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Finite Element Modeling of Shear Failure in Prestressed Girders with a Continuous Cast-In-Situ Deck Slab

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Abstract. For structures whose structural safety cannot be demonstrated using the applicable code regulations, a nonlinear finite element analysis (NLFEA) can be used to investigate the structural behavior and maximum load capacity. The Dutch Ministry of Infrastructure and Water Management uses a dedicated guideline for NLFEA in such an assessment. This guideline lacks validation on multi-span girder bridges with continuous deck slabs. Particularly, the modeling of the concrete-to-concrete interface between girders and slab deserves attention, since the interface shear transfer can significantly affect the structural behavior. In this paper we investigate the impact of different interface constitutive relations, and validate the selected modeling approach on three experimental shear tests of continuous girders. The finite element models accurately describe the failure processes and predict, on average, a 10% lower shear capacity as observed in the tests. In anticipation of future research, this is an indication that the modeling approach is suitable to be used in engineering practice.

Keywords: concrete-to-concrete interface · continuous prestressed girder bridge · nonlinear finite element analysis · shear failure

1 Introduction

The highway network in the Netherlands has about 300 multi-span bridges with inverted T-shape prefabricated prestressed girder decks that are made statically indeterminate by a cast-in-situ deck slab. About 40 percent of these bridges are built before the mid-1970s and have a very low percentage of shear reinforcement. For these bridges it has often appeared to be impossible to demonstrate sufficient shear capacity near the intermediate supports with the current Dutch guidelines [1] and the Eurocode 2 [2]. In these cases, nonlinear finite element analysis (NLFEA) can be used to obtain more accurate estimations of the shear capacity. Guidance for modelers to perform such analyses is provided

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by the guidelines for NLFEA from the Dutch Ministry of Infrastructure and Water Management [3]. With this approach, it has often proved possible to demonstrate sufficient shear capacity. However, these guidelines are primarily based on validation studies that include experiments of statically determinate reinforced and prestressed concrete elements [4]. A validation on continuous girders with an cast-in-situ deck slab is missing, which raises the question how accurately NLFEA can determine the shear capacity of such type of structures.

The importance to assess the capability of NLFEA in the context above is twofold: (*i*) to the authors' best knowledge, the open literature on this topic is rather limited; and (*ii*) the structural behavior of these bridges includes complex phenomena as the composite action between the prefabricated girders and the cast-in-situ deck slab, which is difficult to simulate.

The composite action is particularly determined by the shear transfer in the horizontal concrete-to-concrete interface. Its impact on the failure behavior and shear capacity of structures or structural members can be significant. The characterization of interface shear transfer is studied by many researchers. A comprehensive overview of experimental investigations, theoretical models, and design models is provided in [5]. An interesting experimental study is provided by Loov and Patnaik [6]. In this study, prefabricated girders with a cast-in-situ top flange were tested in a three-point-bending test setup. The tests provided insight in the shear behavior of the interface between the girder and the top flange by means of shear stress – slip relations. Theoretical research on shear resistance between concrete surfaces started in the 1960s. An important contribution was made by Birkeland and Birkeland [7] by presenting the shear friction theory. This theory explains that the shear resistance of two concrete surfaces with protruding reinforcement can be attributed to shear friction and dowel action. Shear friction occurs by shear stress transfer between the concrete surfaces and can include the clamping effect from the protruding reinforcement, which is activated during slipping and the accompanying widening of the concrete surfaces. Dowel action is the contribution to the interface shear resistance that comes from the protruding reinforcement bars. This mechanism has been extensively studied by e.g. Randl [8], who provided analytical expressions to calculate the dowel failure load and dowel shear stiffness.

In this paper we investigate the impact of different interface constitutive relations (derived from the referenced research above) on the structural behavior of a continuous girder and validate a NLFEA-based modeling approach on three experimental shear tests of continuous girders. The tests belong to a well-documented experimental campaign of Mattock and Kaar [9]. Section 2 describes these experimental shear tests in more detail. The finite element modeling approach is explained in Sect. 3. Section 4 presents the results of the numerical simulations and compares them with the experimental results. Finally, concluding remarks are given in Sect. 5.

