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Challenges on the shear behavior of existing continuous precast girder bridges

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Abstract

There are a large number of precast girder bridges in the Netherlands that are made continuous utilizing cast in situ layers and cross beams. When controlled by the Eurocode minimum shear reinforcement requirement, the majority of these bridges that were constructed before the 1970s have insufficient amount of transverse reinforcement, which could make them shear critical. Furthermore, when the continuity is created at intermediate support, the prestressing strands in the precast beams are usually located in the compression zone. This may limit the positive effect of the prestress to the shear capacity of members without transverse reinforcement. The aforementioned concern is not considered by the standard shear design approach of Eurocode. Because of its empirical nature, the shear capacity of these bridges cannot be rationally assessed by the code. Currently, an experimental campaign on full-scale 15m long specimens is underway at Delft University of Technology to investigate the existing challenges. In this paper, the effect of the above-mentioned concerns is illustrated using the predictions of Eurocode and the Dutch Guidelines for the Assessment of Existing Bridges (RBK). Moreover, the difficulties and insights gained from the design and execution of the experiments are discussed.

1 Introduction

Precast concrete girders are extensively used for the construction of bridges. Typically, the precast girders are designed as simply supported members and pre-tensioned with strand located in the bottom flange. As an alternative, the simply supported girders can be made continuous at the intermediate support by using cast in situ top layer and cross beam. This approach is used to build a large number of continuous precast bridges in the Netherlands.

In this system, the bridge girders function as a simply supported member to support their own weight. After establishing continuity at the intermediate supports, all other load types are carried as a continuous system. Hence, the maximum positive moment at mid-span can be reduced and the slenderness of the bridges can be increased due to the continuity. Fig.1 illustrates the construction sequence and shows the qualitative bending moment diagram for a continuous inverted T girder precast bridge. Fig. 1 also shows the interface reinforcements (hereafter referred as hairpin) that are used for the interface's shear strength.

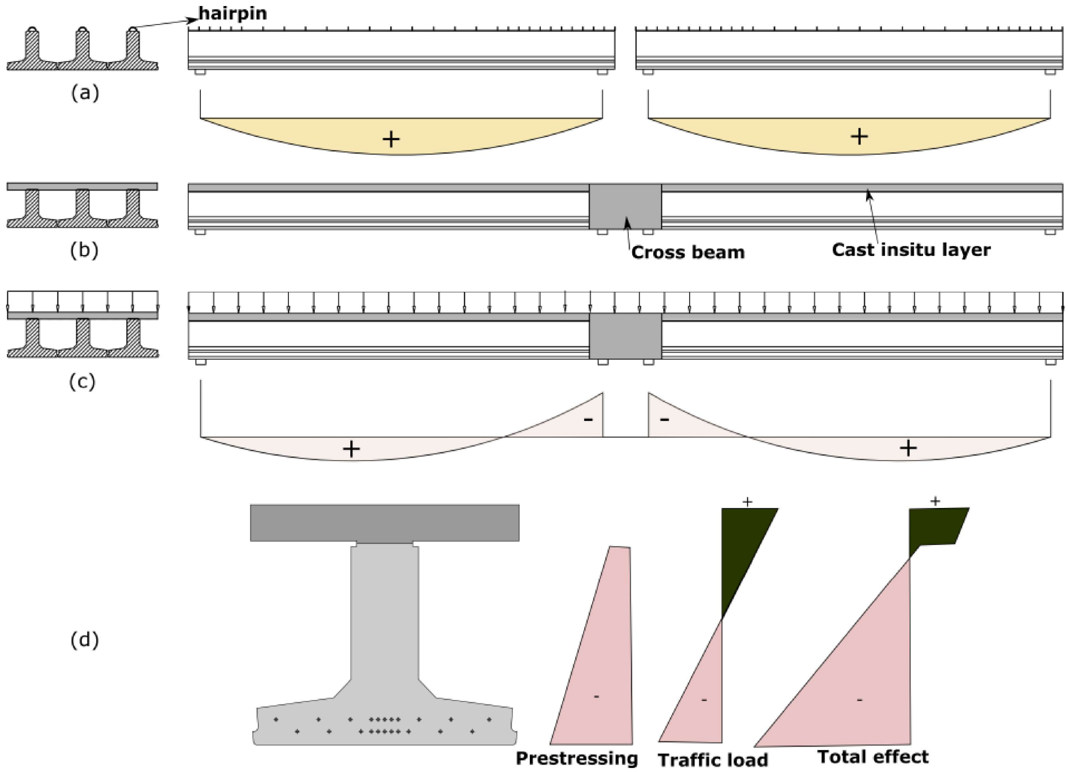


Fig. 1 (a) Precast inverted T girder before continuity (b) Continuous precast girder (c) response to traffic loading (UDL) (d) Strain distribution near the intermediate region

Although the reduced span moment and increase in slenderness are excellent advantages, there are several drawbacks to using this type of bridge. The primary problem is related to the assessment of the shear behavior. Before the 1970s, precast bridges were designed for shear by limiting the principal tensile stress in the web. Consequently, the girders were provided with a small amount of web reinforcement. When compared with the Eurocode [1] minimum shear reinforcement requirement, the majority of these bridges were constructed with an insufficient amount of transverse reinforcement, which could make them shear critical.

