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Research Article

Evaluation of shear strength prediction models for polypropylene fiber-reinforced concrete beams without stirrups: Influence of residual tensile strength estimation on fib-MC2010 code predictions

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ABSTRACT

Predicting the shear strength of polypropylene fiber-reinforced concrete (PFRC) beams remains challenging because most formulations were calibrated using steel fiber-reinforced concrete (SFRC) data, while post-cracking tensile properties required by design models are rarely reported for PFRC members. This study evaluates analytical and code-based models using a database of 98 PFRC beams without transverse reinforcement. The evaluated formulations included analytical models and the RILEM TC 162-TDF, DAFStB, and fib Model Code 2010 approaches. Particular attention is given to fib Model Code 2010, which requires residual tensile strength parameters unavailable for most specimens. Therefore, three alternative procedures were used specifically within the fib Model Code 2010 framework to estimate these parameters. The resulting estimates were incorporated separately into the same shear resistance equation to quantify their influence on predicted capacity. Performance was assessed using the mean experimental-to-predicted shear strength ratio, coefficient of variation, root mean square error, coefficient of determination, mean absolute error, and distributions of safe and unsafe predictions. Considerable differences were observed among the models. Among the analytical formulations, the model proposed by Arslan et al. provided the most balanced performance, whereas DAFStB exhibited the weakest predictive capability. The three fib Model Code 2010 implementations produced substantially different results despite using the same shear resistance equation. The findings show that fib Model Code 2010 predictions are highly sensitive to the residual tensile strength estimation procedure. Reliable PFRC-specific characterization of post-cracking tensile behavior is therefore essential for improving design-oriented shear strength predictions.

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1. Introduction

The shear performance of reinforced concrete (RC) beams has received considerable attention due to the brittle behavior associated with shear failure and its sudden occurrence. While flexural failure normally occurs

along with yielding of longitudinal steel and plastic deformation, shear failure normally involves the fast formation and spread of diagonal cracks. Therefore, the increase of shear strength, restraint of crack propagation, and increase of load-bearing capability after the formation of cracks have been some of the challenges that

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civil engineers face in the analysis and design of RC structures. Among the different methods developed to meet these challenges, the use of fibers in concrete has proven to be an effective method for increasing the tensile properties of concrete (Majdzadeh et al. 2006; Greenough and Nehdi 2008; Jongvivatsakul et al. 2013; Sahoo et al. 2015; Campione et al. 1999; Parmentier et al. 2012).

Fiber reinforced concrete (FRC) has proven itself highly promising in improving the structural behavior of reinforced concrete members. The presence of fibers across cracks helps in transferring tensile stresses, arresting crack opening, increasing aggregate interlocking and improving the residual load carrying capacity of the member post cracking. FRC members generally show higher ductility, energy dissipation capacity, crack distribution and shear strength when compared to ordinary reinforced concrete members (Majdzadeh et al. 2006; Greenough and Nehdi 2008; Jongvivatsakul et al. 2013; Sahoo et al. 2015; Campione et al. 1999; Parmentier et al. 2012). Such promising results have paved the way for investigating and using FRC in structural members that are subjected to shear loading. Of all the fibers that have been studied, steel fiber reinforced concrete (SFRC) has attracted more research interest than any other fiber type both experimentally and analytically. Steel fibers' mechanical properties like high tensile strength, high stiffness and effective bonding properties make them highly useful in bridging shear cracks. Experimental studies have shown that steel fibers help in increasing shear strength of the member and also partly replace transverse reinforcing steel bars (Greenough and Nehdi 2008; Sahoo et al. 2015; Parmentier et al. 2012). Hence, many of the shear strength equations that have been proposed were based on SFRC data (Majdzadeh et al. 2006; Jongvivatsakul et al. 2013; Parmentier et al. 2012). However, their applicability to other fiber types might be questioned since contribution of fibers to the shear resistance depends not only on amount of fibers but also on such factors as fiber stiffness, tensile strength, aspect ratio, bond properties, anchoring process and interaction with concrete matrix. These parameters differ between various fibers which results in decreased efficiency of prediction based on the models created for SFRC. The importance of fiber characteristics, reinforcement distribution, and fiber-matrix interaction mechanisms has also been highlighted in recent theoretical studies on short-fiber-reinforced composite systems. Advanced continuum formulations incorporating non-local elasticity, strain-gradient effects, stress-driven constitutive models, and doublet mechanics have shown that reinforcement configuration, constituent interaction, and scale-dependent effects can considerably influence the mechanical response of fiber-reinforced structures (Akpınar et al. 2025a, 2025b, 2026a, 2026b). This problem is particularly significant for polypropylene fiber-reinforced concrete (PFRC) and other macro synthetic fiber systems. Compared to steel fibers, polypropylene fibers usually demonstrate such characteristics as lower elastic modulus, specific bond properties and pull out behavior. This makes contribution of polypropylene fibers to crack bridging and post cracking properties different from steel fibers. At the same time, there

are several advantages provided by use of polypropylene fibers, including corrosion resistance, low density, easy processing and durability (Conforti et al. 2015; Ababneh et al. 2017; Arslan et al. 2017; Yang et al. 2021; Sahoo et al. 2015).

Recent experiments have shown that polypropylene fibers could significantly affect the shear properties of RC beams. Studies show that crack control, ductility, toughness, and shear resistance were improved, especially for beams with no transverse reinforcements or beams with low stirrup ratios (Conforti et al. 2015; Xiang et al. 2022; Ababneh et al. 2017; Arslan et al. 2017; Yang et al. 2021; Zhang et al. 2020; Ünal et al. 2021). Additionally, the application of polypropylene fibers was also found to positively affect the structural performance of lightweight concrete beams, wide shallow beams, and other types of special structural sections (Conforti et al. 2015; Xiang et al. 2022; Ababneh et al. 2017; Yang et al. 2021). The positive effects of macro-synthetic fibers on the post-cracking properties have also been shown in different studies (Lakavath et al. 2021; Ababneh et al. 2017). However, despite these promising findings, the behavior of polypropylene fibers has proven to be inconsistent among different experimental studies. While in some cases significant improvements in the shear properties of polypropylene fibers were noticed, in others there were no significant changes, especially for lower amounts of polypropylene fibers (Lakavath et al. 2021). On the other hand, many factors such as the compressive strength of concrete, fiber volume fraction, fiber shape, longitudinal reinforcement ratio, beam depth, shear span-to-depth ratio, and many others affect the shear performance of PFRC members (Lakavath et al. 2021; Xiang et al. 2022; Ababneh et al. 2017; Shaaban et al. 2021; Said et al. 2021; Hossain et al. 2023; Al Ghali et al. 2023; Atea 2017). These issues lead to the difficulty in designing reliable and universally acceptable shear strength prediction models. The second significant factor to consider is the variety of experimental data collected so far concerning the elements of PFRCs. In particular, the previously conducted research on various beam types, types of concrete, loading conditions, reinforcement arrangement, and the use of different materials, including lightweight concrete, pre-stressed concrete, recycled aggregate concrete, hybrid fiber systems, and sustainable concrete mixes has been carried out (Lakavath et al. 2021; Xiang et al. 2022; Ababneh et al. 2017; Shaaban et al. 2021; Said et al. 2021; Hossain et al. 2023; Al Ghali et al. 2023; Atea 2017). While these studies yield valuable data concerning specific cases, the variety of the test conditions creates problems in assessing their general applicability.

This diversity of experimental conditions creates additional challenges in developing reliable shear strength prediction models for PFRC members. Various studies have been performed on different beam types, concrete classes, loading scenarios, reinforcement configurations, and material combinations, including lightweight concrete, prestressed concrete, recycled aggregate concrete, hybrid fiber systems, and sustainable concrete mixes (Lakavath et al. 2021; Xiang et al. 2022; Ababneh et al. 2017; Shaaban et al. 2021; Said et al. 2021; Hossain et al.

2023; Al Ghali et al. 2023; Atea 2017). While these studies have considerably improved the current understanding of PFRC shear behavior, the applicability and reliability of existing shear strength prediction models across such a diverse range of structural configurations remain insufficiently established. Furthermore, most available analytical and code-based formulations have been developed and calibrated using steel fiber-reinforced concrete (SFRC) databases, whereas their predictive performance for PFRC beams has not yet been comprehensively evaluated.

An additional challenge arises from the implementation of design-oriented approaches such as fib Model Code 2010 (fib 2010), which require residual tensile strength parameters to represent the post-cracking behavior of fiber-reinforced concrete. These parameters are unavailable for most PFRC beam specimens reported in the literature, introducing considerable uncertainty into model implementation and limiting the direct evaluation of such approaches. Moreover, most available analytical and code-based shear strength formulations were originally developed or calibrated using steel fiber-reinforced concrete databases, and their predictive performance for PFRC beams has not yet been comprehensively established. Therefore, a systematic assessment based on a sufficiently broad PFRC database is required to clarify the applicability, accuracy, and limitations of the available prediction models.

Accordingly, the primary objective of this study is to evaluate the predictive performance of existing analytical, semi-empirical, and code-based shear strength models using a database of 98 PFRC beams without transverse reinforcement collected from published experimental studies. The evaluated formulations include the models proposed by Said et al. (2021), Zhang et al. (2020), Arslan et al. (2017), and Albidah and Abbas (2024), together with the fib Model Code 2010, DAFStB, and RILEM TC 162-TDF approaches. Particular attention is given to fib Model Code 2010 because the residual tensile strength parameters required for its implementation were unavailable for most specimens. To address this limitation, three alternative residual tensile strength estimation procedures were applied separately within the same fib Model Code 2010 shear resistance formulation, allowing the influence of residual tensile strength characterization on predicted shear capacity to be isolated. Model performance was evaluated using multiple statistical indicators and the distributions of safe and unsafe predictions. Through this framework, the study clarifies the applicability and limitations of existing shear strength models for PFRC beams and demonstrates the importance of PFRC-specific residual tensile strength characterization in design-oriented prediction procedures.

2. Experimental Database

2.1. Database compilation

An extensive database for polypropylene fiber reinforced concrete (PFRC) beams without transverse reinforcement has been prepared based on published experimental tests available in literature. The database con-

tains only those PFRC beams that were subjected to monotonic loading and failed in shear. Beam specimens that failed in any other mode besides shear failure have been omitted from the database. In addition, beams containing conventional transverse reinforcement within the investigated shear span were not considered in order to isolate the contribution of polypropylene fibers to shear resistance. The adopted selection criteria were intended to ensure a consistent database focused specifically on the assessment of fiber contribution to shear resistance in PFRC beams. The compiled database consists of 98 beam specimens collected from 28 independent experimental investigations (Altoubat et al. 2009; Navas et al. 2018; Al-Alawi and Mashrei 2022; Xiang et al. 2022; Altoubat et al. 2020; Uğur and Ünal 2022; Zhang et al. 2014; Ababneh et al. 2017; Arslan et al. 2017; Majdazadeh et al. 2006; Sahoo et al. 2015; Shaaban et al. 2021; Bastami 2019; Murad and Abdel-Jabbar 2022; Altoubat et al. 2012; Parmentier et al. 2012; Furlan Jr and de Hanai 1997; Hossain et al. 2023; Shoeib et al. 2023; Mori et al. 2017; Bajaj and Shrivastava 2011; Al-Khafaji and Harba 2023; Kamal et al. 2014; Al Ghali et al. 2023; Atea 2017; Rao 2011; Ozturk et al. 2017; Ketenci 2022). To ensure database consistency, duplicate inclusion of specimens reported in multiple publications was avoided whenever overlapping experimental datasets were identified. The collected specimens cover a wide range of concrete compressive strengths (f_c), fiber volume fractions (V_f), fiber aspect ratios (l_f/d_f), longitudinal reinforcement ratios (ρ), effective depths (d), and shear span-to-effective depth ratios (a/d). The wide range of material, geometric, reinforcement-related, and fiber parameters represented in the database provides a suitable basis for evaluating both the robustness and general applicability of existing shear strength prediction models for PFRC beams. For each specimen, the value of the compressive strength of concrete (f_c), effective depth (d), ratio of longitudinal steel reinforcement (ρ), ratio of shear span to effective depth (a/d), volume fraction of fibers (V_f), fiber aspect ratio (l_f/d_f), and shear strength (V_{exp}) were documented if they were available. This selection of parameters was based on their importance as influential factors of the shear performance of fiber-reinforced concrete members. Table 1 presents a summary of the experimental works used in the database along with the number of specimens per source.

