} \frac{s_{If} - s_I}{s_{If} - s_{Id}}, s_{Id} < s \leq s_{If} \end{cases} \quad (5.7)$$

where the separation of the dynamic bond strength is given by $s_{Id} = \frac{\sigma_{Id}}{K_I}$, initial stiffness K_I is defined as $K_I = \frac{K_e K_c}{K_e + K_c}$ [3], epoxy stiffness K_e and concrete stiffness K_c are given by $K_e = \frac{E_e}{t_e}$ and $K_c = \frac{E_c}{t_c}$, respectively, E_e and t_e are elastic modulus and thickness of the epoxy, respectively, E_c and t_c are elastic modulus and thickness of the concrete, respectively, ultimate separation s_{If} is given by $s_{If} = \frac{2G_{Id}}{\sigma_{Id}}$.

The dynamic enhancing effect on the tensile bond strength and the fracture energy is determined by Eq. (5.2) and Eq. (5.6). With the determined dynamic properties and Eq. (5.7), the predicted bilinear stress-separation curves for specimen B1 under different separation rates are shown in Figure 5.23. The test results and dynamic equations in this study can be used in the definition of interfacial response for FRP strengthened RC structures under dynamic loads.

Figure 5.23 Stress-separation curves for specimen B1 under different separation rates

5.4.2 FE model

LSDYNA is a software commonly used for explicit dynamic finite element (FE) simulations and is used in this study to model the three-point bending tests. The

bilinear stress-separation law introduced in Section 5.4.1 was programmed into a user-defined interface element in LSDYNA. To improve the computational efficiency, the test was simulated by a plane stress model and the stress behaviour along the thickness was assumed to be the same. The FE model was a layer of solid elements, as shown in Figure 5.24. A mesh size of 5 mm was used and the thickness values of FRP and interface elements were 1 mm. The concrete material was modelled using the CSCM concrete model [83], which was a common plastic concrete model for dynamic FE simulations. It was popular due to the ability of automatically generating model parameters with an input concrete strength. Specimen B1 had a concrete strength of 29.3 MPa. DIF was defined for concrete elements due to their sensitivity to dynamic loading. Based on automatic generation, default values were defined for other model parameters. FRP laminate was modelled by an elastic material with an input elastic modulus of 41.4 GPa. Based on Eqs. (5.1, 5.2) and Table 5, the bond properties of the interface elements were defined as follows: $f_{bst}=2.5$ MPa, $a_l=0.025$, $K_l=1000$ MPa/mm and $G_{Fst}=0.12$ N/mm. Loading rates in the FE simulations were the same as the test setup in Table 5.3.

Figure 5.24 FE model of the three-point bending test

5.4.3 Simulation results

The load-displacement curves derived from the FE simulations and the comparison with test results are shown in Figure 5.25. It could be seen that the curves predicted by FE simulations matched the test results well. Compared with the test results, the peak loads of FE predictions show difference below 5% and the fracture energy of FE predictions present difference between 3% and 14%. Through this comparison, the interface properties derived from the direct tension tests and the three-point bending tests are validated.

Figure 5.25 Load-displacement curves of three-point bending tests with specimen B1

The strain contours of the FE simulation of specimen B1-1B are shown in Figure 5.26.

At the moment of maximum load (stage I), the plastic strain is appeared above the notch, as shown in Figure 5.26(a). At the loading stage II (displacement of 1.0 mm), as shown in Figure 5.26(b), the damage plot in interface elements and nearby concrete elements is clear and the fracture extends close to the loading point.

(a) max-principal strain at stage I (maximum load)

(b) max-principal strain at stage II (displacement of 1.0 mm)

Figure 5.26 FE simulation of B1-1B

5.5 Conclusion

Two test setups were proposed in this study to characterise the mode I fracture behaviour of FRP-concrete interface bond. Conclusions summarized from this study are as follows.

Both the direct tension tests in this study and split tests by other researchers provided reasonable quasi-static bond strength, implying that both test methods could be used

to measure this property. A prediction equation for the quasi-static bond strength was given for engineering application. Compared with the test results, the author's proposed equations achieved good agreement. In the analysis of material effect from concrete strength, it was found that a higher concrete strength could improve the bond strength in both quasi-static and dynamic scenarios. However, it seemed that the use of CFRP and GFRP has no significant difference on the interface bond strength, as fracture was occurred in the concrete layer.

In the analysis of the notched three-point bending test results, it was found that the dynamic fracture energy increased significantly with increasing separation rate. A DIF equation was provided to predict the dynamic enhancement of fracture energy, which could be used for the analysis of dynamic fracture behaviour. Specimens prepared with a higher concrete strength and a GFRP laminate had slightly higher fracture energies at lower separation rates. However, such an increase was negligible when compared with the experimental scatter in these tests. Under a similar separation rate, the specimens using different concrete strengths and different FRP materials presented similar fracture energies.

The bond strength and the fracture energy for the specimens under quasi-static loads were similar to those of plain concrete. Such findings were also confirmed in the literature. In dynamic tests, the dynamic enhancing effect on mode I properties of the interface bond depended on dynamic response from the concrete material. The equations of bond strength and fracture energy proposed in this study were also used to define the interfacial elements of FE models of the three-point bending tests under dynamic loads. The comparison between the FE simulations and the test results confirmed the accuracy of the proposed equations.

Chapter 6 A coupled dynamic cohesive zone model for FRP-concrete mixed-mode separation

6.1 Introduction

Cohesive zone model (CZM) is a simple and time efficiency tool for simulating fracture issues of different materials [52, 53]. As mentioned in Section 2.5.3, CZM describes the traction-separation relationship in the fracture zone until the fracture occurs and new free surfaces appear [54]. Therefore, its performance is quite suitable to model the bond failure between materials. With growing attention to the fibre reinforced polymer (FRP) strengthening effect for reinforced concrete (RC) structures, evaluation of the bond interface has been found in both FRP strengthened structures and FRP-concrete debonding tests [1, 3-6, 10, 22, 45, 50, 84]. Due to the concern of FRP strengthened concrete structures failing in shear slip, Chen and Teng [4], Lu et al [3], Dai et al [5] and many other researchers [1, 10, 22] extensively investigated shear failure under quasi-static and dynamic loadings. In addition, considering possible delamination failure between FRP and concrete, Qiao and Xu [6] experimentally studied mode I fracture of the bond joint. However, mixed mode failure of the bond zone is commonly noticed in practice [49], especially for those structures with inclined cracks. Accordingly, Wan et al. [31], Ouyang and Wan [55] and Pan and Leung [84] evaluated such mixed-mode debonding based on experiment methods. To study FRP-concrete debonding behaviours, CZM is used to model the bond between FRP and concrete in numerical simulations [10, 36, 50] and analytical methods [38, 39, 49].

In terms of different crack modes, CZMs are broadly classified into uncoupled models for pure mode fracture and coupled models including features of mode I and mode II for mixed-mode fracture [53, 85]. Due to the advantage of being applicable

to both pure mode and mixed-mode fractures, coupled CZM models are quite popular in engineering applications. Several shapes have been proposed in literature, such as bilinear and trapezoid models, exponential models and viscoplastic models. Bilinear and trapezoid models are frequently used due to their simplicity [86, 87], which assume that both ascending and softening ranges are linear. Considering rate-dependent response of adhesive joint, Marzi et al. [52] proposed a trapezoid CZM with dynamic enhancing formulations. To make simulations more close to realistic scenarios, May et al. [88] further developed the model of Marzi et al. to be applicable for rate variation during the crack process. However, exponential models present nonlinear traction-separation behaviour to pursue more accurate failure processes than bilinear and trapezoid models [53]. To achieve this aim, Xu and Needleman [56], van den Bosch et al. [89], McGarry et al. [85] and Dimitri et al. [53] developed their non-potential-based or potential-based CZMs. Viscoplastic models focus on physical description of dynamic behaviour by including rate effects into material parameters. Corigliano and Racci [57] and Lorefice et al. [58] gave their models for composite and concrete joints, respectively. Due to complicated definition of viscosity, coefficients of these models which control damage behaviour still need much study to correlate with mixed-mode crack of FRP-concrete interface.