Additional to limited web reinforcement, when the bridges were made continuous, the prestressing strands on precast beams is located in the compression zone of the cross-sections near the intermediate support region(see Fig 1d). This may limit the positive effect of the prestress to the shear capacity of members without transverse reinforcement. Due to the mismatch between the location of the strands and flexural cracking zone, it will be difficult to answer to what extent the positive effect of the prestress on the shear capacity shall be considered.

Generally, the shear strength of the precast girders is evaluated by comparing the action effect against the shear resistance namely flexural shear or shear tension capacities. For example, Eurocode [1] provides two separate equations for shear assessment based on whether the section is cracked by bending. A similar approach is also used by ACI 318-19 [2] as the nominal shear capacity is taken as the lesser of flexural shear or web shear strength. Given that only the precast girders are prestressed in the current continuous members (not the topping layer), the section near the intermediate support region is expected to be cracked by bending. As a result, the focus of this paper is solely on the assessment of these bridges for flexural shear capacity. The shear tension (web shear) assessment is required for sections that are not cracked by bending, and further discussion on the subject is beyond the scope.

In this paper, using a representative precast member, the difficulty in assessing the shear capacity using Eurocode as well as the Dutch Guidelines for the Assessment of Bridges (RBK) [3] will be demonstrated. Guided by the knowledge gap and urgency of the challenge, a comprehensive experimental campaign on full-scale specimens is currently underway at Delft University of technology. The paper will present the challenges and insight gained through the design, construction, and execution of the large-scale experiments.

2 Shear assessment of the intermediate support regions

2.1 Code formulations

The shear capacity of continuous precast girders can be evaluated using the Eurocode formula. The code uses an empirical equation for the flexural shear capacity of members without shear reinforcement and a variable angle truss model for members with sufficient reinforcement.

In the Netherlands, the shear assessments of concrete bridges is carried out according to the Dutch Guidelines for the Assessment of Existing Bridges (RBK) [3]. RBK uses the general Eurocode expression for the flexural shear capacity provided by the concrete and it combines the concrete term with the contribution of the stirrups to evaluate the ultimate capacity. In contrast to Eurocode, which employs a variable angle for strut inclination, RBK uses a fixed strut angle. To facilitate the subsequent discussion, the summary of the shear equations is presented in Table 1 and additional details can be referred from the codes ([1] and[3]).

Table 1 Summary of the code equations.

Description	EUROCODE	RBK
$V_{Rd,c}$	$\left[\frac{0.18}{\gamma_c}k\left(100\rho_l f_{ck}\right)^{1/3}+0.15\sigma_{cp}\right]bd$ (1)	$\left[0.12k_{cap}k\left(100\rho_l f_{ck}\right)^{1/3}+0.15\sigma_{cp}\right]b_{wgen}d$ (2)
$V_{Rd,s}$	$\frac{A_{sw}}{s}zf_{ywd}\cot\theta$ (3) $1 \leq \cot\theta \leq 2.5$	$\frac{A_{sw}}{s}zf_{ywd}\cot\theta$ (4) $\theta = 30^\circ$ for prestressed members
V_{Rd}	$\begin{Bmatrix} V_{Rd,c} & \rho_w < \rho_{\min} \\ V_{Rd,s} & \rho_w \geq \rho_{\min} \end{Bmatrix}$ (5)	$V_{Rd,c}+V_{Rd,s}$ (6)

Before delving into examination of the codes, it is vital to discuss the background of the Eurocode shear equation, particularly the concrete contribution. The Eurocode equation (see equation 1) is an empirical formula calibrated using experimental data of members that are simply supported, both with and without prestressing [4]. In the formulation, the basis for the positive effect of prestress is attributed to the assumption that the prestressed member can be considered a reinforced concrete member when the decompression moment is reached. This rationale is used to support the empirically derived shear contribution of the prestress through an equivalent central prestress (see Table 1).

Although the application of Eurocode equation seems straightforward, there will be a mismatch in the model assumption for intermediate zones when the strands are located in the compression zone (see Figure 1d). The stress state that is assumed in the code cannot be found near the intermediate region and this makes it questionable to consider the substantial capacity that can be contributed from the prestress part.