2 Description of the Experimental Tests

2.1 Test Specimens and Experimental Setup

In the experimental work of Mattock and Kaar [9], shear tests have been conducted on fifteen continuous prefabricated prestressed girders. For the validation in this paper, the shear tests denoted as S10, S11 and S12 are used, because of their relatively low shear reinforcement ratio of 0.38% (although this percentage is still higher than used in the older existing prefabricated girder bridges in the Netherlands, these tests have the lowest percentage of the specimens that are representative in terms of other parameters). The specimens of these tests are similar, except that they are tested with different shear spans (x_P), c.q. with different levels of shear stress between the girder and deck slab. Figure 1 shows the experimental setup of the tests and includes information of the geometry, reinforcement and prestressing. The prestress level of the strands after anchorage losses was measured at 1207 N/mm^2. The spacings of the two-legged vertical stirrups in the girder and cantilevered girder stub are respectively 191 mm and 64 mm. The ten Ø12.7 mm reinforcing bars in the deck slab are at the mid-depth of the slab. Five of them have a length of 6706 mm (3658 mm into the main span, the rest over the full length of the cantilevered girder stub), and the other five have a length of 4877 mm (1829 mm into the main span, the rest over the full length of the cantilevered girder stub).

The specimens are half-scale reproductions of real-world prestressed bridge girder types. In order to compensate for size-effects in the dead weight, ten concrete blocks of 3559 N with in-between distances of 914 mm were hung on loops of wire rope around the prefabricated girder before casting of the deck slab. The test setup intends to let the prefabricated girder behave similarly as it would be located in a symmetrically loaded two-span continuous girder. The loading arrangement mimics a vehicle load, represented

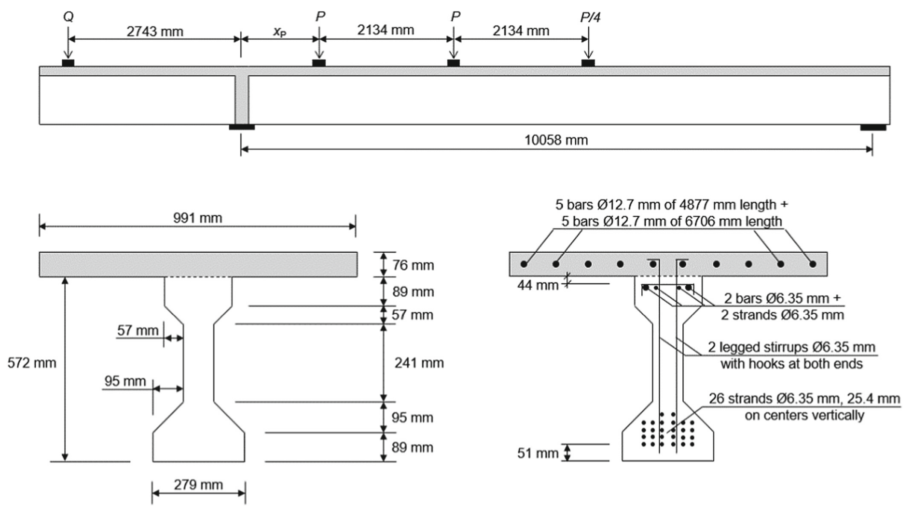

Fig. 1. Experimental setup, geometry and reinforcement layout of the tests S10, S11 and S12.

by three point loads (P) with a load intensity ratio of 1.0: 1.0: 0.25. This load intensity ratio was maintained during testing. A counterbalancing force (Q) on the girder stub was adjusted manually to cancel the rotations at the interior support due to the point loads.

Table 1 provides details on the shear spans (x_P) and the mean concrete cylinder compressive strength values (f_{cm}) of the different structural components. The f_{cm} values are the average from twelve 152 mm × 305 mm cylinder tests. The material properties of the prestressing strands were tested and their values are summarized in Table 2. The reported material properties of the Ø6.35 mm and Ø12.7 mm reinforcing bars correspond with the steel grades Grade 40 and Grade 60, respectively [10].

