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Concrete-to-concrete interface behaviour in precast girder bridges made continuous: deficiencies and challenges

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Summary

Aging infrastructure in the Netherlands presents a significant challenge, particularly with precast girder bridges made continuous, which exhibit inadequate shear reinforcement per current design codes. To address shear capacity and accuracy of current assessment practices, a research program including full-scale shear tests is underway at Delft University of Technology. As a part of the research, a blind prediction contest with two specimens has been organized. The experiments showed that the loss of composite action at the interface is the primary failure mechanism, and generally an accurate model of the interface behaviour in such composite members is missing. In this paper, a further review of the available interface models and previous tests is conducted. This review leads to the challenges in the accurate evaluation of the interface behaviour, as well as the next steps to address them: a new set of full-scale specimens, and a small-scale test setup reflecting a realistic stress distribution.

1 INTRODUCTION

Precast girder bridges have been constructed in the Netherlands since the 1960's. Originally designed as simply supported, they can also be made continuous by including a cast-in-situ top layer and a cross-beam at the intermediate supports. This continuous system reduces the maximum bending moment at the mid span due to live load, allowing for a decrease in deck height [1]. Currently, the Dutch highway system features over 100 precast girder bridges, with provinces and municipalities collectively owning approximately ten times as many [2].

However, the assessment of these bridges poses a challenge. A relevant percentage, when evaluated against standards like Eurocode 2 [3], do not comply with the minimum required shear reinforcement. To address the urgent need for evaluating the flexural shear capacity of these shear-critical structures, a research program including an experimental campaign on full-scale specimens is underway at Delft University of Technology. One of the main outcomes of this study as of yet is the identification of the loss of composite action at the interface as the primary failure mechanism for certain beams.

Given these findings in the current campaign, an accurate numerical and analytical interface model is required for the assessment of these bridges. This paper represents the first steps to study interface behaviour in precast girder bridges, focusing on: (i) a literature study, (ii) identification of the challenges, and (iii) the proposal for the continuation of the experimental campaign.

2 RESEARCH ON INTERFACE SHEAR CAPACITY IN LITERATURE

2.1 Mechanisms of shear transfer

It is generally accepted that the shear strength at a concrete-to-concrete interface is determined by four mechanisms. The first one is adhesion, which is the result of the chemical bond between the two surfaces, e.g., old and new concrete. When a crack initiates along the hardened cement paste, following the edges of the aggregate particles, the relative displacement is resisted by a second mechanism known as aggregate interlocking [4]. Both the presence of reinforcement crossing the interface, and external compressive forces, activate the mechanism of friction [5]. The fourth is dowel action, which describes the capacity of the interface reinforcement to transfer forces perpendicular to its axis [6].

The complexity of understanding and modelling shear transfer at the interface is in the interaction between these four factors. For instance, adhesion is a contribution until the moment a crack appears at an interface, which happens at small relative slip values. After this point, aggregate interlock is activated. Also, dowel action requires a larger relative slip value to start contributing to the shear capacity. Given these mechanisms do not occur simultaneously, a model capable of describing their interaction is needed for an accurate prediction of the shear behaviour at the interface.

2.2 Code provisions for ultimate interface shear capacity

Typically, the ultimate interface shear capacity v_u is calculated with empirical capacity models provided by design codes. In Europe, Eurocode 2 (EC2) [3] is predominantly used for concrete structures design. The approach is based on Eq. 1, with c and μ as cohesion and friction factors, f_{ctd} and f_{cd} as the concrete's design tensile and compressive strength, σ_n as the stress per unit area caused by the external normal force, α, ρ, and f_{yd} as the angle of the interface reinforcement, reinforcement ratio, and design yield strength, respectively, and ν as a strength reduction factor. The first term in Eq. 1 accounts for cohesion (adhesion and aggregate interlocking), the second for friction due to external normal forces, and the third is the friction from the interface reinforcement [3], [7].

$$v_u = c \cdot f_{ctd} + \mu \cdot \sigma_n + \rho \cdot f_{yd} \cdot (\mu \cdot \sin\alpha + \cos\alpha) \leq 0.5 \cdot \nu \cdot f_{cd} \tag{1}$$