To the author's knowledge, existing numerical simulations for FRP-concrete mixed-mode debonding performance has only been analysed with bilinear CZMs. Most existing exponential models were developed for metallic joint and composite laminates. In addition, the dynamic enhancing effect of these introduced bilinear models and exponential models mainly considers the dynamic response of adhesive joints and metallic joints, which are different from the FRP-concrete bond joint. For metallic joints and composite laminates, failure is noticed in adhesive. However, the debonding behaviour of FRP-concrete joint generally failed in the concrete layer [3] and its dynamic response is closely related to material properties of concrete and FRP

[22]. For modelling the FRP-concrete bond subjected to dynamic loading, it is important to include material properties of both adherents.

The failure behaviours of FRP-concrete interface in mode I and mode II show large difference. Qiao and Xu [6] and Coronado and Lopez [7] experimentally studied the bond failure in mode I and found that both the peak bond stress and the fracture energy were close to those of concrete material. However, according to the shear debonding studies conducted by Lu et al. [3] and Dai et al. [5], the shear peak stress could be a bit higher than that in normal direction and fracture energy in shear separation would be 3 to 6 times larger than that in normal direction. CZM is able to perform flexible fitting to accommodate these significant differences between mode I and mode II as well as to give realistic response under mixed mode.

This study proposed a new coupled dynamic CZM to simulate the mixed-mode separation of FRP-concrete interface. To continuously describe the nonlinear fracture process, this model includes elastic, rate-dependent hardening and exponential softening ranges. In the post-peak damage range, the model is irreversible and the unloading behaviour is defined as the path from the current point back to the origin. As the dynamic fracture behaviour of FRP-concrete interface does not always occur in a constant rate [54], current model also considers performance of rate variation in the traction-separation path. Coefficients related to dynamic response are identified based on existing FRP-concrete dynamic tests. To evaluate mixed-mode response of this model, the mixity assessment was conducted for both the traction-separation curves and the energy dissipation under different separation angles.

After the derivation of this model, it is programmed into a commercial finite element code (LS-DYNA) to simulate various FRP-concrete dynamic separation tests, e.g. single-lap shear tests, three-point bending tests and FRP strengthened RC beam

subjected to a dynamic load. The element size effect is evaluated through varying element sizes of finite element (FE) model and comparing the predictions with corresponding test results. Finally, the verified model is used in finite element analysis (FEA) of FRP strengthened RC walls subjected to blast loads.

6.2 Proposed coupled dynamic CZM

6.2.1 Constitutive equations

As shown in Figure 6.1, the fracture behaviour of the interface bond between FRP and concrete is represented through the interface element. According to the fracture directions, mode I fracture represents normal opening mode and mode II is transverse shear slip between FRP and concrete. Mixed-mode crack represents the inclined separation path between mode I and mode II. The interface formulation proposed in this study continuously presents elastic, rate-dependent hardening and exponential softening stages of mode I, mode II and mixed mode. The three-dimension constitutive equations are given as follows with subscripts 1 for normal direction, 2 and 3 for the two in-plane shear directions.

Figure 6.1 Schematic of interface between FRP and concrete

As stress-separation law in normal direction only considers tensile (opening) response, normal displacement s_I is equal to

$$s_I = \langle s_1 \rangle \quad (6.1)$$

where Macauley operator indicates that separation in mode I does not have negative

value.

The resultant shear separation of displacement in directions 2 and 3 is given by

$$s_{II} = \sqrt{(s_2)^2 + (s_3)^2} \quad (6.2)$$

In Eqs. (6.1) to (6.2), s_1 , s_2 and s_3 are the relative displacements between the FRP layer and the concrete layer in three orthogonal directions. The mixed-mode separation s_m is determined by the separations in normal s_I and in shear s_{II} directions, as follows.

$$s_m = \sqrt{s_I^2 + s_{II}^2} \quad (6.3)$$

To calculate the mode mixity, following Jiang et al. [86], the direction cosines for mode I and II are defined as

$$\cos I = \frac{s_I}{s_m}, \cos II = \frac{s_{II}}{s_m} \quad (6.4)$$

In the initial elastic range, the stress in each direction increases linearly with the growth of separation. After that, the starting point of rate-dependent hardening range needs to be determined. The quadratic mixed-mode hardening initiation criterion is used to predict onset of dynamic hardening range.

$$\left(\frac{\sigma_I}{\sigma_{Ist}}\right)^2 + \left(\frac{\sigma_{II}}{\sigma_{IIst}}\right)^2 = 1 \quad (6.5)$$

where σ_I and σ_{II} are tensile and shear stresses, respectively, σ_{Ist} and σ_{IIst} are the static bond strengths in pure mode I and mode II, respectively. Therefore, the mixed-mode separation corresponding to the hardening initiation is determined as.

$$s_{mst} = \frac{1}{\sqrt{(K_I \cos I / \sigma_{Ist})^2 + (K_{II} \cos II / \sigma_{IIst})^2}} \quad (6.6)$$

where K_I and K_{II} are the initial stiffnesses in normal and shear directions,

respectively. Since the mode-dependent stiffness can cause inconsistent crack behaviour in mixed mode [90, 91], the stiffness used in this study is not a physical parameter obtained from actual tests. To keep the mixed-mode crack computation reasonable, a large value is suggested for the stiffnesses in both mode I and mode II, viz. $K_I = K_{II}$.

After reaching dynamic bond strength in the hardening stage, damage initiation is determined by dynamic bond strengths of mode I and mode II, viz σ_{Id} and $\sigma_{II d}$. The quadratic yield criterion is given below.

$$\left(\frac{\sigma_I}{\sigma_{Id}}\right)^2 + \left(\frac{\sigma_{II}}{\sigma_{II d}}\right)^2 = 1 \quad (6.7)$$

The studies of Bazant [92] and Bazant and Li [93] are introduced to determine dynamic bond strengths σ_{Id} and $\sigma_{II d}$. Based on the activation energy theory, Bazant [92] and Bazant and Li [93] derived the relationship between concrete opening rate $\dot{w}$ and the stress increment in the fracture process from quasi-static state to the referred opening rate $\sigma - f(w)$, with an equation written as $\dot{w} = \dot{w}_0 \sinh\left(\frac{\sigma - f(w)}{af_t}\right) \exp\left(\frac{Q}{RT_0} - \frac{Q}{RT}\right)$. The function $f(w)$ represented the fracture stress in quasi-static state. Coefficients $\dot{w}_0$ and f_t represented reference crack rate and tensile strength, respectively. Coefficient a was a constant to be determined from tests. Coefficients Q , R , T and T_0 were activation energy, gas constant, absolute temperature and chosen reference temperature, respectively. For the condition of a constant temperature $T = T_0$, the rate-dependent equation was simplified as $\dot{w} = \dot{w}_0 \sinh\left(\frac{\sigma - f(w)}{af_t}\right)$. In their studies, this equation was validated for

a normal scale of concrete fracture. Although Bazant [92] and Bazant and Li [93] just considered their equation for concrete opening crack, this study extends its application to the dynamic bond strengths in normal and shear directions of FRP-concrete interface. The opening rate $\dot{w}$ is termed as separation rates $\dot{s}_I$ and $\dot{s}_{II}$ for pure mode I and mode II, respectively. The tensile strength f_t is defined as static bond strengths of σ_{Ist} and σ_{IIst} in pure mode I and mode II, respectively. As the equations for $\dot{s}_I$ and $\dot{s}_{II}$ are only considered for determining dynamic bond strengths rather than the whole fracture process, the function of quasi-static fracture process $f(w)$ is defined as $f(w) = \sigma_{Ist}$ or σ_{IIst} for mode I or II, respectively. After some manipulation, the rate-dependent bond strengths in pure mode I σ_{Id} and mode II σ_{IId} are defined as follows.