Table 1. Test specific information and results.

Test	$f_{cm;girder}$ [N/mm^2]	$f_{cm;diaphragm}$ [N/mm^2]	$f_{cm;deck}$ [N/mm^2]	x_P [mm]	V_{cr} [kN]	V_u [kN]	P_u [kN]
S10	43.2	20.4	20.0	1219	195	379	184
S11	43.2	21.4	24.1	1981	206	319	156
S12	45.4	23.0	22.9	2743	236	280	*

f_{cm} is the mean concrete cylinder compressive strength, x_p the shear span, V_{cr} and V_u the total shear force at the support at respectively diagonal tension cracking and failure, and P_u is the point load P (Fig. 1), at failure. * Not reported

2.2 Experimental Results

Table 1 presents the total shear forces at the support at the start of diagonal-tension cracking (V_{cr}) and at failure (V_u), and the value of point load P at failure (P_u). Figure 2 shows the final failure crack patterns. All three specimens failed in a similar way. First, flexural cracks emerged and propagated at the interior support due to the negative bending moment. Subsequently, a long series of diagonal cracks formed in the web and further developed under increased loading. Each specimen showed flexural cracks at midspan and yielding of the continuity reinforcement. In the tests S11 and S12, also horizontal cracks along the connection between the slab and the girder were reported. Ultimately, the specimens failed due to a diagonal compression failure where the concrete near the interior support and in the lower quarter of the web crushed.

3 Description of the Finite Element Model

3.1 Main Aspects of the Finite Element Model

The NLFEAs are performed with the finite element software DIANA FEA 10.4 and follows the guidelines for NLFEA of concrete structures from the Dutch Ministry of Infrastructure and Water Management (Rijkswaterstaat) [3].

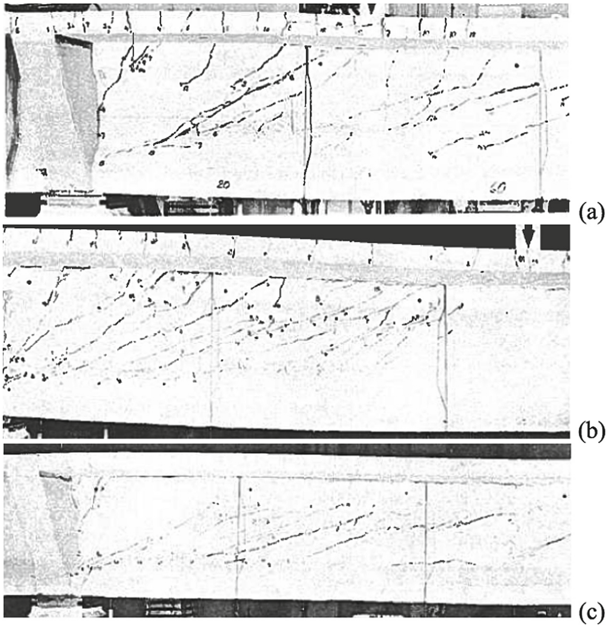

Fig. 2. Failure crack patterns of the tests S10 (a), S11 (b) and S12 (c). The photos are from [9].

The experimental test setup allows us to model the continuous girder as a one-side clamped girder, replacing the cantilevered girder stub with horizontal displacement constraints over the height of the cross-section at the interior support. The concrete is modeled with quadrilateral shaped plane stress elements with an average size of 25 mm by 25 mm. In line with [3], these plane stress elements are based on quadratic interpolation and use a 3 × 3 full Gauss integration scheme. The reinforcement bars and prestressing strands are modeled with embedded reinforcement elements with 2-point Gauss integration. A perfect bond between the steel and concrete was assumed. Anchorage losses (at the ends of reinforcing bars) and the transmission of stresses (at the ends of prestressed strands) are taken into account, such that it results in a linear built-up of the stresses at the bar and strands ends. The cross-sectional area of the continuity reinforcement in the deck slab above the interior support was reduced by 20% to account for the effective flange width, according to Sect. 5.3.2.1 of the Dutch Annex of Eurocode 2 part 2 [11].