The final preliminary release of Eurocode 2 (FprEC2) [8] introduced changes in the calculation of v_u. When no interface reinforcement is required, or when it is adequately anchored, the design shear resistance can be determined with Eq. 2, which is a small modification of Eq. 1. In cases where sufficient anchorage is not guaranteed, a new term accounting for dowel action is incorporated, as shown in Eq. 3. In both equations, f_{ck} represents the concrete characteristic strength, γ_c denotes a partial safety factor, and c_1, c_2 , k_v and k_{dowel} are factors that depend on the surface roughness.

$$v_u = c_1 \cdot \sqrt{f_{ck}}/\gamma_c + \mu \cdot \sigma_n + \rho \cdot f_{yd} \cdot (\mu \cdot sin\,\alpha + cos\,\alpha) \leq 0.3 \cdot f_{cd} + \rho \cdot f_{yd} \cdot cos\,\alpha \tag{2}$$

$$v_u = c_2 \cdot \sqrt{f_{ck}}/\gamma_c + \mu \cdot \sigma_n + k_v \cdot \rho \cdot f_{yd} \cdot \mu + k_{dowel} \cdot \rho \cdot \sqrt{f_{yd} \cdot f_{cd}} \leq 0.25 \cdot f_{cd} \tag{3}$$

Another relevant standard is LRFD Bridge Design Specifications (LRFD) by the AASHTO [9]. Primarily used in the United States, the approach for v_u calculation starts by obtaining the nominal shear force V_u with Eq. 4, where A_{cv} is the interface area, A_{vf} is the area of interface shear reinforcement within A_{cv}, both per meter length, and P_n is the external normal force. Dividing Eq. 4 by A_{cv} results in Eq. 5, where it is evident that, similarly to EC2 and FprEC2, this approach also accounts for cohesion, friction due to external normal forces, and friction due to reinforcement [9].

$$V_u = c \cdot A_{cv} + \mu \cdot (A_{vf} \cdot f_y + P_n) \tag{4}$$

$$v_u = c + \mu \cdot \sigma_n + \mu \cdot \rho \cdot f_y \tag{5}$$

EC2, FprEC2, and LRFD share several similarities in their approaches. They all acknowledge cohesion, friction due to external normal forces, and friction from reinforcement as independent contributions, applying superposition. Furthermore, full yielding of the interface reinforcement is assumed. However, a lack of consistency can be seen in the cohesion and friction factors (c and μ). Another observation is that the mechanism of dowel action is only considered by FprEC2 (see Eq. 3).

2.3 Research on interface shear capacity in literature

The basis of the empirical capacity models in standards are the result of years of research on the characterization of interface shear behaviour, with several relevant contributions. Starting in 1966, Birkeland and Birkeland [10] proposed the shear-friction theory, which accounts for the clamping forces from the reinforcement and includes an angle φ representing surface roughness. In 1972, Mattock and Hawkins [11] modified this theory by introducing a constant term to account for cohesion and dowel action [5], [11]. Loov further acknowledged cohesion by incorporating the concrete compressive strength into his model in 1978, later refining it with new data in 1994 [12], [13]. Walraven's research on aggregate interlock [4] in 1980 resulted on both a theoretical and simplified model for quantifying its contribution to interface shear capacity. Contrary, the model by Tsoukantas and Tassios [14] incorporated only friction due to clamping stresses and dowel action. Finally, Randl [15] proposed superimposing cohesion, friction, and dowel action for a realistic shear resistance prediction. Table 1 summarises the mechanisms considered by each discussed capacity model and indicates if a constitutive relation was also proposed. Since all models are based on empirical data to some extent, it is important to further review past experiments from the literature.

Table 1 Mechanisms considered on relevant models from literature. Abbreviations: AD = adhesion, AG = aggregate interlock, FE = friction due to external normal forces, FR = friction due to reinforcement, DA = dowel action, CR = constitutive relation.