Concrete compressive strength values stated in the original literature were not reported using the same type of specimens. Some reports were using cylinder compressive strengths while others were using cube compressive strengths. In order to unify all the reported compressive strength values, all were converted to the equivalent cylinder compressive strengths. In case of reports stating the cube compressive strength values, $f_c' = 0.80f_{cu}$ was adopted. Hence, the f_c' parameter used in this study denotes the equivalent cylinder compressive strength for all specimens. It is acknowledged that the relationship between cube and cylinder compressive strengths may vary depending on concrete strength level, aggregate characteristics, specimen geometry, and testing procedures. Nevertheless, the conversion factor $f_c' = 0.80f_{cu}$ was adopted as a practical standardization

approach to ensure a consistent compressive-strength parameter across the compiled database and facilitate a uniform comparison of the investigated prediction models. For sake of clarity, the full specimen-based database is provided in Appendix B. The database consists of specimen number, reference source, geometric information, reinforcement information, fiber information, equivalent

cylinder compressive strengths, failure modes and experimental shear strengths. All specimens were subjected to monotonic loading by either three point or four point bending schemes. In case where experimental shear strength values were not stated, they were calculated from ultimate load values based on the static equilibrium equations for the given loading and support condition.

Table 1. Experimental studies included in the database and corresponding number of specimens.

Reference	Number of Specimens
Altoubat et al. (2009)	11
Navas et al. (2018)	4
Al-Alawi and Mashrei (2022)	9
Xiang et al. (2022)	3
Altoubat et al. (2020)	1
Uğur and Ünal (2022)	1
Zhang et al. (2014)	1
Ababneh et al. (2017)	3
Arslan et al. (2017)	6
Majdzadeh et al. (2006)	6
Sahoo et al. (2015)	1
Shaaban et al. (2021)	3
Bastami (2019)	3
Murad and Abdel-Jabbar (2022)	3
Altoubat et al. (2012)	2
Parmentier et al. (2012)	4
Furlan Jr. and de Hanai (1997)	1
Hossain et al. (2023)	2
Shoeib et al. (2023)	3
Mori et al. (2017)	2
Bajaj and Shrivastava (2011)	12
Al-Khafaji and Harba (2023)	2
Kamal et al. (2014)	2
Al Ghali et al. (2023)	2
Atea (2017)	2
Rao et al. (2011)	6
Ozturk et al. (2017)	1
Ketenci (2022)	2
Total	98

2.2. Characteristics of the database

The developed database includes various types of geometrical, material, reinforcement, and fiber properties which affect the shear capacity of PFRC members. The value of the equivalent cylinder compressive strength (f_c) is between 13.85 MPa and 150 MPa, which shows that a variety of concrete strength grades are taken into account in the developed database. The values of the effective depth are varied from 85.25 mm up to 600 mm, whereas the values of the volume fraction of fibers vary from 0.10% to 3.00%. Besides, a great variability can be found in such parameters as the longitudinal reinforcement

ratio (ρ), shear span-to-depth ratio (a/d), and fiber aspect ratio. Such variability provides a good basis to test the validity of the existing shear capacity prediction models for PFRC beams. The ranges of the main parameters used in this study are listed in Table 2.

3. Shear Strength Prediction Models

This research investigates the predictability of available analytical and code-based shear strength prediction equations for PFRC beams without transverse reinforcement. The considered equations cover different method-

ologies for predicting shear resistance and involve different parameters that are related to the strength of concrete, the properties of fibers, the ratio of reinforcement, geometrical properties of the members, size effects, and post-cracking behavior. An important difference between the considered equations is the consideration of the contribution of the fibers. Whereas the analytical equations take into account the properties of the fibers through direct parameters like the volume fraction of fibers and as-

pect ratio of the fibers, the code-based equations use the residual tensile strength parameters, which try to capture the post-cracking behavior of the fiber-reinforced concrete. Attention is paid to fib Model Code 2010 (fib 2010). Three different residual tensile strength estimations were used since there are few reported experimental data about post-cracking tensile properties. The investigated models and the corresponding implementation procedures are described in the following sections.

Table 2. Summary of database parameter ranges.

Parameters	Minimum–Maximum Values
Concrete compressive strength, f'_c (MPa)	13.850 – 150.000
Effective depth, d (mm)	85.250 – 600.000
Shear span-to-depth ratio, a/d	0.580 – 4.870
Longitudinal reinforcement ratio, ρ	0.0020 – 0.0464
Fiber volume fraction, V_f (%)	0.100 – 3.000
Fiber aspect ratio, l_f/d_f	12.500 – 840.000
Fiber tensile strength, f_{tenf} (MPa)	375.000 – 685.000
Fiber elastic modulus, E_f (MPa)	3450.000 – 10500.000
Experimental shear capacity, V_{exp} (kN)	13.720 – 500.000

3.1. Overview of the evaluated models

The investigated models differ considerably in terms of their theoretical assumptions, required input parameters, and implementation procedures. The evaluated formulations include empirical, semi-empirical, and code-based approaches that have been proposed for different types of fiber-reinforced concrete (FRC) structures. Although some of the models were originally developed for steel fiber-reinforced concrete (SFRC), polyvinyl alcohol fiber-reinforced concrete (PVA-FRC), or FRC members in general, they were included in the present study to assess their applicability to PFRC beams without transverse reinforcement. For example, the model proposed by Said et al. (2021) was developed for PVA fiber-reinforced members, whereas the Albidah and Abbas (2024) formulation was calibrated using an SFRC database. Differences in fiber stiffness, bond characteristics, pull-out behavior, and post-cracking tensile response may therefore limit the direct transferability of these formulations to PFRC beams. Nevertheless, both models were included to evaluate their predictive performance when extended beyond their original calibration domains. The principal characteristics, required input parameters, and implementation requirements of the investigated models are summarized in Table 3. As shown in Table 3, the investigated formulations differ not only in their theoretical background but also in the way fiber contribution is incorporated into shear resistance predictions. While empirical and semi-empirical models generally utilize directly reported material, geometric, reinforcement-related, and fiber parameters, the code-based approaches rely on residual tensile strength parameters intended to represent the post-cracking be-

havior of fiber-reinforced concrete. This distinction is particularly important in the present study because residual tensile properties were unavailable for a large proportion of the specimens included in the compiled database. Consequently, additional estimation procedures were required for the implementation of the code-based formulations. The adopted procedures and associated assumptions are described in the following section.

It should be noted that the applicability information available from the original studies is summarized in Tables 2 and 3 through the reported parameter ranges of the present PFRC database and the original fiber types used during model development. However, explicit applicability limits were not consistently defined in several of the original formulations. Consequently, all selected models were evaluated using the same PFRC beam database in order to provide a consistent basis for comparison. In particular, the residual strength equations adopted in fib Model Code 2010 Approach I were originally developed using hook-end steel-fiber-reinforced concrete specimens with low fiber volume fractions. Since the objective of this study was to evaluate the predictive performance of existing models for PFRC beams, these original calibration ranges were not used as exclusion criteria during database compilation.

3.2. Model implementation and assumptions

Several of the investigated formulations require input parameters that were unavailable for a considerable portion of the experimental studies included in the database. In particular, residual flexural tensile strength parameters (f_{R1} and f_{R3}) and residual tensile strength parameters required by the code-based approaches (e.g.,

f_{Ftuk} , $f_{fctR,u}$, and $f_{Rk,A}$) were generally not reported in the original experimental publications. As a result, additional implementation procedures and assumptions were required to ensure the consistent application of all investigated models across the compiled database. Whenever sufficient information was available, the original formulations were implemented without modification. When required parameters were missing, they were estimated using procedures proposed in the corresponding references or commonly adopted in the literature. Consequently, the predictive performance of some

models reflects not only the accuracy of the shear resistance formulation itself but also the reliability of the adopted parameter-estimation procedure. This issue is particularly relevant for the code-based approaches, namely fib Model Code 2010 (fib 2010), DAfStB (2012), and RILEM TC 162-TDF (2003), whose implementation depends on residual tensile strength parameters used to represent the post-cracking behavior of fiber-reinforced concrete. The implementation procedures and assumptions adopted for these formulations are described in the following sections.

Table 3. Characteristics and implementation requirements of the evaluated shear strength prediction models.

Model	Original Fiber Type	Model Category	Main Parameters	Required Residual Parameter	Implementation Procedure	Additional Assumptions
Said et al. (2021)	PVA-FRC	Empirical	$f_c, V_f/\text{Fiber factor}$	No	Direct	None
Zhang et al. (2020)	PP-FRC	Empirical	$f_c, V_f/\text{Fiber factor}$	No	Direct	None
Arslan et al. (2017)	FRC	Semi-empirical	$f_c, V_f, \rho, a/d, d$	No	Direct	None
Albidah and Abbas (2024)	SFRC	Empirical	$f_c, V_f, \rho, a/d$	No	Direct	None
fib Model Code 2010 (fib 2010)	FRC	Code-based	f_c, ρ, d, f_{Ftuk}	f_{Ftuk}	Indirect residual strength characterization (3 approaches)	Residual strength estimation required
DAfStB (2012)	SFRC	Code-based	$f_c, d, f_{fctR,u}$	$f_{fctR,u}$	Indirect residual strength characterization	Residual strength estimation required
RILEM TC 162-TDF (2003)	SFRC	Code-based	$f_c, d, f_{Rk,A}$	$f_{Rk,A}$	Indirect residual strength characterization	$f_{Rk,A} = f_{spfc}$ assumption

3.2.1. fib Model Code 2010 (fib 2010)

The shear resistance of PFRC beams without transverse reinforcement was evaluated using the provisions of fib Model Code 2010 (fib 2010). The model accounts for the contribution of fibers through the post-cracking tensile behavior of fiber-reinforced concrete, represented by the characteristic ultimate residual tensile strength parameter, f_{Ftuk} . In fib Model Code 2010, f_{Ftuk} is derived from the residual flexural tensile strength parameters f_{R1} and f_{R3} obtained from standardized bending tests and therefore plays a central role in the prediction of shear resistance. Although fib Model Code 2010 (fib 2010) was originally developed and calibrated for steel fiber-reinforced concrete (SFRC), it was included in the present study to evaluate its applicability to PFRC beams. However, the residual strength parameters required for its implementation were unavailable for the majority of the specimens included in the compiled database. Consequently, the post-cracking tensile properties could not be determined directly from the original experimental studies. To overcome this limitation, three alternative residual tensile strength estimation procedures were adopted. These approaches were incorporated into the same fib Model Code 2010 formulation in order to systematically investigate the influence of residual tensile strength characterization on shear strength predictions. In Approaches I and II, the residual flexural tensile strengths f_{R1} and f_{R3}

were first estimated and subsequently converted into the characteristic ultimate residual tensile strength f_{Ftuk} according to fib Model Code 2010 provisions. In Approach III, the implementation procedure adopted by Lantsoght (2019) was followed. The adopted approaches are described in the following sections.