Table 2 summarizes the adopted material models and properties. The f_{cm} values are obtained from Table 1and the other parameter values are calculated according to [3].

The analyses are performed in two phases, considering (1) a construction phase where the unhardened deck slab acts as a load on the prefabricated girder without contributing to the girder's stiffness; and (2) a testing phase where the deck slab is hardened and the horizontal displacement constraints at the interior support are activated. In the second phase, the following loads are applied: the self-weight of the girder and deck slab, the prestressing, the concrete blocks weights, and the point loads (P). An incremental-iterative solution procedure is adopted, using a load-controlled method for the increment part and a regular Newton–Raphson method for the iteration part.

Table 2. Overview of the adopted material models.

Material	Property			
		Prefab. Girder	Diaphragm	Deck slab
Concrete[1]	Density ρ [kg/m^3]	2500		
	Young's Modulus [N/mm^2]	34125	27247	27085
	Poisson's ratio, ν	0.2		
	Compression curve	Parabolic		
	Compressive strength [N/mm^2], $f$cm	43.2	20.4	20.0
	Compr. Fracture energy [N/mm]	36.0	31.4	31.3
	Reduction of f_{cm} due to lateral cracking	Yes, with a maximum reduction of 60%		
	Type of tensile softening		Hordijk	
	Tensile strength [N/mm^2]	3.22	1.61	1.57
	Tensile fracture energy [N/mm]	0.144	0.126	0.125
	Crack band width estimator	Govindjee		
	Variable Poisson's ratio	Damage based		
		Prestressing strands	Ø6.35 mm rebars	Ø12.7 mm rebars
Prestressing and reinforcement steel[2]	Young's modulus [N/mm^2]	197880	191674	190985
	Yield strength [N/mm^2]	1731	341	451
	Ultimate strength [N/mm^2]	1931	560	620
	Yield strain [‰]	8.7	1.8	2.4
	Ultimate strain [%]	4.4	15.5	9.0

[1] Total strain based orthogonal rotating smeared crack model
[2] Hardening plasticity model with a Von Mises yield criterion

3.2 Adopted Interface Shear Models

In this paper, three modeling approaches to describe the shear behavior in the concrete-to-concrete interface between the prefabricated girder and cast-in-situ deck slab were considered. They are based on the low-, high- and best-estimate of the interface shear behavior, and adopt the interfaces which we denoted as respectively "dowel action only", "rigid" and "best-guess". In the approach with the "rigid" interface, the finite elements of the prefabricated girder and the deck slab at the interface share the same nodes and are therefore fully connected. The other two approaches ("best-guess" and "dowel action only") adopt zero-thickness quadratic interface elements between the girder and the deck slab with different shear stress – slip relations (see Fig. 3). Only nonlinearity in the tangential direction of the interface is considered. As a simplification, the interactions between the normal and the tangential direction are neglected. The "best-guess" (BG) shear stress – slip relation in Fig. 3 is based on the experimental shear stress - slip relations of Loov and Patnaik [6], albeit tailored to the shear tests of Mattock and Kaar. The "dowel action only" (DAO) shear stress – slip relation only considers dowel action based on the theoretical considerations of Randl [12], and neglects the contribution of shear friction. This model underestimates the shear stress transfer in the concrete-to-concrete interface and can be seen as a lower bound.

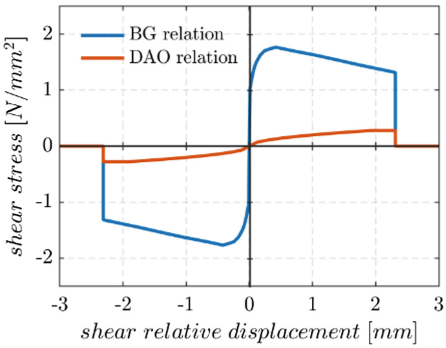

Fig. 3. The "best-guess" (BG) and "dowel action only" (DAO) shear stress - slip relations in the concrete-to-concrete interface.