Model	Ref.	AD	AG	FE	FR	DA	CR
EC2 (Eq. 1)	[3]	X	X	X	X		
FprEC2 (Eq. 2)	[8]	X	X	X	X		
FprEC2 (Eq. 3)	[8]	X	X	X	X	X	
LRFD (Eq. 4)	[9]	X	X	X	X		
Birkeland and Birkeland	[10]		X		X		
Mattock and Hawkins	[11]		X	X	X	X	
Loov	[12], [13]	X	X		X		X
Walraven	[4]		X				X
Tsoukantas and Tassios	[14]			X	X	X	X
Randl	[15], [16]	X	X	X	X	X	

2.4 Experimental campaigns in literature

Both small and large-scale experimental campaigns have helped to the development of models for interface shear capacity. Notable small-scale experiments include those by [4], [11], [15], [17], [18], and [19], while large-scale campaigns include those conducted by [12], [17], [20], [21] and [22].

One of the most comprehensive small-scale experimental campaigns was conducted by Hanson [17], who investigated parameters such as contact surface properties, length, and interface reinforcement. Key findings revealed that for uncracked joints, (i) higher shear stresses at lower slip values are obtained, and (ii) in shorter surface lengths, the stresses are concentrated near the load application point. Conversely, for cracked joints, the entire contact area actively resists slip. Anderson [18] studied the effect of reinforcement ratio and concrete compressive strength, highlighting the role of bond in achieving monolithic behaviour. The results from [17] and [18] were later utilized by Birkeland and Birkeland [10] to support the shear-friction theory.

Hofbeck, Ibrahim and Mattock [19] investigated the influence of bond, concrete compressive strength and interface reinforcement. Their findings revealed that, (i) cracked specimens exhibit lower ultimate shear strength, (ii) the reinforcement ratio remains a critical parameter regardless of spacing or bar diameter, and (iii) dowel action contributes to shear capacity only in cracked joints. This study was complemented by Mattock and Hawkins [11], who examined the influence of stresses acting parallel and transverse to the shear plane. The results showed that, (i) direct tension parallel to the shear plane reduces strength only in uncracked joints, and (ii) compression loads transverse to the plane can be added to the reinforcement contribution in both cracked and uncracked interfaces. The experimental data on cracked specimens from these two campaigns validated the proposed design model presented in [11].

Walraven [4] focused on the mechanism of aggregate interlock, obtaining shear stress – shear displacement relationships and crack opening paths for parameters such as interface roughness and reinforcement ratio. His model, proposed in [4], proved to be consistent with the experimental data. Randl [15] systemically studied parameters such as interface roughness, bond, and interface reinforcement. The findings indicate that with high interface roughness, the primary mechanisms in a cracked joint are friction and aggregate interlock, while for smooth interfaces, dowel action becomes predominant. This research was crucial in developing the design recommendations for Model Code 2010 [16], [23]. The parameters studied in each of the above-mentioned experimental campaigns, and the interface characteristics, are summarized in Table 2. It is evident from the table that certain parameters are repeatedly studied across different experimental campaigns, such as reinforcement ratio and distribution. While most campaigns intentionally provide sufficient anchorage length, only [15] explored the effect of varying anchorage lengths on the interface shear capacity. Notably, the influence of adhesion is systematically studied. Nevertheless, as discussed by [11], [13], cracked specimens are often prioritized for assessing design equations, as they provide a lower bound for ultimate shear transfer strength.

Experiments with composite beam specimens offer valuable insights because interface shear transfer develops through composite action [24]. The experimental campaign by Hanson [17] was not limited to push-off tests but also included girders subjected to flexural loading. Key findings revealed that, (i) rough uncracked interfaces exhibit deflection curves similar to monolithic specimens, and (ii) rough cracked interfaces deflect more and earlier than their uncracked counterparts, indicating partial rather than full composite action. Saemann and Washa [20] studied the impact of interface roughness, reinforcement ratio, and length of the shear span. The shear stress – slip relationships showed the significant influence of interface roughness and reinforcement ratio on shear strength, with higher concrete compressive strength having minimal impact.