Approach I: Residual strength estimation using the model of Nzambi et al. (2022)

In the first approach, the residual flexural tensile strengths were estimated using the empirical model proposed by Nzambi et al. (2022). The model predicts the residual strength parameters based on fiber characteristics and pull-out behavior. The estimated residual flexural tensile strengths f_{R1} and f_{R3} were subsequently used to determine the characteristic ultimate residual tensile strength f_{Ftuk} according to the fib Model Code 2010 (fib 2010) procedure, as expressed in Eq. (1). The equations proposed by Nzambi et al. (2022) are provided in Appendix A.

$$f_{Ftuk} = f_{Fts} - \frac{w_u}{CMOD_3} (f_{Fts} - 0.5f_{R3} + 0.2f_{R1}) \quad (1)$$

where f_{Fts} is the serviceability residual tensile strength, f_{R1} and f_{R3} are residual flexural tensile strengths obtained at CMOD values of 0.5 mm and 2.5 mm, respectively, and w_u is the crack width at ultimate. Detailed definitions of all parameters are provided in Appendix A.

The calculated f_{Ftuk} values were then incorporated into the fib Model Code 2010 (fib 2010) shear resistance formulation to determine the predicted shear capacities of the specimens included in the database.

Approach II: Residual strength estimation using the model of Moraes-Neto et al. (2013)

In the second approach, the empirical equations proposed by Moraes-Neto et al. (2013) were used to estimate the residual flexural tensile strengths f_{R1} and f_{R3} . The corresponding equations are presented in Appendix A. The estimated residual flexural tensile strengths were subsequently converted into the characteristic ultimate residual tensile strength f_{Ftuk} according to the fib Model Code 2010 (fib 2010) procedure, as expressed in Eq. (1). The calculated f_{Ftuk} values were then incorporated into the fib Model Code 2010 (fib 2010) shear resistance formulation to determine the predicted shear capacities of the specimens included in the database.

Approach III: implementation based on the procedure adopted by Lantsoght (2019)

The third approach followed the implementation procedure adopted by Lantsoght (2019) for the evaluation of fiber-reinforced concrete beam databases. In this procedure, the splitting tensile strength of fiber-reinforced concrete was first estimated using the empirical relationship proposed by Thomas and Ramaswamy (2007). The corresponding equation is presented in Appendix A as Eq. (A13). The estimated splitting tensile strength was subsequently used to determine the residual tensile strength parameter $f_{fctR,u}$ according to the DAfStB (2012) procedure. Consistent with the methodology adopted by Lantsoght (2019), the calculated $f_{fctR,u}$ values were directly used as the f_{Ftuk} parameter required for the implementation of fib Model Code 2010 (fib 2010). The resulting f_{Ftuk} values were then incorporated into the fib Model Code 2010 shear resistance formulation to determine the predicted shear capacities of the specimens included in the database.

3.2.2. DAfStB (2012)

The DAfStB (2012) formulation evaluates the contribution of fibers to shear resistance through the residual tensile strength parameter $f_{fctR,u}$. Since direct measurements of the required residual tensile properties were unavailable for most specimens included in the database, the implementation procedure adopted by Lantsoght (2019) was followed. In this procedure, the splitting tensile strength of fiber-reinforced concrete was first estimated using the empirical relationship proposed by Thomas and Ramaswamy (2007). The corresponding splitting tensile strength equation proposed by Thomas and Ramaswamy (2007) is presented in Appendix A as Eq. (A13). The estimated splitting tensile strength was subsequently used to determine the residual tensile strength parameter $f_{fctR,u}$ according to the DAfStB (2012) procedure. The calculated $f_{fctR,u}$ values were then incorporated into the DAfStB shear resistance formulation to predict the shear capacities of the specimens included in the database. Consequently, the predictive

performance obtained for the DAfStB model reflects both the accuracy of the shear resistance formulation and the reliability of the adopted residual tensile strength estimation procedure.

3.2.3. RILEM TC 162-TDF (2003)

The RILEM TC 162-TDF (2003) recommendations evaluate the contribution of fibers to shear resistance through the residual flexural tensile strength parameter $f_{Rk,4}$. Since direct measurements of this parameter were unavailable for most specimens included in the database, the implementation procedure adopted by Lantsoght (2019) was followed. In this procedure, the splitting tensile strength of fiber-reinforced concrete was first estimated using the empirical relationship proposed by Thomas and Ramaswamy (2007). The corresponding splitting tensile strength equation proposed by Thomas and Ramaswamy (2007) is presented in Appendix A as Eq. (A13). Following the methodology adopted by Lantsoght (2019), the residual flexural tensile strength required by the RILEM formulation was assumed equal to the estimated splitting tensile strength, i.e., $f_{Rk,4} = f_{spfc}$. The resulting $f_{Rk,4}$ values were subsequently incorporated into the RILEM TC 162-TDF (2003) shear resistance formulation to determine the predicted shear capacities of the specimens included in the database. Consequently, the predictive performance obtained for the RILEM model reflects both the accuracy of the RILEM shear resistance formulation and the reliability of the adopted residual tensile strength estimation procedure.

The code-based formulations were implemented following the procedures and parameter values reported in their original recommendations and in the implementation framework adopted by Lantsoght (2019), as detailed in Appendix A. Accordingly, the prescribed partial safety factors were retained and were not uniformly set equal to 1.0. This implementation was adopted to evaluate each code-based formulation in accordance with its published framework rather than to derive uniformly unfactored nominal capacities. Therefore, the comparison with the empirical and semi-empirical models should be interpreted as an assessment of the predictive performance of the formulations under their respective implementation frameworks.

4. Statistical Evaluation Methodology

The predictive performance of the investigated shear strength models was evaluated by comparing the experimentally measured shear capacities (V_{exp}) with the corresponding predicted values (V_{pred}). For each specimen, the experimental-to-predicted shear strength ratio (V_{exp}/V_{pred}) was calculated and used as the primary indicator of model performance. A ratio equal to unity indicates perfect agreement between experimental and predicted values, whereas values greater than and less than unity indicate conservative and unconservative predictions, respectively. Since unconservative predictions may lead to unsafe structural assessments, particular attention was given to cases where $V_{exp}/V_{pred} < 1.0$.

To provide a comprehensive evaluation of model performance, the mean value (μ), standard deviation (σ), coefficient of variation (COV), mean absolute error (MAE), root mean square error ($RMSE$), and coefficient of determination (R^2) were calculated. These statistical indicators were used to assess prediction accuracy, consistency, scatter, and overall reliability.

The mean value was determined as:

$$\mu = \frac{1}{n} \sum_{i=1}^n \left(\frac{V_{exp}}{V_{pred}} \right)_i \quad (2)$$

where n denotes the total number of specimens included in the database.

The standard deviation was calculated using:

$$\sigma = \sqrt{\frac{\sum_{i=1}^n \left[\left(\frac{V_{exp}}{V_{pred}} \right)_i - \mu \right]^2}{n-1}} \quad (3)$$

The coefficient of variation was obtained from:

$$COV = \frac{\sigma}{\mu} \quad (4)$$

The mean absolute error was calculated as:

$$MAE = \frac{1}{n} \sum_{i=1}^n |V_{exp,i} - V_{pred,i}| \quad (5)$$

The root mean square error was determined from:

$$RMSE = \sqrt{\frac{1}{n} \sum_{i=1}^n (V_{exp,i} - V_{pred,i})^2} \quad (6)$$

The coefficient of determination (R^2) was obtained as the square of the Pearson correlation coefficient (r) between the experimental and predicted shear strengths:

$$R^2 = r^2 \quad (7)$$

where r denotes the Pearson correlation coefficient.

Models exhibiting mean values close to unity, low coefficients of variation, low prediction errors (MAE and $RMSE$), high coefficients of determination (R^2), and a limited proportion of unconservative predictions were considered to provide more accurate and reliable shear strength estimates. The investigated formulations were subsequently compared based on the collective interpretation of these statistical indicators.

5. Results and Discussion

The predictive capabilities of the selected shear strength models were evaluated using the assembled database of polypropylene fiber-reinforced concrete (PFRC) beams without transverse reinforcement. The investigated models included empirical, semi-empirical, mechanics-based, and code-based formulations available in the literature. In addition to the analytical models proposed by previous researchers (Arslan et al. (2017); Albidah and Abbas (2024); Said et al. (2021); Zhang et al. (2020)), the RILEM TC 162-TDF (2003), DAfStB (2012),

and fib Model Code 2010 (fib 2010) approaches were also assessed. For the latter models, several assumptions and parameter estimation procedures were adopted owing to the limited availability of residual tensile strength data in the collected experimental database.

5.1. Statistical evaluation of existing shear strength models

The statistical performance of the investigated shear strength prediction models is summarized in Table 4, while the corresponding MAE values are illustrated in Fig. 1. The results reveal substantial differences among the evaluated formulations in terms of prediction accuracy, consistency, and overall reliability. As shown in Table 4 and Fig. 1, the MAE values vary considerably, ranging from 31.53 kN to 140.23 kN, indicating that the predictive capability of existing shear strength models for PFRC beams without transverse reinforcement differs significantly.

Among the directly applicable analytical and semi-empirical formulations, the model proposed by Arslan et al. (2017) provided the most balanced overall performance. The model achieved a mean experimental-to-predicted shear strength ratio close to unity (1.029), together with relatively low prediction errors ($MAE = 32.22$ kN and $RMSE = 61.22$ kN). These results indicate a good balance between prediction accuracy and consistency over a wide range of PFRC beam configurations. The satisfactory performance of the model may be attributed to its consideration of several parameters known to influence shear behavior, including concrete compressive strength, longitudinal reinforcement ratio, shear span-to-depth ratio, size effect, and fiber contribution.

The formulation proposed by Albidah and Abbas (2024) achieved the highest coefficient of determination ($R^2 = 0.784$) and the lowest coefficient of variation ($COV = 0.337$) among all investigated models, demonstrating a strong ability to capture the overall trend of the experimental database. However, the corresponding mean value of 0.828 indicates a tendency toward unconservative predictions. Although the prediction scatter remained relatively low, the observed bias suggests that the model may overestimate the shear capacity of some PFRC beams.

In contrast, the models proposed by Zhang et al. (2020) and Said et al. (2021) exhibited less satisfactory predictive performance. The formulation of Zhang et al. (2020) produced a mean value of 0.711 and relatively high prediction errors, whereas the model proposed by Said et al. (2021) yielded highly conservative predictions with a mean value of 1.999 and the lowest coefficient of determination ($R^2 = 0.225$) among the directly applicable formulations. These results indicate a limited ability of both models to consistently predict the shear strength of the PFRC beams included in the database.

Different trends were observed for the code-based formulations. In general, the predictive performance of these approaches was strongly influenced by the procedures adopted to characterize the residual tensile properties of fiber-reinforced concrete. The RILEM TC 162-

TDF (2003) formulation produced relatively low prediction errors and a competitive level of accuracy, whereas the DAFStB (2012) model exhibited the largest prediction errors among all investigated formulations. The performance of the fib Model Code 2010 (fib 2010) implementations also varied considerably depending on the adopted residual tensile strength estimation procedure. A more detailed discussion of the code-based approaches is presented in Sections 5.2 and 5.3.

Overall, the results presented in Table 4 and Fig. 1 indicate that the predictive performance of existing shear strength models for PFRC beams varies substantially. Models incorporating the combined effects of concrete strength, reinforcement ratio, member geometry, and fiber contribution generally provided more accurate and

consistent predictions. Furthermore, the observed variability among the code-based formulations suggests that the characterization of post-cracking tensile behavior plays a critical role in the reliability of shear strength predictions for PFRC members.

In addition to the statistical indicators presented in Table 4, the distribution of experimental-to-predicted shear strength ratios was also examined to assess model reliability. Table 5 summarizes the distribution of safe and unsafe predictions based on the $V_{exp}/V_{pred} = 1.00$ criterion for the investigated models, while Fig. 2 illustrates the relationships between experimental and predicted shear strengths. Together, these results provide additional insight into prediction accuracy, bias, and overall model reliability.

Table 4. Statistical performance of the investigated shear strength prediction models.