$$\sigma_{Id} = \sigma_{Ist} (1 + a_I \cdot a \sinh(\frac{\dot{s}_I}{\dot{s}_{I0}})) \quad (6.8)$$

$$\sigma_{IId} = \sigma_{IIst} (1 + a_{II} \cdot a \sinh(\frac{\dot{s}_{II}}{\dot{s}_{II0}})) \quad (6.9)$$

where $\dot{s}_{I0}$ and $\dot{s}_{II0}$ are reference separation rates of mode I and mode II at quasi-static regime, respectively, a_I and a_{II} are constants related to dynamic effect and they are determined later.

Hardening moduli in normal and shear directions follow the initial stiffnesses K_I and K_{II} , respectively. Following the hardening range, the displacement of damage initiation s_{md} is derived from Eq. (6.7), as shown in Eq. (6.10).

$$s_{md} = \frac{1}{\sqrt{(K_I \cos I / \sigma_{Id})^2 + (K_{II} \cos II / \sigma_{IId})^2}} \quad (6.10)$$

The dynamic mixed-mode energy is required to determine the softening range of mixed-mode separation. The mixed-mode fracture energy G_{Md} in this study follows the suggestion by Benzeggagh and Kenane [94] and May [95]. The measured stiffness values in mode I and mode II tests (K_{I0} and K_{II0}) are used in this equation to realistically describe the effect of actual stiffness on the mixed-mode fracture energy G_{Md} .

$$G_{Md} = G_{Id} + (G_{IId} - G_{Id}) \cdot \left(\frac{K_{II0}(s_{II} / s_I)^2}{K_{I0} + K_{II0}(s_{II} / s_I)^2} \right) \quad (6.11)$$

where G_{Id} and G_{IId} are the dynamic fracture energy in mode I and mode II, respectively, and they are determined in Section 6.2.2.

In the softening stage, the exponential damage behaviour is proved to be close to the realistic softening response [3]. The damage equation is modified from the shear bond-slip model by Lu et al. [3]. The current study extends their shear damage function into mixed-mode form to determine the softening behaviour in mixed mode. The displacement s_{md} is used to determine the rate-dependent response in the softening range. The mixed-mode damage function is given by

$$D = 1 - \frac{s_{md}}{s_m} \exp(-\alpha(\frac{s_m}{s_{md}} - 1)) \quad (6.12)$$

where α is a parameter that controls the damage shape of the softening range and

is defined as $\alpha = \frac{\sigma_{md}s_{md}}{G_{Md} - G_a}$, G_a is the linear elastic fracture energy of initial

ascending branch and is determined by $G_a = \frac{1}{2} \sigma_{md} s_{md}$.

The stress components in each direction are shown in Eqs. (6.13) to (6.15), including

elastic, rate-dependent hardening and softening ranges. In the softening range, stiffness degradation is irreversible and unloading/reloading path in stress-separation curve is defined as from current state to the origin. Additionally, to prevent penetration between CFRP and concrete material through interface element, the penalty function is defined in Eq. (6.13) when $s_I < 0$.

$$\sigma_1 = \begin{cases} K_I s_1 & s_1 < 0 \\ K_I s_1 & s_1 \geq 0 \text{ and } s_m < s_{mst} \\ K_I s_1 & s_1 \geq 0 \text{ and } s_{mst} \leq s_m < s_{md} \\ (1-D) \cdot K_I s_1 & s_1 \geq 0 \text{ and } s_{md} \leq s_m \end{cases} \quad (6.13)$$

$$\sigma_2 = \begin{cases} K_{II} s_2 & s_m < s_{mst} \\ K_{II} s_2 & s_{mst} \leq s_m < s_{md} \\ (1-D) \cdot K_{II} s_2 & s_{md} \leq s_m \end{cases} \quad (6.14)$$

$$\sigma_3 = \begin{cases} K_{II} s_3 & s_m < s_{mst} \\ K_{II} s_3 & s_{mst} \leq s_m < s_{md} \\ (1-D) \cdot K_{II} s_3 & s_{md} \leq s_m \end{cases} \quad (6.15)$$

The stress-separation curves of these constitutive equations are shown in Figure 6.2. In the positive domain of Figure 6.2(a), it presents similar traction-separation curves between pure mode I and mode II in elastic, rate-dependent hardening and softening ranges. However, unlike symmetrical behaviour of mode II in the negative domain, the curve in the normal direction (mode I) uses penalty function to prevent compressive penetration. As far as the stress behaviour under mixed-mode separation is concerned, Figure 6.2(b) shows the coupled stresses of mode I and mode II in three-dimensional (3D) domain. Character θ denotes the separation angle in mixed-mode crack.

(a) traction-separation curves of mode I and mode II

(b) coupled 3D domain

Figure 6.2 (a) Pure mode traction-separation curves and (b) coupled 3D domain for the CZM

6.2.2 Identify coefficients

Since FRP-concrete debonding commonly occurs in concrete layer [96, 97], the static bond strength in mode I can be correlated to concrete strength f'_c . Based on regression study in Chapter 5, the static bond strength in mode I could be predicted using the equation below.

$$\sigma_{Ist} = 0.257(f'_c)^{2/3} \quad (6.16)$$

The dynamic coefficient a_I in Eq. (6.8) is used to determine dynamic bond strength

σ_{ld} and it is identified in the experimental study in Chapter 5 as follows.

$$a_I = 0.0082\left(\frac{\dot{f}_c}{f_0}\right) + 0.0008 \quad (6.17)$$

Qiao and Xu [6] and Coronado and Lopez [7] experimentally studied mode I fracture energy of FRP-concrete interface and indicated that the values of static fracture energy in mode I (G_{Ist}) were close to those of plain concrete (G_{FC}), as shown below.

$$G_{Ist} \cong G_{FC} \quad (6.18)$$

Based on the mode I dynamic study in Chapter 5, the mode I dynamic fracture energy G_{ld} is determined by the static fracture energy (G_{Ist}) and the corresponding dynamic increase factor (DIF_{GI}) as follows.

$$G_{ld} = DIF_{GI} \cdot G_{Ist} \quad (6.19)$$

where dynamic increase factor $DIF_{GI} = 0.26 \sin h^{-1}\left(\frac{\dot{s}_I}{1.53}\right) + 1.0$.