The tests conducted by Patnaik [12] provided insights into the transition between adhesion, aggregate interlock, and the activation of the interface reinforcement, contributing to the model proposed in [13]. Kahn and Slapkus [21] replicated the specimens in [12], but with high strength concrete, and contrarily to [20], the results suggested that the concrete properties do play a significant role in the interface shear strength. Halicka [22] focused on the influence of adhesion in reinforced and unreinforced interfaces, finding that for the same failure mechanism, different interface properties resulted in varying cracking patterns. The main variables studied, and characteristics of the above-mentioned experimental campaigns are summarized in Table 3. The five campaigns exhibit common features, such as boundary conditions and sufficient anchorage length. Similarly to the small-scale experiments, the reinforcement ratio and distribution are also systematically studied, followed by bond, and concrete compressive strength.

Table 2 Interface characteristics and studied parameters in small-scale experiments. Abbreviations: IC = interface conditions, RO = interface roughness, B = presence of bond, RE = reinforcement ratio and distribution, AL = reinforcement anchorage length, CC = concrete compressive strength, CW = crack width, U = uncracked, C = cracked, R = reinforced, N = unreinforced.

Campaign	Ref.	IC	RO	B	RE	AL	CC	CW
Hanson	[17]	U-R, U-N, C-R	X	X	X			
Anderson	[18]	U-R			X		X	
Hofbeck et. al.	[19]	C-R, U-R		X	X		X	
Mattock and Hawkins	[11]	C-R, U-R		X	X		X	
Walraven	[4]	C-N*, C-R	X		X		X	X
Randl and Wicke	[15]	C-R, U-R	X	X	X	X	X	

*External reinforcement not crossing the interface.

The experimental campaigns briefly introduced in this section highlight the continuous interest in investigating shear transfer in interfaces throughout the last 60 years. Despite the comprehensive understanding on the main parameters, a large scatter is observed when investigating specific parameters. For instance, Fig. 3 of [25] demonstrates notable scatter when examining the interface roughness. This variability, also noted by [23] and [24], could be attributed to: (i) the numerous variables affecting shear transfer and their interdependent influences, and (ii) the variation in specimen sizes and test setups. Consequently, using experimental data for direct comparison becomes a challenging task.

Table 3 Interface characteristics and studied parameters in large-scale experiments. Abbreviations: BC = boundary conditions, LC = loading conditions, RO = interface roughness, B = presence of bond, RE = reinforcement ratio and distribution, CC = concrete compressive strength, NA = location of joint regarding neutral axis, SS = simply supported, PL = point load, 1/2/3 = number of point loads.

Campaign	Ref.	BC	LC	RO	B	RE	CC	NA
Hanson	[17]	SS	PL - 2/3	X	X	X		
Saemann and Washa	[20]	SS	PL - 2	X	X	X	X	X
Patnaik	[12]	SS	PL - 1		X	X	X	
Kahn and Slapkus	[21]	SS	PL - 2		X	X	X	
Halicka	[22]	SS	PL - 2		X	X		

3 DEFICIENCY OF CODE PROVISIONS IN EXPERIMENTAL OBSERVATIONS

A full-scale specimen campaign is underway at Delft University of Technology with the aim to investigate the shear behaviour of continuous precast girders. The specimens, with a total length of 15 m, are composed by two inverted T-girders connected by a cross-beam and a cast-in-situ topping (See Fig. 1). In January 2023, a blind prediction contest was organized for two of the specimens: S10H1A, and S10H2D. The specimens, loaded at two points, showed loss of composite action as the primary failure mechanism. Table 4 summarizes the relevant information regarding these experiments, while additional details can be found in [26].

Fig. 1 Geometry of specimens and location of loading points P1 and P2 (dimensions in mm).

Table 4 Summary of experimental results.

Specimen	f_c [MPa]	f_{ct} [MPa]	f_y [MPa]	ρ [%]	P_1 [kN]	P_2 [kN]	b [mm]	z [mm]
S10H1A	44	3.7	524	0.113	932	586	250	908
S10H2D	38.4	3.4	524	0.452	1947	1225	250	905

The experimental results allow for comparison with the previously introduced code provisions. Table 5 presents the ultimate interface shear capacity v_u for specimens S10H1A and S10H2D, alongside the calculated capacity according to EC2, FprEC2, and LRFD. The contribution from cohesion and friction due to reinforcement are also shown separately. The calculation of $v_{u,exp}$ follows simplified beam theory, recommended by AASHTO [9], and presented in Eq. 6. Here, V_u is the ultimate shear force at the shear critical span, excluding self-weight, b_i denotes the interface width, and z is the inner lever arm.