Model	Mean	COV	RMSE (kN)	R ²	MAE (kN)
Arslan et al. (2017)	1.029	0.367	61.220	0.592	32.220
Albidah and Abbas (2024)	0.828	0.337	58.110	0.784	39.730
Said et al. (2021)	1.999	0.574	93.670	0.225	54.840
Zhang et al. (2020)	0.711	0.537	86.070	0.536	63.330
RILEM TC 162-TDF (2003)	1.281	0.584	76.380	0.400	32.600
DAFStB (2012)	0.458	0.561	202.470	0.333	140.230
FibMC2010 Approach I	0.713	0.608	110.000	0.390	73.540
FibMC2010 Approach II	1.046	0.608	75.470	0.389	37.560
FibMC2010 Approach III	1.139	0.566	72.460	0.430	31.530

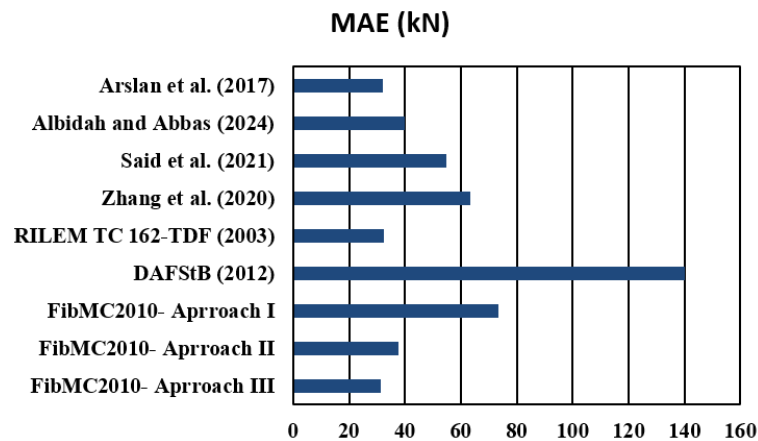


Fig. 1. Comparison of MAE values obtained from the investigated shear strength prediction models.

Table 5. Distribution of experimental-to-predicted shear strength ratios (V_{exp}/V_{pred}) for the investigated models.

Model	V_{exp}/V_{pred}		Unsafe (%)	Safe (%)
	< 1.00	≥ 1.00		
Arslan et al. (2017)	53	45	54	46
Albidah and Abbas (2024)	78	20	80	20
Said et al. (2021)	14	84	14	86
Zhang et al. (2020)	85	13	87	13
RILEM TC 162-TDF (2003)	29	69	30	70
DAFStB (2012)	93	5	95	5
FibMC2010-Approach I	86	12	88	12
FibMC2010-Approach II	68	30	69	31
FibMC2010-Approach III	60	38	61	39

The columns “< 1.00” and “≥ 1.00” represent the number of unsafe and safe predictions, respectively.

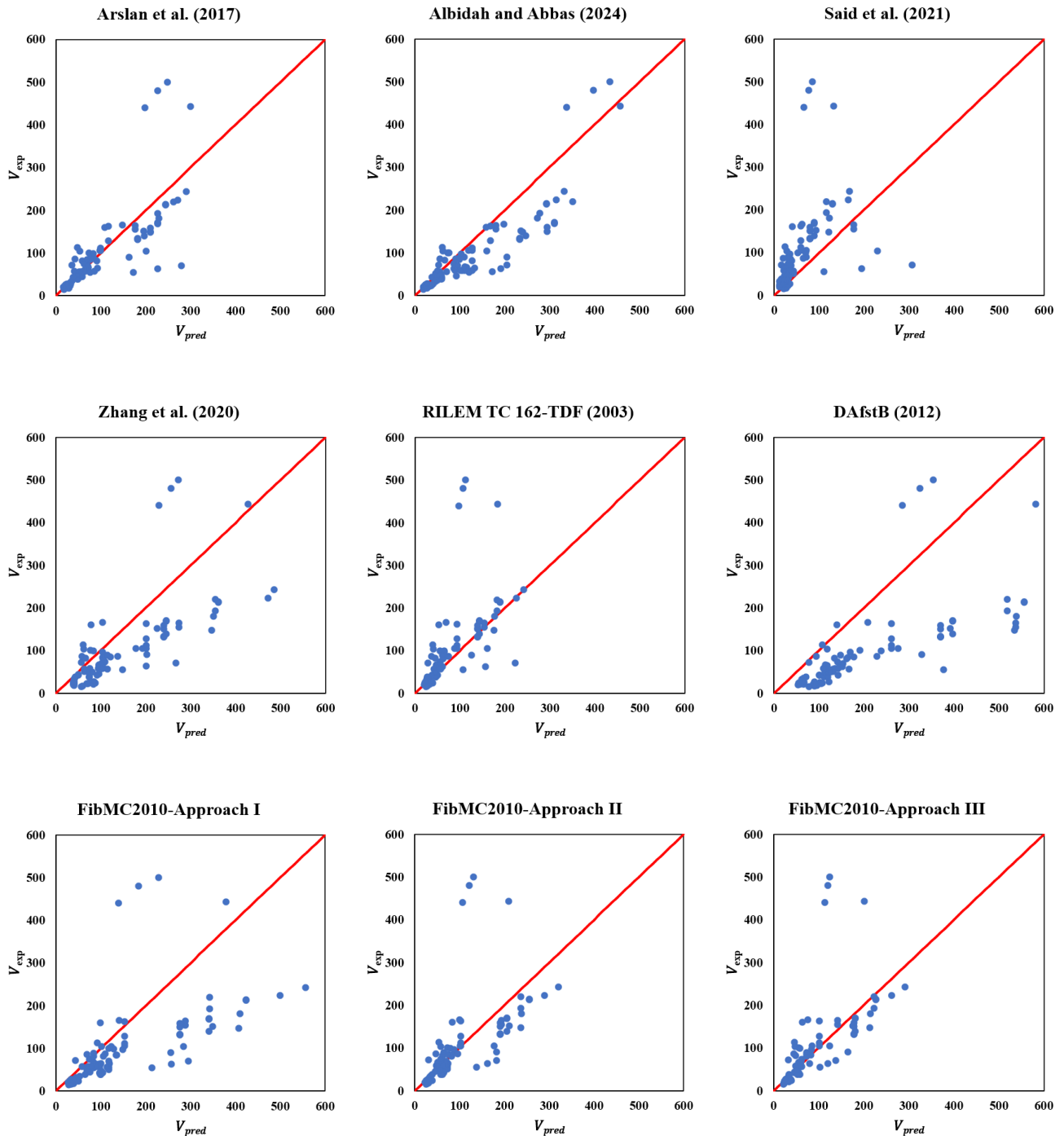


Fig. 2. Experimental-versus-predicted shear strength relationships for the investigated shear strength prediction models.

The scatter plots presented in Fig. 2 further highlight the differences among the investigated formulations. Models exhibiting lower prediction errors and mean V_{exp}/V_{pred} ratios close to unity generally produced data points more closely distributed around the line of perfect agreement. In contrast, models with larger prediction errors showed greater scatter and more pronounced deviations from the experimental results.

Inspection of the experimental-versus-predicted shear strength relationships shown in Fig. 2 also provides insight into possible prediction bias. The Said et al. (2021) model generally produced conservative pre-

dictions, with deviations becoming more pronounced for specimens with relatively high experimental shear capacities. In contrast, the DAfStB (2012) model exhibited a strong tendency to overestimate shear strength, consistent with its low mean experimental-to-predicted ratio and high proportion of unsafe predictions. The formulations proposed by Arslan et al. (2017) and fib Model Code 2010 Approach III showed a comparatively more balanced distribution of predictions across the investigated shear-strength range. These observations complement the statistical indicators presented in Table 4.

5.2. Performance of the code-based shear models

The code-based formulations exhibited markedly different levels of predictive performance when applied to the compiled PFRC beam database. Since the implementation of both RILEM TC 162-TDF (2003) and DAFStB (2012) required estimated residual tensile strength parameters, the obtained results reflect not only the accuracy of the underlying shear resistance formulations but also the reliability of the adopted parameter-estimation procedures. Among the investigated code-based approaches, the RILEM TC 162-TDF (2003) model demonstrated a relatively balanced predictive performance. Although a certain degree of prediction scatter was observed, the overall error levels remained comparable to those obtained from the better-performing analytical formulations. As shown in Table 5, the RILEM formulation produced a substantially lower percentage of unsafe predictions than the DAFStB model, suggesting that the adopted implementation procedure can provide reasonable estimates of PFRC beam shear strength despite the absence of experimentally determined residual flexural tensile strength parameters. In contrast, the DAFStB (2012) formulation consistently exhibited the weakest performance among the investigated code-based approaches. As indicated by the safety classification results presented in Table 5, the DAFStB model produced the highest proportion of unsafe predictions (95%) among the investigated formulations. This behaviour indicates a strong tendency to overestimate the shear resistance of PFRC beams and may be attributed to both the characteristics of the DAFStB shear resistance formulation and the residual tensile strength estimation procedure adopted for its implementation. Since neither the shear formulation nor the parameter-estimation framework was originally developed for PFRC members, additional uncertainty may have been introduced when the approach was applied to the present database. It is difficult to isolate the contribution of individual parameters within the DAFStB procedure. However, part of the observed discrepancy may stem from the estimation of the splitting tensile strength using the Thomas and Ramaswamy model, which was originally calibrated using steel-fiber-reinforced concrete data. In addition, the orientation and residual-strength factors embedded in the DAFStB formulation were developed for SFRC systems and may not fully represent the fiber–matrix interaction, pull-out behavior, and post-cracking response of polypropylene fibers. Consequently, both the residual tensile strength estimation procedure and the original DAFStB parameter calibration are likely contributors to the high proportion of unsafe predictions observed for PFRC beams. The contrasting performances of the RILEM TC 162-TDF (2003) and DAFStB (2012) approaches highlight the importance of residual tensile strength characterization in code-based shear strength prediction. Although both formulations attempt to incorporate the post-cracking contribution of fibers through residual strength parameters, their predictive capabilities differed substantially when applied to the PFRC beam database considered in this study. These findings suggest that the reliability of code-based shear strength

predictions depends not only on the shear resistance formulation itself but also on the procedure used to characterize the residual tensile behaviour of the material. Owing to its particular relevance to this issue, the performance of fib Model Code 2010 (fib 2010) is discussed separately in the following section.

5.3. Sensitivity of fib Model Code 2010 to residual strength estimation procedures

The statistics for the three fib Model Code 2010 (fib 2010) approaches are reported in Table 6 below. While the same shear resistance equation was utilized in all cases, there were noticeable disparities in the results produced through the three different methods of estimating residual tensile strength. Given that the key factor that distinguishes between the considered approaches is their representation of tensile behavior after cracking, the comparison will be beneficial in demonstrating the influence of residual tensile strength estimations on the accuracy of the predictions under fib Model Code 2010. The fib approach relying on the Nzambi et al. (2022) model (Approach I) was the least accurate one among the three approaches. According to Table 6, the model had the lowest mean experimental-to-predicted ratio of 0.713 as well as the highest RMSE (110.00 kN) and MAE (73.54 kN) values out of all the fib-based approaches. Moreover, the coefficient of variation (COV = 0.608) was quite high. These results suggest that the adopted residual tensile strength estimation procedure was less compatible with the PFRC beam database considered in this study. Part of this discrepancy may be attributed to the fact that the residual-strength relationships proposed by Nzambi et al. (2022) were originally calibrated using hook-end steel-fiber-reinforced concrete specimens. Differences in elastic modulus, fiber–matrix bond characteristics, pull-out mechanisms, and crack-bridging behavior between steel fibers and polypropylene fibers may therefore contribute to the reduced compatibility of the resulting residual tensile strength estimates when applied to PFRC beams.