According to studies of Chen and Teng [4] and Lu et al. [3], static shear bond strength σ_{Ist} is influenced by different bond widths between the FRP sheet and the concrete substrate. Based on regression analyses, the width calibration factor β_w was given by Chen and Teng [4] as a function of the FRP sheet width b_f and the concrete substrate width b_c , as shown in Eq. (6.20). Its role is to convert the condition where bond widths are equal ($b_f = b_c$) to that of different widths $b_f \neq b_c$.

$$\beta_w = \sqrt{\frac{2 - b_f / b_c}{1 + b_f / b_c}} \quad (6.20)$$

Incorporating the width calibration factor β_w , static shear bond strength σ_{Ist} can

be defined as a function of the concrete shear strength τ_{con} in this study and is shown below.

$$\sigma_{lst} = \frac{\beta_w}{\beta_0} \tau_{con} \quad (6.21)$$

where β_0 represents the condition of $b_f = b_c$ and is equal to $1/\sqrt{2}$. In accordance with the investigation by Guo and Shi [73], concrete shear strength is close to its tensile strength, i.e. $\tau_{con} \approx f_t$. The concrete tensile strength is given by the ceb model concrete code 2010 [74] as a function of cylinder compressive strength: $f_t = 0.3(f_c')^{2/3}$. Through these equations, static bond strength σ_{lst} can be determined using known material and geometric parameters of concrete and FRP.

The dynamic coefficients a_{II} in Eq. (6.9) is determined for dynamic bond strength σ_{lld} . Coefficient a_{II} is determined using the following method.

There were four groups of single-lap shear tests conducted in Chapter 3 including different material and geometric parameters, as shown in Table 6.1. Specimens C40-1 were a control group with carbon fibre reinforced polymer (CFRP) stiffness of 41.4 GPa·mm and bond width of 33 mm. Specimens C40-2 had a higher CFRP stiffness at 82.7 GPa·mm and specimens C40-3 reduced CFRP bond width at 26 mm. Compared with C40-1 specimens, C60-1 specimens adopted higher concrete strength. Parameters f_c' , $E_f t_f$ and b_f/b_c represent the cylinder concrete strength, the CFRP stiffness and the CFRP-concrete bonding width ratio, respectively. The CFRP sheets in the experiment had a tensile strength of 750 MPa and a bond length of 150 mm. Due to its wide investigation on dynamic shear debonding, the test results of Chapter 3 are used to determine the value of a_{II} , as shown in Figure 6.3. Test data is given and the fitting curve for each group is determined following Eq. (6.9), in which

quasi-static separation rate $\dot{s}_{II0}=0.004$ mm/s. Coefficient a_{II} for each group is determined through regression analysis of data in Figure 6.3 and is shown in Table 6.1.

Table 6.1 Fitting constants for test groups in Chapter 3

Test	Material Properties	Number of tests	a_{II}
C40-1	$f'_c=32.0$ MPa, $E_ft_f=41.4$ GPa·mm, $b_f/b_c=0.33$	18	0.048
C40-2	$f'_c=32.0$ MPa, $E_ft_f=82.7$ GPa·mm, $b_f/b_c=0.33$	11	0.078
C40-3	$f'_c=32.0$ MPa, $E_ft_f=41.4$ GPa·mm, $b_f/b_c=0.26$	9	0.040
C60-1	$f'_c=53.8$ MPa, $E_ft_f=41.4$ GPa·mm, $b_f/b_c=0.33$	13	0.016

Figure 6.3 Dynamic enhancing effect on the peak bond stress

Investigated in Chapter 3, the dynamic enhancing effect of the CFRP-concrete interface bond was determined by static tensile load capacity of CFRP denoted by F_f and concrete cylinder strength f'_c . A larger CFRP load capacity could make the bond sustain a significant dynamic loading at the impact regime. However, concrete with a higher strength could make the ultimate load of the bond approach to the CFRP

load capacity and result in the bond being less sensitive to dynamic enhancing effect. Coefficient η is introduced to represent this effect and its relationship with coefficient a_{II} is investigated. According to Chapter 3, η is calculated as the ratio of CFRP tension capacity to concrete cylinder strength, as shown in Eq. (6.22) and Table 6.2.

$$\eta = \frac{F_f / F_0}{f_c' / f_0} \quad (6.22)$$

where F_0 is defined to be 1 kN and f_0 is defined to be 10 MPa.

Table 6.2 Coefficient η

Test group	Concrete strength f_c' (MPa)	FRP tensile strength f_{tf} (MPa)	CFRP interface area A_f (mm ²)	CFRP ultimate load F_f (kN)	Coefficient η
C40-1	32.0	750	33*0.5	12.38	3.87
C40-2	32.0	750	33*1.0	24.75	7.73
C40-3	32.0	750	26*0.5	9.75	3.05
C60-1	53.8	750	33*0.5	12.38	2.30

As shown in Figure 6.4, a monotonically increasing relationship is found between the coefficients a_{II} and η for the four test groups. Group C40-2 with the highest coefficient η presents the most sensitive dynamic enhancing effect, and vice versa for C60-1 group.

Figure 6.4 Coefficient a_{II} and coefficient η

As indicated in Figure 6.4, a regression fitting line is generated for a_{II} . The predictive function is provided as below, with $R^2=0.97$.

$$a_{II} = 0.049 \cdot \ln(\eta) - 0.0198 \quad (6.23)$$

According to the mode II dynamic study in Chapter 3, the dynamic fracture energy in mode II ($G_{II d}$) is determined by the corresponding static fracture energy ($G_{II st}$) and dynamic increase factor (DIF_{GII}) as follows.

$$G_{II d} = DIF_{GII} \cdot G_{II st} \quad (6.24)$$

where static fracture energy $G_{II st} = 0.308 \beta_w^2 \sqrt{f_t}$, dynamic increase factor

$$DIF_{GII} = \left(\frac{\dot{s}_{II}}{\dot{s}_{II 0}} \right)^b \text{ and } b = 0.0264 \cdot \ln(\eta) + 0.0295.$$

6.2.3 Rate variation

In actual dynamic fracture tests and crack events, the separation rate is not always a constant in the failure process. As early models were a lack of consideration for this behaviour, May et al. [54] proposed new formulations to continuously update the shape of bilinear and trapezoid CZMs. Different from May's method, the current nonlinear CZM is naturally applicable to the effect of rate variation. The behaviour of rate variation in pure mode I and pure mode II shows in the softening range in Figures 6.5(a) and (b), respectively. The interface is initially cracked at a constant rate of 100 mm/s. When the separation rate of the softening curves increases to 1000 mm/s, the stress-separation curve jumps to the softening path of the higher crack rate, and vice versa.

Figure 6.5 Stress-separation curves of (a) pure mode I and (b) pure mode II under rate variation scenarios

6.2.4 Numerical implementation

The model is implemented into the explicit finite element code LS-DYNA via a user-defined material (UMAT). The calculation procedure is presented as a flowchart in Figure 6.6. Firstly, material parameters need to be defined in material cards. During the calculation process, the displacement increment in each direction is collected in every time step. In this stage, separation rates in both mode I and mode II are calculated. The rate-dependent bond strengths and the mixed-mode coefficients are determined. Then the damage state is calculated following Eq. (6.12). Correspondingly, the stress states of σ_1 , σ_2 , σ_3 are updated and the procedure is repeated in the next time step. When the damage parameter D reaches 1.0, the element is failed.

Figure 6.6 Flow chart for interface stress computation

6.3 Assessment of mode mixity

6.3.1 Effect of mode mixity on the traction-separation curves

Dimitri et al. [53] mentioned that a good coupling behaviour between the normal and the shear directions is necessary in a CZM to describe the interface crack realistically. If the coupling behaviour is not well considered, the load carrying capacity of the

interface may lead to unreliable predictions. Therefore, the proposed model in this study is assessed for stress-separation curves and energy dissipation of FRP-concrete interface. The stress-separation curves under mixed-mode crack could provide insight on the effect of coupling. The mode mixity is expressed in terms of separation angle. The parameters used in the mode mixity analysis are $\sigma_{Ist}=3$ MPa, $\sigma_{IIst}=5$ MPa, $G_{Ist}=0.1$ N/mm and $G_{IIst}=0.6$ N/mm.