$$v_u = V_u/(b_i \cdot z) \quad (6)$$

For the calculation of v_u with EC2, FprEC2, and LRFD, mean values were used and partial safety factors were set to 1. The interface was intentionally roughened by pressing aggregates with a maximum diameter of 32mm on the top surface after the precast beams were produced [27]. Thus, for $v_{u,\mathrm{EC2}}$, a rough interface was assumed. For $v_{u,\mathrm{FprEC2}}$, sufficient anchorage was assumed and Eq. 2 was used. Given c and μ require quantified roughness, both rough (R) and very rough (VR) interfaces were assumed, providing lower and upper bounds for ultimate shear capacity. This approach was also applied for LRFD with Eq. 5, and for this case, f_y was taken as 420 MPa as stipulated by [9]. The friction due to external compressive loads was taken as zero.

Table 5 Ultimate interface shear capacity prediction with EC2, FprEC2, and LRFD in comparison to experimental results. Abbreviations: C = cohesion, FF = friction due to reinforcement, R = rough surface, VR = very rough surface.

Ultimate shear stress [MPa]	Specimen	S10H1A			S10H2D		
	Surface conditions (c, μ)	C	FF	Total	C	FF	Total
$v_{u,exp}$	-	-	-	2.38	-	-	4.99
$v_{u,\mathrm{EC2}}$	R (0.45, 0.7)	1.67	0.41	2.08	1.53	1.66	3.19
$v_{u,\mathrm{FprEC2}}$	R (0.15, 0.7)	1.00	0.41	1.41	0.93	1.66	2.59
	VR (0.19, 0.9)	1.26	0.53	1.79	1.18	2.13	3.31
$v_{u,LRFD}$	R (0.52, 0.6)	0.52	0.29	0.81	0.52	1.14	1.66
	VR (1.9, 1.0)	1.90	0.48	2.38	1.90	1.90	3.80

The results presented in Table 5 provide valuable insights into the proposed calculation methods for each design code. Focusing on cohesion, the concrete contribution, a significant variation is observed across models. This broad range can be attributed to the different calculation approaches and varying cohesion factors. In contrast, the expression for the reinforcement contribution remains consistent across all equations. The variation then comes from the different friction factors and the limitation of f_y by LRFD. For specimen S10H1A, the majority of the shear strength is attributed to the concrete contribution. Conversely, for S10H2D, the reinforcement contribution either equals or exceeds cohesion. The comparison between the prediction and experimental results reveals consistent underestimation across all codes, with LRFD's upper bound providing the closest estimate. Notably, the disparity is larger for specimen S10H2D, characterized by a higher reinforcement ratio. Overall, the inconsistency in the calculation procedures leads to a wide range of strong underestimations.

4 DISCUSSION

The review on code provisions, available models from literature, and past experimental campaigns highlights potential reasons for the inconsistencies observed in shear strength predictions. Firstly, most models, including the widely used Eurocode 2, are empirical capacity models aimed at providing a conservative prediction for ultimate shear strength at the interface. Therefore, the effort is in obtaining a lower bound, which can be challenging given the large scatter in the experimental data available. Furthermore, as seen in Table 1, several models may exclude mechanisms proven to contribute to the interface shear capacity, such as adhesion and dowel action. Furthermore, the comparison between predicted and experimental ultimate shear capacity reveals varying concrete and reinforcement contributions, suggesting inconsistencies in the cohesion and friction factors.

In assessment, particularly with NLFEM, relying solely on a capacity model might be insufficient. Experimental findings indicate that interface properties do not only influence the failure mechanism, but also the structure's response during loading [17], [22]. Therefore, a constitutive relation is necessary. Typically, this relation is established between shear and slip. However, as mentioned by [4], shear transfer is a complex mechanism that also involves normal stress and crack width. Therefore, a shear-slip constitutive relation alone may not suffice to accurately describe this behaviour.