Significant improvement was made possible through the use of the approach described by Moraes-Neto et al. (2013) (Approach II). In this case, the accuracy of prediction of shear strength was greatly improved, which allowed for the attainment of the mean ratio equal to 1.046, which is rather close to unity. In addition, RMSE and MAE were significantly reduced when compared to Approach I. This implies that the values of residual tensile properties estimated through the application of the Moraes-Neto et al. (2013) approach can be considered as being more appropriate for the utilization of the fib Model Code 2010 in the analysis of the given PFRC specimens. The most successful implementation of the fib Model Code 2010 was attained through the application of the procedure suggested by Lantsoght (2019) (Approach III). According to Table 6, this approach allowed for the reduction of MAE to 31.53 kN, RMSE to 72.46 kN, COV to 0.566 and provided R^2 of 0.430, which was higher than for other approaches. These results indicate a more accurate and consistent representation of the post-cracking tensile contribution incorporated into the fib Model Code 2010 framework.

The results of the safety classifications provided in Table 5 confirm these findings. The percentage of the unsafe predictions ($V_{exp}/V_{pred} < 1.00$) was reduced from 88% in Approach I to 69% in Approach II and 61% in Approach III, thus reflecting the progressively improving reliability of predictions with the change of the applied residual tensile strength estimation procedure. Among all considered approaches, the lowest percentage of unsafe predictions and the highest percentage of safe predictions were obtained for Approach III. This confirms the significant impact of the residual tensile strength estimation procedure on the reliability of fib Model Code 2010 shear strength predictions. A particularly important finding is that considerable differences were obtained between the three approaches even though the same shear resistance formula of the fib Model Code 2010 (fib 2010) was used. The main difference between the three approaches being in the method used to estimate the residual tensile behavior of fiber reinforced concrete, the discrepancies in terms of prediction accu-

racy could be largely attributed to the method chosen to characterize the residual tensile strength. This implies that the prediction capabilities of the fib Model Code 2010 are extremely dependent on the method used to estimate the post cracking behavior of fiber reinforced concrete especially where no experimental residual strength is available. It must be acknowledged that the residual strength estimation procedures employed in Approaches I, II and III were initially developed through databases not identical to the one used in this experiment for polypropylene fiber reinforced concrete beams. Some of the discrepancies in terms of prediction accuracy therefore can be explained by differences in fiber type, bond behavior and post cracking behavior. However, it is clear from the results obtained that a good method to characterize residual tensile properties is important in achieving accurate fib Model Code 2010 predictions. The findings further highlight the need for residual tensile strength estimation procedures specifically calibrated for polypropylene fiber-reinforced concrete.

Table 6. Statistical comparison of fib Model Code 2010 implementations.

Approach	Mean	COV	RMSE (kN)	R ²	MAE (kN)
Approach I (Nzambi et al. (2022))	0.713	0.608	110.000	0.390	73.540
Approach II (Moraes-Neto et al. (2013))	1.046	0.608	75.470	0.389	37.560
Approach III (Lantsoght (2019))	1.139	0.566	72.460	0.430	31.530

5.4. Influence of shear span-to-depth ratio and concrete strength

To further examine the influence of database heterogeneity on model performance, additional subgroup analyses were conducted based on the shear span-to-effective depth ratio (a/d) and concrete compressive strength. The specimens were classified into two groups according to a/d (< 2.5 and ≥ 2.5) and cylinder compressive strength ($f'_c < 50$ MPa and $f'_c \geq 50$ MPa). The corresponding statistical results are presented in Table 7. Particular attention was given to the performance of the three fib Model Code 2010 implementations, since the primary objective of this study was to investigate the influence of alternative residual tensile strength estimation procedures on shear strength prediction accuracy.

It can be observed from the results above that the predictive performance of fib Model Code 2010 is affected by the ratio between shear span and depth of the member. Where $a/d \geq 2.5$, Approach III gave the best prediction out of the three approaches under consideration, having a V_{exp}/V_{pred} of 1.006 and MAE of 19.60 kN. Approach II and I had MAE of 27.26 kN and 77.16 kN, respectively. However, where $a/d < 2.5$, the prediction performance of all three approaches worsened with an increase in MAE to 45.03 kN, 49.20 kN, and 69.44 kN, respectively. From the results, it is clear that the predictive performance of fib Model Code 2010 decreases in cases where the a/d ratio is relatively low.

A comparison of the three residual tensile strength estimation procedures reveals that the relative ranking of

the fib Model Code 2010 implementations remains essentially unchanged across both a/d categories. Approach III consistently exhibited the lowest prediction errors and mean values closest to unity, whereas Approach I produced the largest deviations from the experimental results. Approach II generally provided intermediate performance between the other two procedures. This observation confirms that the procedure adopted for residual tensile strength characterization has a substantial effect on fib Model Code 2010 predictions regardless of member geometry.

The influence of concrete compressive strength was also investigated by separating the database into specimens with $f'_c < 50$ MPa and $f'_c \geq 50$ MPa. Although the effect of concrete strength was less pronounced than that of the a/d ratio, the prediction accuracy of the fib Model Code 2010 implementations generally decreased for higher-strength concretes. For Approach III, the MAE increased from 24.83 kN for specimens with $f'_c < 50$ MPa to 56.10 kN for specimens with $f'_c \geq 50$ MPa. Similarly, the mean V_{exp}/V_{pred} ratio increased from 1.075 to 1.375. Comparable trends were observed for Approaches I and II, indicating that the investigated residual tensile strength estimation procedures tend to produce increasingly conservative predictions as concrete compressive strength increases.

Similar tendencies were also observed for several of the other investigated models. In particular, the Arslan et al. and RILEM formulations generally exhibited lower prediction errors for specimens with $a/d \geq 2.5$ than for those with $a/d < 2.5$. Nevertheless, the subgroup anal-

yses consistently demonstrated that the conclusions obtained from the full database remained unchanged. Among the three fib Model Code 2010 implementations, the procedure adopted from Lantsoght (Approach III) provided the most reliable and consistent predictions, whereas the implementation based on the residual

strength equations proposed by Nzambi et al. (Approach I) exhibited the weakest overall predictive performance. These findings further demonstrate the critical importance of residual tensile strength characterization in the implementation of fib Model Code 2010 for PFRC beams.

Table 7. Statistical performance of the investigated models for different a/d ratios and concrete-strength ranges.

Model	Arslan et al. (2017)	Albidah and Abbas (2024)	Said et al. (2021)	Zhang et al. (2020)	RILEM TC 162-TDF (2003)	DAFStb (2012)	FibMC2010 Approach I	FibMC2010 Approach II	FibMC2010 Approach III
$a/d \geq 2.5$									
Mean	1.045	0.901	1.794	0.696	1.158	0.415	0.594	0.887	1.006
COV	0.326	0.316	0.421	0.447	0.378	0.380	0.348	0.355	0.378
RMSE (kN)	31.859	49.692	45.411	85.099	22.682	194.465	112.509	37.187	28.185
R^2	0.885	0.891	0.691	0.872	0.877	0.832	0.884	0.878	0.877
MAE (kN)	21.517	32.541	35.600	59.224	14.343	138.600	77.164	27.259	19.595
$a/d < 2.5$									
Mean	1.012	0.746	2.230	0.727	1.420	0.506	0.847	1.226	1.289
COV	0.414	0.336	0.648	0.621	0.687	0.653	0.669	0.681	0.644
RMSE (kN)	82.694	66.358	127.910	87.162	108.839	211.147	107.092	102.809	101.432
R^2	0.528	0.789	0.102	0.543	0.286	0.193	0.321	0.305	0.389
MAE (kN)	44.313	47.863	76.580	67.961	53.245	142.063	69.436	49.201	45.031
$f'_c < 50$									
Mean	1.019	0.856	1.830	0.710	1.210	0.431	0.668	0.950	1.075
COV	0.368	0.335	0.575	0.529	0.527	0.538	0.588	0.545	0.494
RMSE (kN)	52.826	61.269	71.381	86.204	51.827	222.247	113.805	54.983	48.309
R^2	0.632	0.768	0.278	0.614	0.530	0.416	0.538	0.571	0.601
MAE (kN)	29.665	40.263	46.219	62.766	24.640	152.915	77.961	32.674	24.834
$f'_c \geq 50$									
Mean	1.066	0.727	2.618	0.712	1.540	0.556	0.876	1.398	1.375
COV	0.369	0.312	0.492	0.579	0.674	0.575	0.614	0.630	0.680
RMSE (kN)	85.208	44.673	149.203	85.592	131.809	100.941	94.750	124.467	126.280
R^2	0.928	0.958	0.749	0.751	0.683	0.718	0.674	0.698	0.702
MAE (kN)	41.574	37.789	86.429	65.375	61.801	93.697	57.315	55.468	56.104

5.5. Implications for PFRC shear strength prediction

It can be clearly seen from the results of the current investigation that the performance of the shear strength prediction of existing models of PFRC beams highly depends on the parameters that describe the role of fibers in providing resistance to shear forces. In general, the models, which take into account the influence of various factors such as concrete strength, reinforcement ratio, geometrical dimensions of members and the influence of fibers, provided better predictions than models using fewer parameters. These results mean that an accurate evaluation of the shear strength of PFRC beams cannot be done by means of simplified descriptions of fiber contribution. In particular, an important point is the use of code-based approaches. While these models attempt to capture the post-cracking behavior of fiber reinforced concrete based on the parameters of the residual tensile strength of fibers, it has been shown that the accuracy of such prediction heavily relies on the methodology used to define the above properties. Such an influence was seen to be the strongest for the case of fib Model Code

2010 (fib 2010), with a considerable variation in the accuracy of prediction being achieved even while using the same shear resistance equation. This means that the way of estimating the residual tensile strength plays a decisive role in the accuracy of code-based prediction of shear strength. In view of the design practice, this can be considered to mean that currently available formulations for the steel fiber reinforced concrete could be employed to provide reasonably accurate estimation of PFRC shear resistance if proper methodology is applied to estimate the post-cracking behavior of tensile strength. Yet, the difference between the methods evaluated suggests that one should be careful when applying residual tensile strength estimation techniques based on the SFRC database to the PFRC members. This limitation is largely associated with the fundamental differences between polypropylene and steel fibers. Compared with steel fibers, polypropylene fibers generally exhibit a lower elastic modulus, different pull-out behavior, reduced bond stiffness, and distinct crack-bridging mechanisms. As a result, residual tensile strength relationships calibrated using SFRC databases may not accurately rep-

resent the post-cracking behavior of PFRC members. Overall, the results emphasize that improvements in the characterization of post-cracking tensile properties may have a greater influence on prediction reliability than modifications to the shear resistance equations themselves. Future research should therefore focus on developing PFRC-specific residual tensile strength characterization procedures and design-oriented prediction frameworks capable of more accurately representing the contribution of polypropylene fibers to shear resistance.

6. Conclusions

This study evaluated the predictive performance of existing analytical and code-based shear strength prediction models using a database of 98 polypropylene fiber-reinforced concrete (PFRC) beams without transverse reinforcement collected from the literature. Particular attention was given to the implementation of fib Model Code 2010 (fib 2010) and the influence of different residual tensile strength estimation procedures on prediction accuracy. Based on the results obtained, the following conclusions can be drawn:

- Important distinctions have been found between the analyzed models of shear strength prediction, suggesting that the choice of the model used is an important factor in estimating the shear strength of PFRC.
- Among the applicable forms of analysis or semi-empirical equations, the model suggested by Arslan et al. (2017) performed relatively evenly across all aspects of predictions, whereas the model suggested by Albidah and Abbas (2024) showed a good capability to represent the trend of the experimental database.
- Performance of the code-based methods was quite variable. The RILEM TC 162-TDF (2003) model per-

formed reasonably well in terms of prediction accuracy while the DAfStB (2012) method performed worst among all for the constructed PFRC dataset.