Figure 6.7 shows the response of the stress-separation curves in normal and shear directions under varying separation angles. Pure mode I separation is defined to be 90 degrees and pure mode II is defined to be 0 degree. In Figures 6.7(a) and (b), the stiffness of the ascending range is not affected by the mixed-mode separation. It is reasonable to find a gradual reduction in the peak bond stresses and the softening ranges in the normal direction with decrease of separation angles. Conversely, the maximum stresses and the descending ranges in mode II are continuously increased until the separation angle reaches pure mode II. This consistent behaviour could help numerical simulations reasonably indicate mixed-mode cracks.

(a)

(b)

Figure 6.7 Stress-separation curves of (a) mode I and (b) mode II under mixed-mode separation

Figure 6.8 shows the rate-dependent stress-separation curves in the normal and shear directions. Due to dynamic enhancing effect, peak bond stresses and in both directions increase clearly with higher separation rates. Among the four charts, with the reduction of separation angles, the contribution from mode I is gradually decreased and the crack is dominated by traction in mode II. The consistent response of stress-separation curves from quasi-static case to high separation rate of 1000 mm/s present that the proposed CZM has a reasonable performance under dynamic mixed-mode separation cases.

Figure 6.8 Stress-separation curves of mode I and mode II under various separation rates

6.3.2 effect of mode mixity on the energy dissipation

The energy dissipation under consistent separation angles is analysed for the coupled CZM. The total mixed-mode fracture energy is the sum of dissipated energy in mode I and mode II. As shown in Figure 6.9, the total, normal and shear energy dissipations are continuously captured under different separation angles. The BK law represents the equation proposed by Benzeggagh and Kenane [94] to predict the mixed-mode fracture energy, i.e. Eq. (6.11). Figure 6.9(a) presents the condition that fracture energy in mode I is same as that in mode II, $G_{Ist} = G_{IIst}$, which is usually defined in mixity analyses of other CZMs [53, 85]. Energy dissipation in normal and shear

directions vary by equal and opposite amounts with the increase of separation angle. As a result, the total energy dissipation is maintained at the same level. It means that, with same fracture energies in pure mode I and mode II, the total fracture energy is not influenced by mode mixity and could maintain a same extent under different crack angles.

Figure 6.9(b) presents the consistency of these curves when $G_{Ist}=0.1$ N/mm and $G_{IIst}=0.6$ N/mm, which is found normal in FRP-concrete interface separation [3, 6]. The total energy reduces monotonically from pure mode II to mode I. In this process, the fracture energy in mode II reduces faster than the total energy. The fracture energy in mode I gradually increase to the same value as total energy in pure mode I. The consistent responses of fracture energy in mode I, mode II and total energy indicate that the proposed CZM can reasonably predict the mixed-mode separation.

Figure 6.9 Mixed-mode energy dissipation

To verify the mixed-mode energy dissipation in dynamic scenarios, Figure 6.10 compares the mixed-mode fracture energy between the CZM and the predictions by BK law. With the increase of separation rate from static to 1000 mm/s, the dynamic enhancement in fracture energy under different separation angles is observed. In this process, the predictions of CZM under varying crack angle match those of BK law

very well, which indicate that the fracture energy of the proposed CZM still follows BK law for mixed-mode dynamic separation.

Figure 6.10 Mixed-mode energy dissipation under different separation rates

6.4 FRP-concrete interface debonding

The commercial software LS-DYNA is used for the FE simulations in this study due to its common application in explicit dynamic simulations. The constitutive equations proposed in Section 6.2.1 are programmed into a user-defined material model. The material properties of the interface element, including the static bond strength σ_{Ist} and σ_{IIst} , fracture energy G_{Ist} and G_{IIst} and coefficient a_I and a_{II} can be defined in the card UMAT.

6.4.1 CFRP-concrete interface shear debonding

The single-lap shear test is commonly used to study the FRP-concrete shear bond behaviour. In such tests, FRP is bonded on the concrete surface and is subjected to tensile load. The shear tests done in Chapter 3 are simulated to verify the mode II behaviour of the CZM. A sketch of dimensions and FE model are shown in Figure 6.11. The length, width and height of concrete block are 190 mm, 100 mm and 100 mm, respectively. The CFRP is in the top of the FE model with bonding length and

width of 150 mm and 33 mm, respectively. The interface elements are inserted between the FRP and the concrete substrate along the bond length to study bond behaviours. The thicknesses of CFRP and the interface layer are set to 1 mm. To model the actual test condition, the model is restrained along the whole bottom and the front edge with height of 80 mm.

Figure 6.11 (a) Geometry sketch and (b) FE model

Due to the linear elastic behaviour along the fibre direction, the CFRP sheet is simulated using a linear elastic model. To maintain the stiffness for the CFRP sheet, the value of the elastic modulus multiplying the thickness of CFRP layer in the FE model should be equal to the actual stiffness of the measured CFRP sheet, i.e. $E_{f-FE}t_{f-FE} = E_{f-test}t_{f-test}$. The input CFRP stiffness equals to 41.4 GPa·mm. The concrete component is simulated by K&C concrete damage model [44] which is a typical plastic concrete material model and has been widely used in dynamic simulations [10]. With an input concrete cylinder strength, relevant parameters of concrete material could be automatically generated in this concrete model. The concrete cylinder strength f'_c is 32.0 MPa. Considering the significant dynamic enhancing effect on concrete material, dynamic increase factors (DIFs) are defined in the concrete model according to the LS-DYNA material manual [47]. The interface element is defined by the proposed CZM and, according to Section 6.2.2, input

parameters are given as follows: $\sigma_{Ist}=2.6$ MPa, $\sigma_{IIst}=4.8$ MPa, $G_{Ist}=0.1$ N/mm, $G_{IIst}=0.68$ N/mm, $a_I=0.027$ and $a_{II}=0.048$.

To verify the size effect on the proposed CZM, element sizes of 2, 5, 11 and 33 mm are used to simulate the single-lap shear test at a loading rate of 1130 mm/s, as shown in Figure 6.12.

Figure 6.12 FE models with different element sizes

Figure 6.13 shows the predicted load-displacement curves. The load-displacement histories predicted by the four different element sizes match the test results well. The influence of element size can be found in the gradients of these curves at the initial range. The increase of element size from 2 to 33 mm results in an increase of element stiffness in the concrete material, which present a steeper gradient in the beginning of these curves. It should be noted that the predicted load-displacement curves in Figure 6.13 are not so sensitive to different element size. In fact, unlike solid elements which are determined by strain in three directions, the CZM is determined by separation which is a physical displacement and its response is not so much affected by different element sizes.

Figure 6.13 Predicted load-displacement histories by FE models with different element sizes

After investigation of element size effect, the mesh size of 5 mm is used in following simulations. Four tests are simulated from quasi-static to impact scenarios, namely, M1-C40-1C, M4-C40-1A, S1-C40-1C and S2-C40-1B, with loading rates of 0.02 mm/s, 30 mm/s, 1130 mm/s and 2150 mm/s, respectively. Strain contours of numerical simulations are shown in Figure 6.14. The interface elements are intact before loading is applied in Figure 6.14(a). In Figure 6.14(b), applied load attains the ultimate load and failure occurs at the interface elements near the loaded end.