The reviewed experimental campaigns also raise questions about the suitability of available models for precast girder bridges made continuous. The transition from tension to compression along the top section of these bridges results in varying kinematics and stress conditions along the interface. This spatial variation, as observed in Fig. 2 for specimen S10H1A, has not been explored in previous campaigns. Additionally, consideration must be given to typical detailing of Dutch bridges, such as insufficient anchorage length, and the presence of prestressing in the precast girders.

As demonstrated by the preceding discussion, further investigation on the shear capacity at the interface between precast and cast-in-situ concrete for precast girder bridge made continuous is required. Within this context, four main knowledge gaps have been identified. First, the influence of stress conditions and kinematics on the structural behaviour has not been clearly investigated yet. Secondly, there are no physical models available for a sufficiently accurate estimation of the ultimate shear capacity. Thirdly, there are no constitutive relations available for assessment with NLFEM. Finally, the influence of typical detailing found in Dutch bridges on the shear transfer at the interface remains unclear. To address these gaps, a continuation of the full-scale campaign is planned, with a focus on interface behaviour. The parameters to be studied include: type, amount, distribution and detailing of interface reinforcement, amount of top reinforcement, and level of prestressing. Additionally, small-scale experiments will explore interface roughness, concrete strength, bond, ratio and distribution of interface reinforcement.

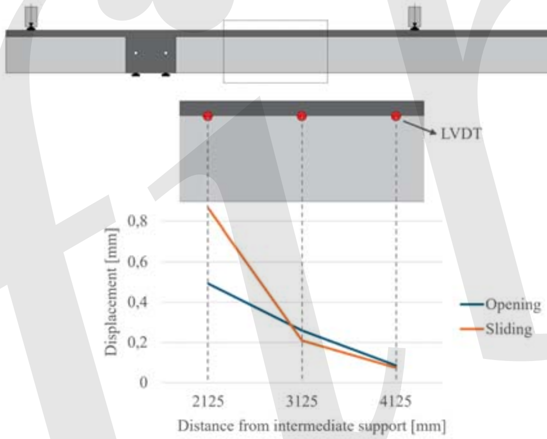

Fig. 2 Spatial variation of opening and sliding in specimen S10H1A before reaching ultimate load.

5 CONCLUSIONS

The interface between the precast girder and the cast-in-situ layer has proven to be relevant in the shear capacity assessment of precast girder bridges made continuous. Available guidelines, including Eurocode 2, underestimate the ultimate shear capacity at the interface, which reveals its unsuitability for an accurate assessment. A review on available models, and past experimental campaigns further demonstrates that the current literature is not applicable to the interface problem presented in this paper. Therefore, a dedicated study on the interface shear transfer is needed. This study will focus on four main topics: (i) stress conditions and kinematics at interface; (ii) capacity models for accurate estimation of ultimate interface shear capacity; (iii) constitutive relation for assessment with NLFEM; and (iv) influence of typical detailing in precast girder Dutch bridges made continuous.

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References

[1] M. Sirage, Y. Yang, M. A. Roosen, and M. Hendriks, "Challenges on the shear behavior of existing continuous precast girder bridges," Rome: 14th International PhD Symposium in Civil Engineering, Sep. 2022. [Online]. Available: https://www.researchgate.net/publication/363518021

[2] Y. Yang, M. Park, N. Kostense, and E. Lantsoght, "Research proposal on shear behaviour of intermediate support," 2020.

[3] European Standard, "NEN-EN 1992-1-1 Eurocode 2: Design of concrete structures - Part 1.1: General rules and rules for buildings," 2005.