- The examination of the distribution of the ratio of experimentally determined to predicted shear strengths demonstrated considerable variability in the reliability of the model predictions, particularly in terms of the percentage of unsafe predictions.
- The predictive capability of fib Model Code 2010 (fib 2010) was found to be highly sensitive to the adopted residual tensile strength estimation procedure. Despite the use of the same underlying shear resistance formulation, substantial variations in prediction accuracy were observed among the investigated implementations.
- The results demonstrate that reliable characterization of post-cracking tensile behavior is essential for accurate code-based shear strength prediction. Among the investigated alternatives, the implementation based on the procedure adopted by Lantsoght provided the most consistent overall performance.
- Additional subgroup analyses indicated that the predictive performance of most investigated models was affected by the shear span-to-depth ratio. Lower prediction errors were generally observed for specimens with $a/d \geq 2.5$, while prediction accuracy tended to decrease for members with smaller a/d ratios.

Overall, the findings highlight the importance of residual tensile strength characterization in the assessment of PFRC shear resistance and demonstrate the need for PFRC-specific material characterization procedures and design-oriented prediction approaches. Future research should focus on developing residual tensile strength estimation methods specifically calibrated for polypropylene fiber-reinforced concrete and on expanding the available database of experimentally determined post-cracking tensile properties.

Appendix A. Prediction procedures for shear resistance of FRC beams

A.1. fib Model Code 2010 (fib 2010)

For the purposes of this study, the shear resistance was evaluated using the provisions of fib Model Code 2010 (fib 2010). The corresponding shear resistance was determined as:

$$V_{Rd,F} = \frac{0.18}{\gamma_c} k \left[100\rho \left(1 + 7.5 \frac{f_{Ftuk}}{f_{ctk}} \right) f_{ck} \right]^{1/3} b_w d \quad (A1)$$

The size effect factor was determined from:

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \quad (A2)$$

The residual tensile strength corresponding to the serviceability limit state was obtained as:

$$f_{Fts} = 0.45 f_{R1} \quad (A3)$$

The residual tensile strength corresponding to the ultimate limit state was calculated as:

$$f_{Ftuk}(w_u) = f_{Fts} - \frac{w_u}{2.5} [f_{Fts} - (0.5 f_{R3} + 0.2 f_{R1})] \quad (A4)$$

In accordance with fib Model Code 2010, the crack width at ultimate was taken as $w_u = 1.5$ mm.

The mean tensile strength of concrete was determined from:

$$f_{ctm} = 0.3f_{ck}^{2/3} \quad (A5)$$

For the purpose of the present comparative assessment, the characteristic tensile strength was assumed equal to the mean tensile strength ($f_{ctk} = f_{ctm}$) in all implementations. This assumption was adopted uniformly for all specimens and all fib Model Code 2010 implementations in order to ensure consistency among the investigated residual tensile strength estimation procedures and to isolate their influence on the resulting shear strength predictions.

In these equations, $V_{Rd,F}$ denotes the shear resistance provided by SFRC; k is the size-effect factor; ρ is the longitudinal reinforcement ratio; f_{Ftuk} is the characteristic ultimate residual tensile strength; f_{ctm} and f_{ctk} are the mean and characteristic tensile strengths of concrete, respectively; f_{ck} is the characteristic compressive strength of concrete; b_w is the web width; and d is the effective depth. The parameters f_{Fts} and f_{Ftu} represent the residual tensile strengths associated with the serviceability and ultimate limit states, respectively.

The residual flexural tensile strengths f_{R1} and f_{R3} , required in Eqs. (A3) and (A4), were determined using the procedures described in Appendices A.2 and A.3. Alternatively, the procedure presented in Appendix A.4 was used to estimate the residual tensile strength parameter required for the fib Model Code 2010 method.

A.2. Residual strength equations proposed by Nzambi et al. (2022)

The residual flexural tensile strengths f_{R1} and f_{R3} were estimated using the empirical equations proposed by Nzambi et al. (2022). The maximum fiber tensile stress was first calculated as:

$$\sigma_{f,max} = 4\lambda f_{ct} \left(\frac{l_f \phi}{d_f} \right) \quad (A6)$$

The residual flexural tensile strength corresponding to CMOD = 0.5 mm was determined from:

$$f_{\{R1\}} = 1.121(\sigma_{f,max} V_f)^{\{0.91\}} \quad (A7)$$

The residual flexural tensile strength corresponding to CMOD = 2.5 mm was determined from:

$$f_{\{R3\}} = 1.150(\sigma_{f,max} V_f)^{\{0.76\}} \quad (A8)$$

In this equation, $\sigma_{f,max}$ represents the tensile strength of the fiber, λ is the coefficient relating to the pull out strength of the fiber, ϕ represents the coefficient due to the fiber matrix interaction, f_{ct} is the tensile strength of the concrete, V_f represents the fiber volume fraction, l_f is the fiber length and d_f is the fiber diameter. The resulting values of f_{R1} and f_{R3} were subsequently used in Eqs. (A3) and (A4).

A.3. Residual strength equations proposed by Moraes-Neto et al. (2013)

The residual flexural tensile strengths f_{R1} and f_{R3} were also estimated using the equations proposed by Moraes-Neto et al. (2013). The fiber reinforcement index was calculated as:

$$RI = V_f \left(\frac{l_f}{d_f} \right) \quad (A9)$$

The residual flexural tensile strength corresponding to CMOD = 0.5 mm was calculated as:

$$f_{R1} = 7.5(RI)^{0.8} \quad (A10)$$

The residual flexural tensile strength corresponding to CMOD = 2.5 mm was calculated as:

$$f_{R3} = 6.0(RI)^{0.7} \quad (A11)$$

where RI is the fiber reinforcement index, V_f is the fiber volume fraction, l_f is the fiber length, and d_f is the fiber diameter. The resulting values of f_{R1} and f_{R3} were subsequently used in Eqs. (A3) and (A4).

A.4. Thomas model and Lantsoght-based procedure

A third approach was adopted following the methodology proposed by Lantsoght. First, the fiber factor was calculated as:

$$F = V_f \left(\frac{l_f}{d_f} \right) r_f \quad (\text{A12})$$

The splitting tensile strength of fiber-reinforced concrete was estimated as:

$$f_{spfc} = 0.63\sqrt{f_{cu}} + 0.288F\sqrt{f_{cu}} + 0.052F \quad (\text{A13})$$

where F is the fiber factor, V_f is the fiber volume fraction, l_f is the fiber length, d_f is the fiber diameter, r_f is the fiber bond factor, f_{spfc} is the splitting tensile strength of steel fiber-reinforced concrete, and f_{cu} is the cube compressive strength of fiber-reinforced concrete.

Since all database compressive strengths were standardized to equivalent cylinder compressive strengths, the corresponding cube compressive strengths required by Eq. (A13) were obtained using the inverse of the adopted conversion relationship ($f_{cu} = f'_c/0.80$). Therefore, Eq. (A13) was implemented using the compressive strength definition originally required by the Thomas and Ramaswamy formulation.

The splitting tensile strength obtained from Eq. (A13) was subsequently employed in the DAfStB procedure described in Appendix A.5. Following the approach proposed by Lantsoght, the residual tensile strength obtained from the DAfStB model was assumed to represent the characteristic ultimate residual tensile strength required by the fib Model Code 2010 procedure.

A.5. DAfStB (2012)

The DAfStB (2012) formulation was implemented using the residual tensile strength parameter $f_{fctR,u}$. The total shear resistance was calculated as:

$$V_{f,Rd,c} = V_{Rd,c} + V_{Rd,cf} \quad (\text{A14})$$

The concrete contribution was determined from:

$$V_{Rd,c} = C_{Rd,c} \frac{k(100\rho_l f_{ck})^{1/3} b_w d}{\gamma_c} \quad (\text{A15})$$

The fiber contribution was calculated as:

$$V_{Rd,cf} = \alpha_{fc} \frac{f_{fctR,u} b_w h}{\gamma_{fct}} \quad (\text{A16})$$

The residual tensile strength parameter was obtained from:

$$f_{fctR,u} = k_{fF} k_{fG} (0.37) f_{cf1k,L2} \quad (\text{A17})$$

The size factor was determined as:

$$k_{fG} = 1 + 0.5 A_{fct} \leq 1.7 \quad (\text{A18})$$

where

$$A_{fct} = b_w \min(d, 1.5 \text{ m}) \quad (\text{A19})$$

and the size effect factor was calculated from:

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \quad (\text{A20})$$

In these equations, $V_{f,Rd,c}$, $V_{Rd,c}$, and $V_{Rd,cf}$ refer to the total shear resistance, the resistance due to the concrete material and the resistance due to the fibers respectively. Additionally, $f_{fctR,u}$ represents the tensile strength residual parameter, b_w , h and d are the dimensions of the member. k_{fF} and k_{fG} represent fiber orientation effect parameter and size effect parameter respectively while A_{fct} represents tensile effective area.

Based on the recommendations by Lantsoght, the following values were chosen during calculations:

$$k_{fF} = 0.50, \alpha_{fc} = 0.85, \gamma_{fct} = 1.25 \quad (A21)$$

The resulting value of $f_{fctR,u}$ was subsequently used as the residual tensile strength parameter required by the fib Model Code 2010 procedure.

A.6. RILEM TC 162-TDF (2003)

The shear resistance was evaluated according to the recommendations of RILEM TC 162-TDF (2003). The total shear resistance was calculated as:

$$V_{Rd} = V_{cd} + V_{fd} \quad (A22)$$

The concrete contribution was determined from:

$$V_{cd} = 0.12k(100\rho_l f_{ck})^{1/3} b_w d \quad (A23)$$

The fiber contribution was calculated as:

$$V_{fd} = 0.7k_f k \tau_{fd} b_w d \quad (A24)$$

where the flange factor was determined as:

$$k_f = 1 + n \left(\frac{h_f}{b_w} \right) \left(\frac{h_f}{d} \right) \leq 1.5 \quad (A25)$$

and

$$n = \frac{b_f}{b_w} - 1 \leq 3 \quad (A26)$$

The design bond strength parameter was obtained from:

$$\tau_{fd} = 0.12 f_{Rk,4} \quad (A27)$$

Following the methodology adopted by Lantsoght, the characteristic residual flexural tensile strength required by the RILEM procedure was assumed equal to the splitting tensile strength predicted by the Thomas model:

$$f_{Rk,4} = f_{spfc} \quad (A28)$$

In Eqs. (A22)–(A28), V_{Rd} represents total shear resistance, V_{cd} represents the concrete contribution, V_{fd} represents the fiber contribution, k is the size effect factor, ρ_l is the longitudinal reinforcement ratio, f_{ck} represents the characteristic compressive strength of concrete, b_w is the web width, d is the effective depth, k_f is the flange factor, h_f is the flange depth, b_f is the flange width, τ_{fd} represents the design bond strength parameter, $f_{Rk,4}$ is the characteristic residual flexural tensile strength corresponding to a CMOD of 3.5 mm and f_{spfc} is the splitting tensile strength determined using the Thomas model.

For rectangular beam specimens, $k_f=1.0$. The splitting tensile strength f_{spfc} computed using Eq. (A13) was then used in Eq. (A28) for calculation of the fiber contribution to shear resistance using the RILEM method.

A.7. Zhang et al. (2020)

Zhang et al. proposed a modified compression field theory (MCFT)-based shear model for polypropylene fiber reinforced concrete beams. In the original formulation, the principal tensile stress consists of the sum of aggregate interlock stress and fiber-induced tensile stress. In the present study, only the fiber-induced tensile stress component was considered and directly adopted as the principal tensile stress.

The crack inclination angle was determined according to:

$$\theta = 29^\circ + 7000\varepsilon_x \quad (A29)$$

where ε_x is the longitudinal strain. Following the recommendation of Zhang et al.,

$$\varepsilon_x = 0.002 \quad (A30)$$

was assumed, resulting in a crack inclination angle of $\theta = 43^\circ$.