(a) before load

(b) the moment of reaching ultimate load

Figure 6.14 Strain contours of S1-C40-1C

The load-displacement curve shows the general performance of CFRP-concrete shear test. The load-displacement curves shown in Figure 6.15 compare the results of numerical simulations with the test results to verify the predicted load process. Identical to the actual curves, the predicted curves showed a linear increase at initial displacements, indicating no damage behaviour among the interface elements. After gradually decreased gradients, the load reaches the ultimate load value and the interface element failure starts from the load end. Subsequently, the ultimate load is maintained, and the interface elements fail one by one from the load end. Different from smooth curves in numerical simulations, the test results show more oscillation in the displacement range from 0.2 mm to 0.4 mm, which is attributed to surface roughness and non-uniformly distributed sand and stone in actual concrete material. Generally, the FE model with the interface element can accurately simulate the CFRP-concrete debonding process.

Figure 6.15 Comparison of load-displacement curves between test results and FEA results

6.4.2 FRP-concrete interface mode I

Three-point bending tests with notched specimen are used to measure the concrete tensile fracture energy [35, 98, 99]. Qiao and Xu [6] inserted a layer of FRP at the mid span of the concrete beam to study the mode I fracture behaviour of FRP-concrete interface. This method was also used to study epoxy-concrete interface by Coronado and Lopez [7]. In the current study, the damage process of mode I debonding in the three-point bending tests is simulated with the proposed CZM. The numerical result is compared with experimental result of CFRP-concrete and GFRP-concrete specimens in Qiao's study [6].

The geometry sketch of the three-point bending test is shown in Figure 6.16(a). The three-point bending test is idealized as plane stress scenario and stress behaviour along the thickness is assumed to be same. Therefore, concrete, FRP sheet and interface layer are simulated by one layer of solid elements, as shown in Figure 6.16(b). Mesh size of 2 mm is used in the FE model to properly simulate the crack process in a 50 mm long interface bond. The material models of concrete and interface element follow those in the simulations of the shear tests in Section 6.4.1.

According to the material properties provided by Qiao and Xu [6], the concrete cylinder strength $f'_c=50.0$ MPa is defined here. The input parameters of the interface element are given as follows: $\sigma_{lst}=4.42$ MPa, $\sigma_{llst}=4.42$ MPa, $G_{lst}=0.12$ N/mm, $G_{llst}=0.34$ N/mm and $a_l=a_{ll}=0.0$. No dynamic enhancing effect is defined in the concrete component and the interface elements for this quasi-static test. As the CFRP and GFRP sheets are an anisotropic material and they tend to deflect out of their plane. Hence, they are simulated with an orthotropic elastic model. The CFRP and GFRP stiffness values are 45.4 GPa·mm and 43.4 GPa·mm in the in-plane direction, respectively. Young's modulus in their normal directions are given as 3.0 GPa. A loading rate of 0.05 mm/s is defined for these simulations.

Figure 6.16 (a) Geometry sketch and (b) FE model

As a comparison, the load-displacement curves derived from the FE model and the test data are shown in Figure 6.17. The load-displacement curve of CFRP-concrete specimen in Figure 6.17(a) is a typical curve in Qiao's study [6]. The predicted ascending range of CFRP-concrete specimen matches the gradient of test result well. The peak load of FE model shows a slight difference of 5.1%. Though the FE model slightly overestimates the load values in the descending range, it can be seen that the FE simulation with CZM properly model CFRP-concrete mode I crack process. In Figure 6.17(b), initial slip is found at the beginning of the testing curve of GFRP-concrete specimen. After avoiding the initial slip, the load-displacement curve of FE simulations predicts the gradient and the maximum load in the ascending range fairly well. The maximum loads between the FE model and the actual test in Figure 6.17(b) show a difference of 5.7%.

Figure 6.17 Load-displacement curves of three-point bending test

6.4.3 CFRP strengthened RC beam subjected to a dynamic loading

White et al. [12] tested CFRP strengthened RC beams under a dynamic load to investigate dynamic effect on the strengthened RC beams. Beams with two types of CFRP strengthening are modelled in this study and a diagram of the strengthened beam is presented in Figure 6.18. As they observed CFRP peeling in their tests, which is a kind of mixed-mode debonding [8], these beam tests are used to verify the

response of the proposed CZM under mixed-mode separation. The concrete cast for the beam had a compressive strength of 45.6 MPa and a tensile strength of 3.3 MPa. Reinforcing steel bars in flexure and shear resistance were Grade 400W (minimum yield strength of 400 MPa). Two types of CFRP laminates were used to strengthen RC beams: S-type and R-type. S-type had an elastic modulus of 155 GPa and a sectional area of $100 \times 1.2 \text{ mm}^2$, while R-type had an elastic modulus of 160 GPa and a sectional area of $135 \times 0.55 \text{ mm}^2$. The dynamic tests were conducted under a loading rate of 36 mm/s.

Figure 6.18 Geometry sketch of the beam

The flexural tests are simulated with three-dimensional FE models. Concrete and interface elements are simulated with solid elements, while steel bars and CFRP laminates are modelled with beam and shell elements, respectively. Due to the large dimensions of the beam, a mesh size of 20 mm is used in the FE model. The interface elements are defined as 1 mm thick. Material models of concrete, CFRP sheet and interface element are kept the same as those in Section 6.4.1. Dynamic enhancing effect of pure concrete material is defined in the concrete component according to the LS-DYNA user's manual [47]. Moreover, steel bars are simulated by a kinematic hardening plasticity model. According to identification of coefficients in Section 6.2.2, the input parameters of the interface elements for S-type strengthening are given by: $\sigma_{Ist} = 3.3 \text{ MPa}$, $\sigma_{IIst} = 4.4 \text{ MPa}$, $G_{Ist} = 0.12 \text{ N/mm}$, $G_{IIst} = 0.45 \text{ N/mm}$, a_I

$=0.038$ and $\sigma_{II}=0.16$. For R-type strengthening, the interface elements have properties as follows: $\sigma_{Ist}=3.3$ MPa, $\sigma_{IIst}=3.3$ MPa, $G_{Ist}=0.12$ N/mm, $G_{IIst}=0.32$ N/mm, $a_I=0.038$ and $a_{II}=0.15$.

Figure 6.19 shows the load-deflection curves of the experimental data and the numerical predictions. The predicted load-deflection curves of both S-type and R-type strengthened beams fit the test results well in the hardening range and the maximum loads. However, the gradients of the initial range in predicted curves are slightly higher than the actual test. Such discrepancy could be due to that the simplified boundary condition in the FE models which slightly restricts the flexure behaviour and shows a little compression at the beginning. With the increase of loading, this discrepancy gradually reduces and becomes negligible. The CFRP debonding is noticed after the maximum load is attained, as shown in Figure 6.20. Generally, the predicted curves by the FE models accurately match the load-deflection history and CFRP debonding behaviour in the tests. Thus, the proposed model can be used in FE simulations of the flexural tests of FRP strengthened RC beams.

Figure 6.19 Load-deflection curves of the flexural tests

Figure 6.20 Final damage plot and CFRP debonding in the beam with R-type strengthening: (a) FE simulation and (b) actual test

6.5 FRP-concrete wall under blast loading

FRP strengthening system helps RC structures to reduce deflection and spalling at the rear face in blast tests [16, 17]. Due to the impulsive nature of blast loading, dynamic enhancing effect on the FRP-concrete interface bond becomes significant. In the simulations of FRP strengthened RC wall under blast loading, Li X.Q. [10] and Nam et al. [45] considered the dynamic enhancing effect on the interface bond between FRP and concrete. In the current study, blast test of G-2S-1L-a conducted by Tanapornraweekit et al. [16] is modelled to verify the dynamic response of the proposed CZM under blast loading. The dimension of this test slab is 2000 mm × 1000 mm × 75 mm. A single-layer glass fibre reinforced polymer (GFRP) sheet was externally bonded on the front and the rear side of the slab, respectively. Reinforcement details of the test slab is shown in Figure 6.21(a). The test slab was supported in one direction by a steel frame. Trinitrotoluene (TNT) charge of 0.45 kg was suspended at 0.5 m above the test slab, as shown in Figure 6.21(b).