4 Results

4.1 Impact of Interface Shear Models

The impact of the interface shear models on the structural behavior is investigated for test S11. Figure 4 (left) shows the point load P plotted against the midspan deflection obtained from the experiment and the three NLFEAs. The experimental failure load P_u is 156 kN and is underestimated in all the model predictions: for the analyses with the "rigid", "best-guess" and "dowel action only" interfaces, P_u is underestimated by respectively 7%, 4% and 27%. The predicted failure processes (incl. the order of events) in the analyses "rigid" and – particularly – "best-guess" match well with the experimental observations. Flexural cracks first appear above the interior support (see Fig. 5a). In the subsequent load steps a series of diagonal- tension cracks initiate in the shear span, though they do not fully develop as in the experiment (see Fig. 5b). Instead, the crack deformations of these diagonal cracks localize in a single major diagonal crack (see Fig. 5b–d). The shape and global orientation of the diagonal crack correspond well with the diagonal cracks observed in the test. Stirrups in this diagonal crack start to yield, but their strain values remain far below the ultimate strain. With increasing load, flexural cracks emerged and develop at midspan (Fig. 5b–d). When the failure load is approached, the continuity reinforcement start to yield (only in the "best-guess" analysis), and also the concrete-to-concrete interface reveal slip. In the last step, before the analyses stopped due to divergence, the concrete in the girder web near the interior support is locally close to a fully crushed state, i.e. the ultimate strain value under compression is almost reached. This indicates a diagonal compression failure in the simulations, as observed in the experiments (see Sect. 2.2).

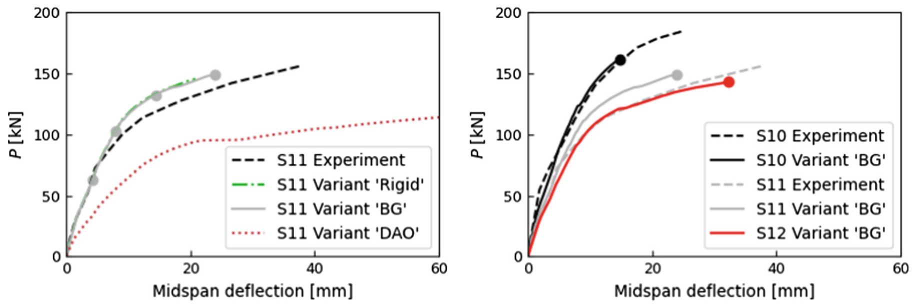

Fig. 4. Load-deflection curves of the experiments and NLFEAs. Left: test S11 and the predictions from three interface shear models. Right: tests S10–S12 and the predictions from the "best-guess" interface shear model. The experimental load-deflection curve of S12 was not reported in [9].

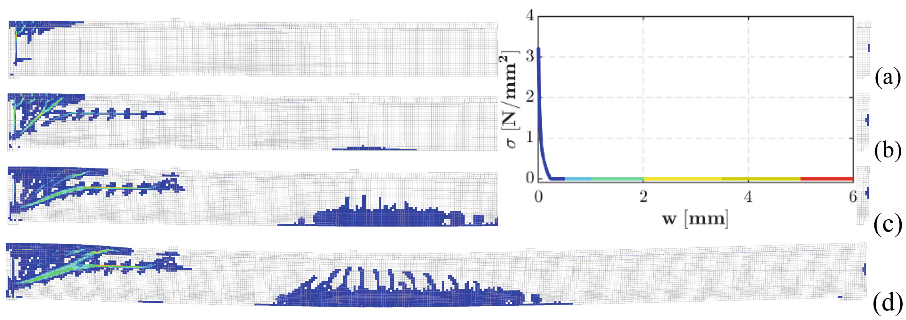

Fig. 5. Crack width plots in a deformed mode (with amplification factor: x5), showing the crack pattern development of the analysis "best-guess" of test S11. The shown plots correspond to the marked steps (gray dots) of Fig. 4 (left).

The analysis "dowel action only" reveals a totally different behavior. The failure process is dominated by a localized bending crack above the interior support and ends with a loss of the composite action between the prefabricated girder and the deck slab over a large distance. This behavior is indicative of an interface shear failure.