The principal tensile stress was then calculated directly from the fiber-induced tensile stress expression proposed by Zhang et al.:

$$\sigma_f = \left(1 + \eta_0 \eta_1 \eta_2 V_f \frac{l_f}{d_f}\right) f_t \quad (\text{A31})$$

where V_f is the fiber volume fraction, l_f/d_f is the fiber aspect ratio, and f_t is the tensile strength of concrete. Here, η_0 , η_1 , and η_2 denote the fiber orientation factor, fiber length efficiency factor, and bond coefficient, respectively. Zhang et al. suggested using the values $\eta_0=0.20$, $\eta_1=0.50$, and $\eta_2=1.0$ for a 3-D random fiber orientation.

In the current analysis, the interlock effect which forms part of the original equation was ignored, and the maximum tensile stress was taken to be the same as the tensile stress due to the fibers, $f_t = \sigma_f$. This was then used to estimate the residual tensile effect of the PFRC.

A.8. Said et al. (2021)

Said et al. proposed a modified shear strength model based on the Canadian Standards Association (CSA) provisions to account for the contribution of polyvinyl alcohol (PVA) fibers to beam shear resistance. The total shear resistance is expressed as the sum of the concrete, fiber, and stirrup contributions:

$$V_u = V_c + V_f + V_s \quad (\text{A32})$$

where V_c , V_f , and V_s represent the contribution of the concrete, fibers, and transverse reinforcement, respectively.

The concrete contribution is calculated by the CSA model formula:

$$V_c = \phi_c k \beta \sqrt{f'_c} b d_v \quad (\text{A33})$$

where $\phi_c = 0.65$, $k = 1.0$, and $\beta = 0.21$. The effective shear depth, d_v , is determined as follows:

$$d_v = \max(0.9d, 0.75h) \quad (\text{A34})$$

where d is the effective depth and h is the overall section depth.

The fiber contribution is given by:

$$V_f = F_f \beta_0 \tau b d_v \quad (\text{A35})$$

in which F_f is the fiber factor, β_0 is the fiber orientation factor, and τ is the fiber-matrix interface bond strength.

The fiber factor is given by:

$$F_f = V_f \left(\frac{l_f}{d_f}\right) k_f \quad (\text{A36})$$

in which V_f stands for volume fraction of fibers, l_f represents length of fibers, d_f is diameter of fibers, and k_f represents shape factor of fibers.

For PVA fibers, Said et al. adopted the following parameters:

$$\beta_0 = 0.41 \text{ and } k_f = 0.50 \quad (\text{A37})$$

$$\tau = 2.93 \text{ MPa} \quad (\text{A38})$$

Although the original model includes the contribution of transverse reinforcement, all specimens in the present database were tested without stirrups. Therefore, the contribution of transverse reinforcement was neglected:

$$V_s = 0 \quad (\text{A39})$$

Consequently, the predicted shear resistance was calculated as:

$$V_u = V_c + V_f \quad (\text{A40})$$

In the present study, the coefficients suggested by Said et al. were used as such without any change; however, the effect of transverse reinforcement was not included due to lack of any stirrup in the tested specimens.

A.9. Arslan et al. (2017)

Arslan et al. proposed a shear strength model for polypropylene fiber-reinforced concrete (PFRC) beams without stirrups. The model was derived from a previously developed formulation for steel fiber reinforced concrete beams and modified by introducing a polypropylene-to-steel ratio (PSR) to account for the lower efficiency of polypropylene fibers. The model incorporates the effects of concrete strength, longitudinal reinforcement ratio, fiber contribution, and shear span-to-depth ratio.

The nominal shear strength is calculated as:

$$v_n = \frac{0.2f_c^{\frac{2}{3}}}{\left(\frac{c}{d}\right)} + PSR \left(\frac{\sqrt{\rho(1+4F)}f_c}{3\left(\frac{a}{d}\right)} \right)^{\frac{1}{3}} \quad (\text{A41})$$

where f_c is the concrete cylinder compressive strength in MPa, ρ is the longitudinal steel ratio, a/d is the shear span to effective depth ratio, PSR is the polypropylene to steel fiber ratio, and F is the fiber factor.

The depth of the compression zone is obtained from:

$$\left(\frac{c}{d}\right)^2 + 600 \left(\frac{\rho}{f_c}\right) \left(\frac{c}{d}\right) - 600 \left(\frac{\rho}{f_c}\right) = 0 \quad (\text{A42})$$

where c is the depth of the compression zone and d is the effective depth.

The fiber factor is defined as:

$$F = \frac{1}{4} \left(\frac{\psi l_f}{A_f} \right) V_f d_f \quad (\text{A43})$$

where ψ is the fiber perimeter, l_f is the fiber length, A_f is the fiber cross-sectional area, V_f is the fiber volume fraction, and d_f is the fiber shape and bond factor.

In this formulation, d_f denotes the fiber shape and bond factor, which differs from the notation adopted in some of the other models where d_f represents the fiber diameter.

For non-circular fibers, Arslan et al. adopted the fiber intrinsic efficiency ratio proposed by Naaman and incorporated it into the fiber factor expression to better represent the bond characteristics of polypropylene fibers.

To account for the reduced bond and stiffness characteristics of polypropylene fibers relative to steel fibers, the polypropylene-to-steel ratio was taken as:

$$PSR = 0.70 \quad (\text{A44})$$

as recommended by the authors.

The fiber shape bond factor depends on fiber geometry. According to Narayanan and Darwish, values of 0.50, 0.75, and 1.00 may be used for round, crimped, and indented fibers, respectively.

In the present study, the original formulation proposed by Arslan et al. was adopted without modification. All specimens in the database were tested without transverse reinforcement; therefore, no stirrup contribution was considered. The nominal shear strength predicted was used directly for comparison with the experimentally measured shear strengths.

A.10. Albidah and Abbas (2024)

The authors, Albidah and Abbas, came up with a simpler design-oriented model based on a stacked ensemble machine learning model using a database consisting of 394 SFRC beams without stirrups. The sensitivity analysis revealed that the most important factors influencing the shear capacity of the beam were the shear span to depth ratio, the ratio of longitudinal reinforcement, the compressive strength of the concrete, and the fiber factor. Using these results, an empirical equation was formulated by the researchers.

The predicted shear strength is calculated as:

$$v_u = \alpha F^\beta \sqrt{f'_c} \left(\frac{\rho d}{a} \right)^\gamma \quad (\text{A45})$$

where v_u = Shear strength in MPa, f'_c = Concrete compressive strength in MPa, ρ = Longitudinal reinforcement ratio, d = Effective depth, a = Shear span length and F = Fiber Factor. The formula for the fiber factor is:

$$F = V_f \left(\frac{l_f}{d_f} \right) \quad (\text{A46})$$

where V_f is the fiber volume fraction, l_f is the fiber length, and d_f is the fiber diameter.

Using the Gauss-Newton iterative calibration technique, the parameters of the model were obtained as follows:

$$\alpha = 6.588 \quad (\text{A47})$$

$$\beta = 0.134 \quad (\text{A48})$$

$$\gamma = 0.537 \quad (\text{A49})$$

Appendix B. Complete specimen-level database

The complete specimen-level database used in the present study is provided in Table B. The database contains the geometric, material, reinforcement-related, fiber-related, and experimental properties of all specimens included in the assessment of shear strength prediction models.

Table B. The complete specimen-level database

B.N.	Research	Beams	b (mm)	h (mm)	d (mm)	a/d	ρ_s	f_y (MPa)	V_f (%)	Aspect ratio	f'_c (MPa)	f_{sf} (MPa)	E_{sf} (MPa)	Failure mode	V_{exp} (kN)
1	Altoubat et al. (2009)	L1-0.50-b	280	460	400	3.53	0.0215	420	0.50	90.00	41.90	620	9500	Shear	193.00
2	Altoubat et al. (2009)	L2-0.50-a	230	390	330	3.50	0.0318	420	0.50	90.00	41.90	620	9500	Shear	131.50
3	Altoubat et al. (2009)	L2-0.50-b	230	390	330	3.50	0.0318	420	0.50	90.00	41.90	620	9500	Shear	133.50
4	Altoubat et al. (2009)	L1-0.75-a	280	460	400	3.53	0.0215	420	0.75	90.00	41.90	620	9500	Shear	214.50
5	Altoubat et al. (2009)	L1-0.75-b	280	460	400	3.53	0.0215	420	0.75	90.00	41.90	620	9500	Shear	213.00
6	Altoubat et al. (2009)	L2-0.75-b	230	390	330	3.50	0.0318	420	0.75	90.00	41.90	620	9500	Shear	139.50
7	Altoubat et al. (2009)	L2-1.0-b	230	390	330	3.50	0.0318	420	1.00	90.00	35.60	620	9500	Shear	151.50
8	Altoubat et al. (2009)	Sh1-0.50-a	280	460	400	2.28	0.0215	420	0.50	90.00	41.90	620	9500	Shear	219.50
9	Altoubat et al. (2009)	Sh2-0.50-b	230	390	330	2.27	0.0318	420	0.50	90.00	41.90	620	9500	Shear	150.00
10	Altoubat et al. (2009)	Sh2-0.75-a	230	390	330	2.27	0.0318	420	0.75	90.00	41.90	620	9500	Shear	168.50
11	Altoubat et al. (2009)	Sh2-0.75-b	230	390	330	2.27	0.0318	420	0.75	90.00	41.90	620	9500	Shear	170.50
12	Navas et al. (2018)	OAP1	305	552	472.9	3.87	0.0167	579	1.10	56.50	43.12	400	4700	Shear	223.34
13	Navas et al. (2018)	OAP2	305	552	472.5	4.84	0.0223	579	1.10	56.50	44.91	400	4700	Shear	243.13
14	Navas et al. (2018)	OBP1	229	552	470.6	3.89	0.0224	579	1.10	56.50	42.67	400	4700	Shear	180.87
15	Navas et al. (2018)	OBP2	229	552	469.2	4.87	0.0225	579	1.10	56.50	42.00	400	4700	Shear	147.80
16	Al-Alawi and Mashrei (2022)	B-Sy M PF1–6.4	100	150	130	2.31	0.0464	500	0.69	64.00	150.00	685	10500	Shear	65.26
17	Al-Alawi and Mashrei (2022)	B-Sy M PF1–4.8	100	150	130	2.31	0.0464	500	0.52	64.00	150.00	685	10500	Shear	58.84
18	Al-Alawi and Mashrei (2022)	B-Sy M PF1–3.2	100	150	130	2.31	0.0464	500	0.34	64.00	150.00	685	10500	Shear	57.77
19	Al-Alawi and Mashrei (2022)	B-Sy M PF2–6.4	100	150	130	2.31	0.0464	500	0.69	180.00	150.00	620	6600	Shear	64.72
20	Al-Alawi and Mashrei (2022)	B-Sy M PF2–4.8	100	150	130	2.31	0.0464	500	0.52	180.00	150.00	620	6600	Shear	57.77
21	Al-Alawi and Mashrei (2022)	B-Sy M PF2–3.2	100	150	130	2.31	0.0464	500	0.34	180.00	150.00	620	6600	Shear	54.56
22	Al-Alawi and Mashrei (2022)	B-Sy M PF1–4.8 -p3	100	150	130	2.31	0.0464	500	0.52	64.00	150.00	685	10500	Shear	66.86
23	Al-Alawi and Mashrei (2022)	B-Sy M PF1–4.8 -p2	100	150	130	2.31	0.0396	500	0.52	64.00	150.00	685	10500	Shear	58.84
24	Al-Alawi and Mashrei (2022)	B-Sy M PF1–4.8 -p1	100	150	130	2.31	0.0329	500	0.52	64.00	150.00	685	10500	Shear	44.93
25	Xiang et al. (2022)	F3-A-3	150	300	241.5	2.98	0.0281	542	0.32	37.50	34.50	543	8200	Shear	62.86
26	Xiang et al. (2022)	F6-A-3	150	300	241.5	2.98	0.0281	542	0.63	37.50	37.50	543	8200	Shear	83.89
27	Xiang et al. (2022)	F9-A-3	150	300	241.5	2.98	0.0281	542	0.95	37.50	36.90	543	8200	Shear	96.72
28	Altoubat et al. (2020)	VB-0L-0.50	230	390	330	3.48	0.0318	420	0.50	90.00	32.30	620	9500	Shear	90.00
29	Uğur and Ünal (2022)	SCC-F2-S1	125	250	240	2.50	0.0052	366	0.28	77.14	38.61	550	7060	Shear	23.35
30	Zhang et al. (2014)	RE-00	150	300	250	2.80	0.0270	400	3.00	333.33	32.80	482	5000	Shear	104.38
31	Ababneh et al. (2017)	B1V3S0	100	150	125	2.40	0.0322	470	0.33	90.00	44.40	620	9500	Shear	32.00
32	Ababneh et al. (2017)	B1V5S0	100	150	125	2.40	0.0322	470	0.54	90.00	45.10	620	9500	Shear	35.00
33	Ababneh et al. (2017)	B1V7S0	100	150	125	2.40	0.0322	470	0.76	90.00	45.90	620	9500	Shear	38.50