(a)

(b)

Figure 6.21 (a) Dimension of test slab and (b) overall test setup

In this simulation, concrete and interface elements are simulated by solid elements. Steel bars are simulated with beam elements and GFRP sheets are modelled with shell elements. In terms of material models, concrete follows that in Section 6.4.1, which is modelled by K&C concrete model with input dynamic increase factors (DIFs). According to material properties provided by Tanapornraweekit et al. [16], the input concrete strength is 32.0 MPa. Steel bars are modelled by plastic kinematic model in LS-DYNA, which is applicable for isotropic material with plastic kinematic hardening behaviour. Input elastic modulus, yield stress and failure strain are 194 GPa, 356 MPa and 22.2%, respectively. Orthotropic elastic model is used to simulate the anisotropic mechanical properties of GFRP sheet. The elastic moduli along the fibre direction and the transverse direction are 75.6 GPa and 17.7 GPa, respectively. The thickness of the GFRP sheet is given as 0.353 mm. Based on the identification

of coefficients in Section 6.2.2, the input parameters of the interface elements are defined as follow: $\sigma_{Ist}=2.6$ MPa, $\sigma_{IIst}=3.0$ MPa, $G_{Ist}=0.11$ N/mm, $G_{IIst}=0.32$ N/mm, $a_I=0.027$ and $a_{II}=0.112$. The blast pressure-time history in this study is defined by the keyword Load_blast_enhanced in LS-DYNA which is based on Friedlander exponential equation [47].

The comparison between the FE prediction and the test result of G-2S-1L-a is shown in Figure 6.22. The FE model accurately predicts the first peak and the first trough of the displacement-time history, which means that the FE model could properly simulate the stiffness and damage process of the slab. Points A, B and C are selected in the displacement-time history and the corresponding plastic contour of FE simulations are shown in Figure 6.23. Finally, the FE model predicts a residual mid-span deflection of 4.9 mm, close to the test result of 4.5 mm. This comparison shows that the proposed CZM can give accurate simulations of the interface bond behaviour under blast loading.

Figure 6.22 Displacement-time history of G-2S-1L-a

(a)

(b)

Figure 6.23 (a) Strain contour of G-2S-1L-a in different time stages and (b) failure contour of the tested wall

6.6 Conclusion

Considering a lack of a proper dynamic model for the FRP-concrete interface bond, this study proposed a coupled dynamic CZM to simulate mixed-mode dynamic separation of FRP-concrete interface. The dynamic related coefficients are linked with the material parameters of FRP and concrete. The rate variation behaviour is considered for fracture process in which crack rate is not constant. Existing studies show that the failure behaviour of FRP-concrete interface in mode I and mode II are quite different. Based on the assessment of mode mixity, the proposed CZM can properly fit the stress-separation behaviour of FRP-concrete interface in mode I, mode II and mixed mode.

After the introduction of this model, it is programmed into LS-DYNA as a user-defined material model, viz. UMAT. Through the verification of element size effect, it is shown that the developed CZM is not sensitive to different element sizes. The

proposed CZM is used in various FE models to simulate FRP-concrete debonding behaviour. The dynamic response of the proposed model in shear direction is validated by simulations of FRP-concrete dynamic shear tests. The three-points bending test validates mode I crack of the CZM. The FRP strengthened RC beam subjected to a dynamic loading shows that the CZM could reasonably simulate the mixed-mode separation between the FRP and the RC beam. Finally, simulation of the FRP strengthened RC slab subjected to blast shows that the proposed CZM can also be used to model the FRP-concrete interface bond under ultra-fast dynamic load and without preset separation direction.

Chapter 7 Conclusions and future works

7.1 Conclusions

This research programme experimentally investigated the dynamic fracture behaviour of FRP-concrete interface bond and proposed suitable models to simulate the fracture process based on fracture mechanics. Shear debonding, normal delamination and mixed-mode separation were considered in depth.

Experimental study of FRP-concrete dynamic shear debonding behaviour

In the investigation of shear debonding, FRP-concrete bonded specimens were tested from quasi-static to high slip rates (0.004 mm/s to about 2000 mm/s). For the high slip rate regime, a novel experimental method with a modified SHPB set-up was proposed to reach a rate of 2000 mm/s. As concrete was weak in tension, all the specimens under quasi-static load and most specimens under dynamic load failed by debonding in concrete substrate. Some specimens in dynamic tests failed by fracture in both concrete and epoxy due to stress concentration under dynamic loading. However, at a significant high slip rate, the maximum load in the single-lap shear test could reach FRP load capacity and specimens might fail by FRP fracture itself rather than debonding from concrete substrate. This finding indicated that the maximum load of such tests could be limited by the FRP load capacity.

Through a detailed parametric study, the effect of investigated parameters was understood better. FRP laminate with a higher stiffness could result in a stiffer bond and reduce the relative slip between FRP and concrete. FRP bond width could affect its load capacity. At the impact regime, a lower FRP load capacity could lead to FRP fracture before completing its debonding. In this way, FRP did not withstand the violent impact and lost its ability to provide further protecting for concrete structures. However, FRP laminate with a higher load capacity could behave better in this

condition. When the bond length was concerned, at a low slip rate, reducing the bond length below the effective bond length could decrease the maximum load. However, at a high slip rate, specimens with a much shorter bond length could attain equal ultimate loads as those specimens with longer bond lengths, which indicated that effective bond length would reduce due to dynamic increase of concrete strength. From the comparison of specimens with different concrete strengths, higher strength concrete could make the ultimate loads of the single-lap shear tests approach CFRP load capacity and result in the bond-slip behaviour becoming less sensitive to dynamic enhancing effect.

In terms of interface bond properties, it was clearly found that, with a growth of slip rate, the peak bond stress, the fracture energy and the ultimate load showed dynamic enhancing effect. DIF equations were proposed for the peak bond stress and the fracture energy and their accuracy was validated through comparison with a variety of test results.

Theoretical study of FRP-concrete dynamic shear debonding behaviour

Following the experimental study, a continuum damage model was proposed for the interface bond to study the dynamic debonding behaviour based on nonlinear fracture mechanics. Model analyses and regression test data revealed the physical relation between the model coefficient and characteristic parameters of the single-lap shear tests. It was found that the dynamic enhancing effect became more significant due to a larger ultimate load ratio of FRP to concrete. A larger ultimate load ratio indicated a relatively higher FRP load capacity than that of concrete, which made the interface bond able to transfer a large dynamic load from FRP to concrete at the impact regime.

The model was substituted into the stress equilibrium equations of the interface bond to generate functions for the load-displacement history and strain distribution along

the bond length. In this way, the local bond behaviour was related to the global response and the detailed debonding process. The predictions derived from these functions were validated by test results in single-lap shear tests in Chapter 3 and in literature.

Apart from the numerical solutions of the proposed functions, the continuum damage model was defined as an interface element and applied in FE simulations of the single-lap shear tests through a LS-DYNA user-defined material model. The simulations with interface element were compared with those with perfect bond. Although both kinds of simulations predict the test results at low slip rates well, the FE model with interface element performed much more accurately at the high slip regime.