This small sensitivity analysis supports the use of the “best-guess” interface shear model, although the “rigid” model is almost as equally good. The “dowel action only” model clearly results in inaccurate predictions of the failure behavior and shear capacity.

4.2 Validation of the “Best-Guess” Interface Model on Tests S10 and S12

The “best-guess” interface shear model is used to simulate the tests S10 and S12. Figure 4 (right) shows its load-deflection curves, together with the experimental outcomes. Note that the load-deflection curve of S12 was not reported in [9], only the total shear force at the interior support at the time of failure is given (see Table 1). The experimental failure load P_u in S10 is 184 kN. The model prediction underestimates the failure load by 16%. For the test S12, the shear force near the support at failure was extracted from the model. This shear force is 6% lower than the experimentally obtained total shear force at failure V_u of 280 kN.

Figure 6 shows the final crack pattern of the three shear tests predicted with the “best-guess” model. The predicted events during the failure process for the tests S10 and S12 are in line with what we have seen for S11. This similarity is in agreement with the experimental observations. Nevertheless, there are some small differences. The diagonal cracks in the girder web of S10 are steeper compared to S11 and S12, and less widespread. Also the continuity reinforcement above the interior support did not yield. It indicates that the structural behavior is more governed by a direct load transfer from the loading plate to the interior support. In the results of the analysis of S12 we noticed that the continuity reinforcement yields at a lower load level, and also fully crushing of concrete near the interior support starts earlier and covers a larger region.

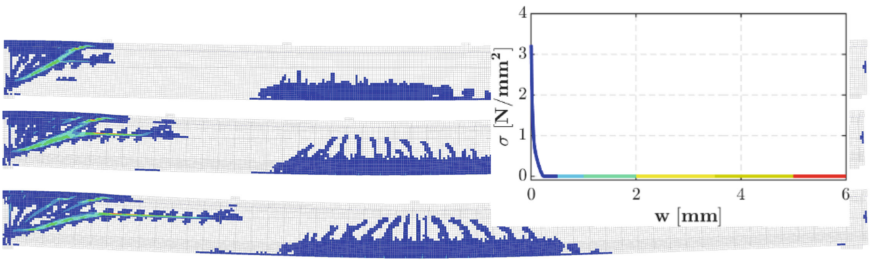

Fig. 6. Crack width plot in a deformed mode (with amplification factor: x5) at the last converged steps of the “best-guess” analyses of test S10 (a), S11 (b) and S12 (c).

5 Concluding Remarks

In this paper we validated a finite element modeling approach, based on the guidelines for NLFEA of concrete structures of Rijkswaterstaat, on three shear tests of continuous girders with a shear reinforcement ratio of 0.38% and various shear spans. The “best-guess” model provides safe side predictions of the shear capacities with 16% (S10), 4%

(S11), and 6% (S12) difference compared to the test results. Furthermore, the predicted events during the failure processes incl. the order in which they appeared, the concrete crack pattern development, and the failure mode are in good agreement with experimental observations.

These promising results give confidence in the use of NLFEA for the assessment of existing bridges with continuous prefabricated prestressed girder decks. Nevertheless, there are aspects that deserve further attention. For instance, the derivation of the "best-guess" shear stress – slip relation relies on a narrow scope of experiments that do not cover the full range of the variables typically found in the Dutch bridges. It is therefore important to gather and analyze more representative data on interface shear behavior, which can serve as input for the interface constitutive relation. Furthermore, it is recommended to extend the validation study, by considering a wider range of variables, such as the shear reinforcement ratio and bond quality of the interface.

Finally, it is noted that the failure behavior of continuous prefabricated prestressed girders is currently further investigated in a multi-year research program (incl. experimental, analytical and numerical research) at the Delft University of Technology.

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Author Contributions and Conflicts of Interest Statement. R.T. and A.S. performed the numerical analyses, evaluated the NLFEA results, drafted the main body of the text and prepared the figures. M.R. reviewed the work and provided feedback throughout the research. All authors have read and agreed to the published version of this paper. The authors declare no conflict of interest.

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