34	Arslan et al. (2017)	B2.5P1.0	150	240	210	2.50	0.0128	492	1.00	50.00	27.00	470	3600	Shear	47.92
35	Arslan et al. (2017)	B2.5P2.0	150	240	210	2.50	0.0128	492	2.00	50.00	13.85	470	3600	Shear	42.94
36	Arslan et al. (2017)	B2.5P3.0	150	240	210	2.50	0.0128	492	3.00	50.00	18.45	470	3600	Shear	56.06
37	Arslan et al. (2017)	B3.5P1.0	150	240	210	3.50	0.0128	492	1.00	50.00	27.00	470	3600	Shear	46.75
38	Arslan et al. (2017)	B3.5P2.0	150	240	210	3.50	0.0128	492	2.00	50.00	13.85	470	3600	Shear	42.50
39	Arslan et al. (2017)	B3.5P3.0	150	240	210	3.50	0.0128	492	3.00	50.00	18.45	470	3600	Shear	50.72
40	Majdzadeh et al. (2006)	B6	150	150	120	3.02	0.0262	500	0.50	85.00	43.90	620	9500	Shear	86.00
41	Majdzadeh et al. (2006)	B7	150	150	120	3.02	0.0262	500	1.00	85.00	44.20	620	9500	Shear	113.00
42	Majdzadeh et al. (2006)	B8	150	150	120	3.02	0.0262	500	1.50	85.00	43.10	620	9500	Shear	104.00
43	Majdzadeh et al. (2006)	B9	150	150	120	3.02	0.0262	500	0.50	360.00	43.40	375	3500	Shear	82.00
44	Majdzadeh et al. (2006)	B10	150	150	120	3.02	0.0262	500	1.00	360.00	44.80	375	3500	Shear	100.00
45	Majdzadeh et al. (2006)	B11	150	150	120	3.02	0.0262	500	1.50	360.00	42.00	375	3500	Shear	99.00
46	Sahoo et al. (2015)	PFRC	150	200	172	3.49	0.0234	500	1.00	25.00	37.60	460	5000	Shear	38.40
47	Shaaban et al. (2021)	B5	120	300	260	2.35	0.0326	700	1.50	666.67	45.90	400	3450	Shear	63.00
48	Shaaban et al. (2021)	B6	120	300	260	2.35	0.0326	700	2.50	666.67	45.90	400	3450	Shear	70.80
49	Shaaban et al. (2021)	B7	120	300	260	2.35	0.0326	700	0.75	666.67	45.05	400	3450	Shear	55.00
50	Bastami (2019)	SCC-0.75%S1-15M-0	125	250	201	3.69	0.0160	434	0.75	74.00	51.60	625	9500	Shear	42.85
51	Bastami (2019)	HSC-0.75%S1-15M-0	125	250	201	3.69	0.0160	434	0.75	74.00	71.40	625	9500	Shear	56.10
52	Bastami (2019)	HSC-0.75%S1-20M-0	125	250	201	3.69	0.0238	426	0.75	74.00	78.30	625	9500	Shear	85.00
53	Murad and Abdel-Jabbar (2022)	P2	150	250	225	1.56	0.0119	420	0.22	76.50	45.30	585	6600	Shear	73.90
54	Murad and Abdel-Jabbar (2022)	P4	150	250	225	1.56	0.0119	420	0.44	76.50	40.80	585	6600	Shear	57.85
55	Murad and Abdel-Jabbar (2022)	P8	150	250	225	1.56	0.0119	420	0.88	76.50	37.70	585	6600	Shear	69.75
56	Altoubat et al. (2012)	L-F	230	390	330	3.50	0.0318	420	0.50	90.00	42.00	620	9500	Shear	132.50
57	Altoubat et al. (2012)	Sh-F	230	390	330	2.27	0.0318	420	0.50	90.00	42.00	620	9500	Shear	159.00
58	Parmentier et al. (2012)	Sy4.5-1-1	200	300	270	1.50	0.0116	500	0.50	70.00	54.50	600	5000	Shear	128.10
59	Parmentier et al. (2012)	Sy4.5-1-2	200	300	270	1.50	0.0116	500	0.50	70.00	54.50	600	5000	Shear	162.60
60	Parmentier et al. (2012)	Sy4.5-2-1	200	300	270	2.50	0.0116	500	0.50	70.00	54.50	600	5000	Shear	105.50
61	Parmentier et al. (2012)	Sy4.5-2-2	200	300	270	2.50	0.0116	500	0.50	70.00	54.50	600	5000	Shear	111.70
62	Furlan Jr. and de Hanai (1997)	P2B	100	100	85.25	3.52	0.0166	500	0.50	840.00	48.00	400	3500	Shear	17.50
63	Hossain et al. (2023)	R0C5F1	200	300	268	1.85	0.0075	496	0.15	666.67	28.30	400	4010	Shear	86.70
64	Hossain et al. (2023)	R0C10F1	200	300	268	1.85	0.0075	496	0.15	666.67	41.90	400	4010	Shear	105.10
65	Shoeib et al. (2023)	B2	120	650	600	0.58	0.0141	400	0.30	56.47	44.20	400	4700	Shear	440.00
66	Shoeib et al. (2023)	B3	120	650	600	0.58	0.0141	400	0.60	56.47	51.00	400	4700	Shear	480.00
67	Shoeib et al. (2023)	B4	120	650	600	0.58	0.0141	400	0.90	56.47	54.40	400	4700	Shear	500.00
68	Mori et al. (2017)	C-P12	100	200	164	1.22	0.0130	500	0.40	42.86	49.10	500	9500	Shear	71.45
69	Mori et al. (2017)	D-P12	250	500	434	1.15	0.0136	500	1.20	42.86	57.60	500	9500	Shear	443.15
70	Bajaj and Shrivastava (2011)	1-B2	100	150	130	3.85	0.0121	428	0.10	158.80	40.12	680	5800	Shear	19.95
71	Bajaj and Shrivastava (2011)	2-B2	100	150	130	3.85	0.0174	423	0.10	158.80	40.12	680	5800	Shear	24.30
72	Bajaj and Shrivastava (2011)	1-B3	100	150	130	3.85	0.0121	428	0.20	158.80	40.97	680	5800	Shear	21.00
73	Bajaj and Shrivastava (2011)	2-B3	100	150	130	3.85	0.0174	423	0.20	158.80	40.97	680	5800	Shear	25.35
74	Bajaj and Shrivastava (2011)	1-B4	100	150	130	3.85	0.0121	428	0.30	158.80	39.78	680	5800	Shear	22.20
75	Bajaj and Shrivastava (2011)	2-B4	100	150	130	3.85	0.0174	423	0.30	158.80	39.78	680	5800	Shear	26.40
76	Bajaj and Shrivastava (2011)	1-B5	100	150	130	3.85	0.0121	428	0.40	158.80	39.53	680	5800	Shear	22.80
77	Bajaj and Shrivastava (2011)	2-B5	100	150	130	3.85	0.0174	423	0.40	158.80	39.53	680	5800	Shear	27.15
78	Bajaj and Shrivastava (2011)	1-B6	100	150	130	3.85	0.0121	428	0.50	158.80	38.25	680	5800	Shear	21.75
79	Bajaj and Shrivastava (2011)	2-B6	100	150	130	3.85	0.0174	423	0.50	158.80	38.25	680	5800	Shear	26.25
80	Bajaj and Shrivastava (2011)	1-B7	100	150	130	3.85	0.0121	428	0.60	158.80	36.55	680	5800	Shear	21.45

81	Bajaj and Shrivastava (2011)	2-B7	100	150	130	3.85	0.0174	423	0.60	158.80	36.55	680	5800	Shear	25.05
82	Al-Khafaji and Harba (2023)	BS2	150	200	175	2.86	0.0129	678	0.10	666.67	37.20	400	3500	Shear	57.27
83	Al-Khafaji and Harba (2023)	BS3	150	200	175	2.86	0.0129	678	0.20	666.67	39.26	400	3500	Shear	49.32
84	Kamal et al. (2014)	B10P	100	150	130	2.31	0.0121	550	0.11	12.50	130.00	400	3500	Shear	20.95
85	Kamal et al. (2014)	B12P	100	150	130	2.31	0.0174	550	0.11	12.50	130.00	400	3500	Shear	27.00
86	Al Ghali et al. (2023)	PP-42	150	250	225	1.48	0.0186	565	0.25	42.00	49.20	600	9500	Shear	89.15
87	Al Ghali et al. (2023)	PP-270	150	250	225	1.48	0.0186	565	0.25	270.00	40.10	570	3500	Shear	81.30
88	Atea (2017)	BM(30)	100	300	280	1.00	0.0287	500	0.50	667.00	31.00	400	3500	Shear	160.00
89	Atea (2017)	BM(40)	100	300	280	1.00	0.0287	500	1.00	667.00	40.00	400	3500	Shear	166.00
90	Rao et al. (2011)	0,5-MixA	100	300	280	0.71	0.0020	500	0.12	300.00	18.47	600	3500	Shear	13.72
91	Rao et al. (2011)	0,5-MixB	100	300	280	0.71	0.0020	500	0.12	300.00	24.36	600	3500	Shear	20.16
92	Rao et al. (2011)	0,5-MixC	100	300	280	0.71	0.0020	500	0.12	300.00	29.78	600	3500	Shear	24.08
93	Rao et al. (2011)	1-MixA	100	300	280	0.71	0.0020	500	0.23	300.00	19.07	600	3500	Shear	15.12
94	Rao et al. (2011)	1-MixB	100	300	280	0.71	0.0020	500	0.23	300.00	24.60	600	3500	Shear	20.44
95	Rao et al. (2011)	1-MixC	100	300	280	0.71	0.0020	500	0.23	300.00	31.47	600	3500	Shear	24.64
96	Ozturk et al. (2017)	B2.5P1.0	150	240	210	2.50	0.0128	492	1.00	50.00	27.00	470	3600	Shear	38.34
97	Ketenci (2022)	C30PP1	250	400	367	2.45	0.0088	420	0.66	350.00	30.00	630	8000	Shear	164.34
98	Ketenci (2022)	C30PP2	250	400	367	2.45	0.0088	420	0.66	350.00	30.00	630	8000	Shear	154.92

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Data Availability

The datasets generated and/or analyzed during the current study are not publicly available but are available from the corresponding author upon reasonable request.

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During the preparation of this manuscript, AI models were used exclusively for language editing and stylistic refinement. The authors take full responsibility for the content, interpretation, and conclusions of the published article.

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