[4] J. Walraven, "Aggregate Interlock: A theoretical and experimental analysis," Delft University of Technology, Delft, 1980.
[5] P. M. D. Santos and E. N. B. S. Júlio, "A state-of-the-art review on shear-friction," *Eng Struct*, vol. 45, pp. 435–448, Dec. 2012, doi: 10.1016/j.engstruct.2012.06.036.
[6] J. Walraven, "Mechanisms of shear transfer in cracks in concrete A survey of literature," Delft, Dec. 1978.
[7] A. Monserrat-López, A. Nogales, and A. de la Fuente, "Experimental characterization of concrete-to-concrete interface shear strength design-oriented constitutive parameters," Sep. 01, 2023, *Elsevier Ltd*. doi: 10.1016/j.istruc.2023.06.075.
[8] European Standard, "Eurocode 2 - Design of concrete structures-Part 1-1: General rules and rules for buildings, bridges and civil engineering structures," 2023.
[9] American Association of State Highway and Transportation Officials (AASHTO), *AASHTO LRFD Bridge Design Specifications (9th Edition)*, 9th ed. American Association of State Highway and Transportation Officials (AASHTO), 2020.
[10] P. W. Birkeland and H. W. Birkeland, "Connections in Precast Concrete Construction," *Journal Proceedings*, vol. 63, no. 3, pp. 345–368, 1966.
[11] A. Mattock and N. Hawkins, "Shear Transfer in Reinforced Concrete - Recent Research," *PCI Journal*, 1972.
[12] A. Kumar Patnaik, "Horizontal Shear Strength of Composite Concrete Beams With a Rough Interface," University of Calgary, Calgary, 1992.
[13] R. E. Loov D Phil, P. Eng, and A. K. Patnaik, "Horizontal Shear Strength of Composite Concrete Beams With a Rough Interface," *PCI Journal*, 1994.
[14] S. G. Tsoukantas and T. P. Tassios, "Shear Resistance of Connections between Reinforced Concrete Linear Precast Elements," *Structural Journal*, vol. 86, no. 3, pp. 242–249, 1989.
[15] N. Randl and M. Wicke, "Schubübertragung zwischen alt- und neubeton. Experimentelle untersuchungen, theoretischer hintergrund und bemessungsansatz," *Beton- und Stahlbetonbau*, vol. 95, no. 8, pp. 461–473, 2000, doi: 10.1002/best.200000870.
[16] N. Randl, "Design recommendations for interface shear transfer in fib Model Code 2010," *Structural Concrete*, vol. 14, no. 3, pp. 230–241, 2013, doi: 10.1002/suco.201300003.
[17] N. W. Hanson, "Precast Prestressed Concrete Bridges 2. Horizontal Shear Connections," *Journal of the PCA Research and Development Laboratories*, vol. 2, no. 2, pp. 38–58, 1960.
[18] A. Anderson, "Composite Designs in Precast and Cast-in-Place Concrete," *Prog Arch*, vol. 41, no. 9, pp. 172–179, Sep. 1960.
[19] J. A. Hofbeck, I. 0 Ibrahim, and A. H. Mattock, "Shear Transfer in Reinforced Concrete," *Journal Proceedings*, vol. 66, no. 2, pp. 119–128, 1969.
[20] J. C. Saemann and G. W. Washa, "Horizontal Shear Connections Between Precast Beams and Cast-in-Place Slabs," *Journal of the American Concrete Institute*, 1964.
[21] L. F. Kahn and A. Slapkus, "Interface Shear in High Strength Composite T-Beams," *PCI Journal*, vol. 49, no. 4, pp. 102–110, 2004.
[22] A. Halicka, "Influence new-to-old concrete interface qualities on the behaviour of support zones of composite concrete beams," *Constr Build Mater*, vol. 25, no. 10, pp. 4072–4078, Oct. 2011, doi: 10.1016/j.conbuildmat.2011.04.045.
[23] Fédération internationale du béton, *Model code 2010 Volume 1*. Fédération internationale du béton, 2010.
[24] K. Wieneke, "Horizontal Shear Design of Concrete Interfaces in Beam and Slab Structures," 2019.
[25] V. Palieraki, E. Vintzileou, and J. F. Silva, "Behavior of RC interfaces subjected to shear: State-of-the art review," Nov. 01, 2021, *Elsevier Ltd*. doi: 10.1016/j.conbuildmat.2021.124855.
[26] M. Ibrahim, M. Poliotti, Y. Yang, and M. Hendriks, "Experimental results of Delft blind prediction contest on shear behaviour of continuous precast girders," in *15th fib International PhD Symposium in Civil Engineering*, 2024.
[27] M. Ibrahim and Y. Yang, "Specimens preparation for the experiments on precast concrete bridge at intermediate support: Group 1," 2022.