Experimental study of FRP-concrete dynamic tensile fracture behaviour

Two test procedures were conducted to characterise the tensile fracture behaviour of FRP-concrete interface bond. The direct tension tests and the notched three-point bending tests were used to obtain dynamic tensile bond strength and dynamic fracture energy, respectively. In the analysis of direct tension tests, it was found that both the direct tension tests in this study and the split tests in literature provided reasonable bond strength for quasi-static scenario. Based on regression data, a prediction equation for the quasi-static bond strength was provided. In the dynamic tension tests, it was found that concrete tensile strength took a dominant effect on the interface bond strength in both quasi-static and dynamic scenarios. The values of interface bond strength in these tests are just slightly lower than the dynamic tensile strength of concrete material. On the other hand, using of CFRP and GFRP materials had no significant difference on the interface bond strength, as fracture behaviour was dominant in the concrete layer. Accordingly, an equation to predict dynamic bond strength was given and its accuracy was validated by comparing with test results.

In the analysis of the notched three-point bending tests, the dynamic fracture energy increased obviously with the growth of loading rate. DIF equation was generated to predict dynamic enhancing effect on the fracture energy. However, the changes of concrete strength and FRP materials did not lead to a significant different fracture energy.

In the mode I fracture study, the tensile bond strength and the fracture energy for the specimens under quasi-static load were similar to those of plain concrete in magnitude. The dynamic enhancing effect on the mode I bonding properties still relied on the dynamic response of concrete material.

A coupled dynamic CZM for FRP-concrete mixed-mode separation

To consistently model the fracture behaviour from mode I to mode II, a coupled dynamic CZM was proposed to represent the mixed-mode dynamic separation of FRP-concrete interface bond. In the assessment of mode mixity, the proposed CZM could properly describe the stress-separation behaviour of both mode I, mode II and mixed mode.

The proposed CZM was programmed into LS-DYNA as a user-defined material model to simulate the fracture behaviour. The simulations of dynamic shear and tensile fracture behaviours were validated by comparing with the single-lap shear tests and the three-point bending tests, respectively. The numerical simulations of flexural tests showed the ability to model the mixed-mode separation between FRP and concrete. Finally, the simulation of FRP strengthened RC slabs subjected to blast test showed that the proposed CZM could be used to model the interface bond behaviour under high speed load and without pre-set separation direction.

7.2 Future works

The single-lap shear tests introduced in Chapter 3 covered the range of normal strength concrete (below 55 MPa) and an FRP load capacity below 25 kN. The highest slip rate reached 2150 mm/s. Further study of the dynamic interfacial properties is recommended for specimens with a higher concrete strength (above 55 MPa) and a higher CFRP load capacity. Considering extreme impact and blast conditions with a slip rate above 2150 mm/s, the test under a higher loading rate is also recommended.

The continuum damage model and the ultimate load ratio are proposed in Chapter 4. Their predictions on the dynamic enhancing behaviour were validated in the ratio range of 0.85 to 2.36 and below a slip rate of 2150 mm/s. The model application for a higher ratio and at a slip rate of above 2150 mm/s can be verified with the suggested tests in future.

As the Chapters 3, 4 and 5 presented the fracture behaviour under shear and tensile loads, the mixed-mode fracture behaviour has not been well investigated yet. Accordingly, the dynamic tests for the investigation of mixed-mode fracture are suggested.

Based on the CZM proposed in Chapter 6, the dynamic fracture behaviour of FRP-concrete interface bond could be properly simulated. Its further application in the FE simulations could be used to study the FRP strengthening effect on the RC structures in the dynamic scenarios.

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Appendix A: Determination of coefficients in damage equation

This appendix provides the determination details of coefficients n, H, m . The relation between ultimate load ratio μ and damage function power n is shown in Fig. A.1, which clearly indicates that n gradually reduces with increasing μ . According to Eq. (4.2), Eq. (4.19) and Fig. 4.5, a lower value of n indicates more significant dynamic enhancing effect on bond strength $\sigma_{II d}$. This trend is the same with the finding in Chapter 3 that an increase of CFRP load capacity or a decrease of concrete strength could lead to a more significant dynamic enhancing effect on interfacial properties. In Fig. A.1, an accurate fitting equation with an R^2 value of 0.99 is given and it is Eq. (4.21).

Figure A. 1 Power n and ultimate load ratio μ

According to Eq. (4.19) and Fig. 4.5, coefficient H represents the value of $\ln(\sigma_{II d})$ at slip rate $\dot{s}_{II}=1$ mm/s and is expressed in Eq. (A.1a), which is determined by the component of quasi-static peak bond stress $\frac{n}{n+1} \ln(\sigma_{II st})$ and the increment component from quasi-static to dynamic state at a slip rate of 1 mm/s ($H - \frac{n}{n+1} \ln(\sigma_{II st})$). The increment component is indicated by Δ in Eq. (A.1b).

$$H = \frac{n}{n+1} \ln(\sigma_{II st}) + \ln \left(K_{II}^{\frac{1}{n+1}} \left(\frac{n+1}{m+2} \right)^{\frac{m+2}{(m-n+1)(n+1)}} \left(\frac{1}{B} \right)^{\frac{1}{n+1}} \right) \quad (A.1a)$$

$$H = \frac{n}{n+1} \ln(\sigma_{II st}) + \Delta \quad (A.1b)$$

The values of Δ are obtained by $H - \frac{n}{n+1} \ln(\sigma_{IIst})$ and are shown in Table 3.1. The relation between increment Δ and ultimate load ratio μ is shown in Fig. A.2. A monotonically increasing relationship is observed between increment Δ and ultimate load ratio μ for the four test groups. Same with the finding in Chapter 3, C40-2 group with the highest ultimate load ratio, presents the most significant dynamic enhancing effect, and vice versa for C60-1 group with the lowest ultimate load ratio.

Figure A. 2 Increment Δ and ultimate load ratio μ

As indicated by the dashed line in Fig. A.2, a regression fit is generated for coefficient Δ , as shown in Eq. (A.2), with $R^2=0.99$.

$$\Delta = \begin{cases} 2.42\mu - 1.99, & 0.85 < \mu \leq 0.93 \\ 0.11\mu + 0.16, & 0.93 < \mu \end{cases} \quad (A.2)$$

Including the term Δ , an expression for H is shown as follows

$$H = \begin{cases} \frac{n}{n+1} \ln(\sigma_{IIst}) + 2.42\mu - 1.99, & 0.85 < \mu \leq 0.93 \\ \frac{n}{n+1} \ln(\sigma_{IIst}) + 0.11\mu + 0.16, & 0.93 < \mu \end{cases} \quad (A.3)$$

After determining n and H , m is identified for the damage equation (Eq. (4.2)). It is found in regression analysis of the bond stress-slip curves that, when maintaining other coefficients to be unchanged, a smaller value of m can mitigate the damage extent and improve fracture energy, as shown in Fig. A.3.

Figure A. 3 Comparison of bond stress-slip curves with different values in coefficient m

Therefore, coefficient m is related to the fracture energy G_{IId} . According to the bond stress-slip relation in Eq. (4.6), the values of m obtained from regression of the bond-slip results in are shown in Table 4.1. The relationship between coefficient m and ratio μ is identical to that between coefficient n and ratio μ , as shown in Fig. A.4. A best-fitting equation with an R^2 value of 0.97 is given in Fig. A.4.

Figure A. 4 Coefficient m and ultimate load ratio μ