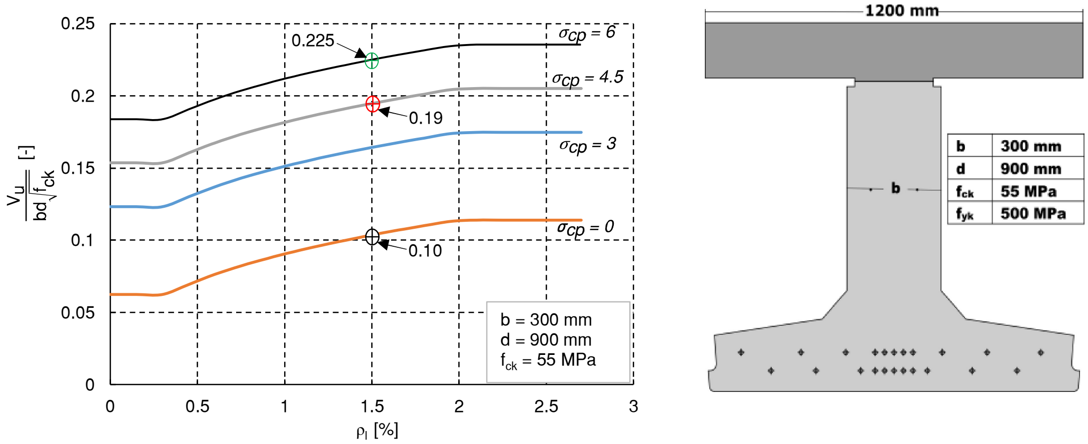
Due to the incompatibility in the assumption and to have a safe shear capacity estimation, structural engineers are currently required to use crude engineering judgments, such as disregarding the positive effect of the prestress. This type of assumption is not rational and it usually provides an overly conservative assessment.

2.2 Detail analysis of the codes

The effect of the above-mentioned challenges is further illustrated using the capacity prediction curves of the codes (see Fig. 2 and 3). The following assessment will be done using a typical precast type and concrete grade that is commonly used in the construction of continuous precast girders.

An inverted T-girder section with a web width of 300 mm and height of 1070 mm is chosen (see Fig. 2b). The effective depth of the girder is 900 mm and cast from a concrete grade of C55/67. The selected section is located in the intermediate support region in which the prestress strands are located in the bottom flange. The reinforcement in the cast-in-situ layer is considered to act as the tensile reinforcement. Furthermore, the shear and tensile reinforcement are ribbed bars with a grade of B500B.

Fig. 2a presents the relationship between normalized shear capacity and the amount of longitudinal tension reinforcement. Both Eurocode and RBK essentially give the same response for members without shear reinforcement. As a result, only the Eurocode prediction is shown for various levels of prestress.



(a) (b)

Fig. 2 (a) shear capacity against reinforcement ratio for varying prestress level (b) Inverted T girder section

To observe the difficulty of assessing intermediate regions, consider a member with prestressing level of 4.5 MPa and 1.5% tension reinforcement. The presented plot indicates the normalized capacity of the section is 0.19. As described earlier, the assumption used in the code is incompatible with the condition at the intermediate support. If the prestress is neglected, the normalized capacity for the regions in the intermediate support is 0.10, which is 52 percent of the initial capacity with the prestress. In most cases, this type of crude assumption will make the member be classified as shear critical. Because higher prestress levels result in higher shear capacity, neglecting the prestress penalizes those members the most.

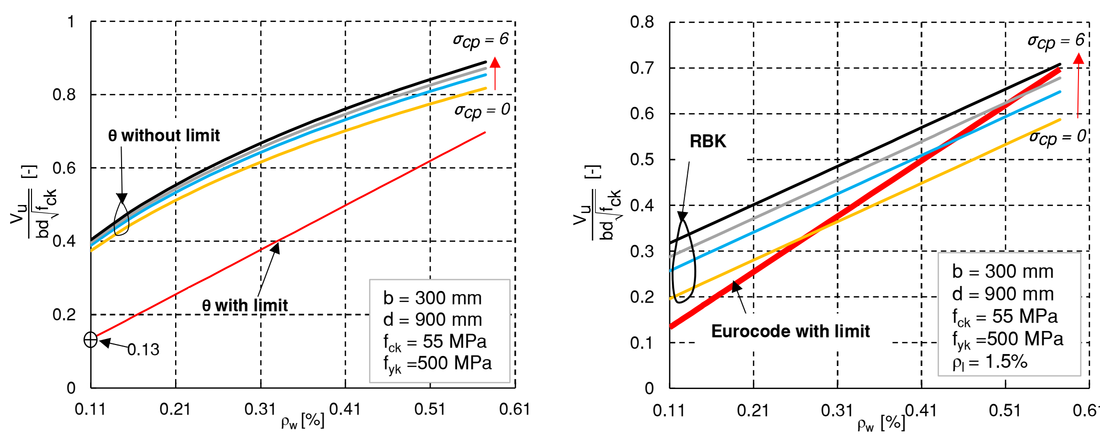


Fig. 3 (a) Eurocode analysis with and without considering the limit (b) RBK against Eurocode with the limit

The primary difference between the Eurocode and RBK can be observed for members with shear reinforcement, particularly for members with small amounts of reinforcement. The Eurocode equation for members with shear reinforcement is independent of longitudinal reinforcement amount. Because the concrete and reinforcement contributions are combined in RBK, a longitudinal reinforcement design must be chosen to ensure a consistent comparison between the codes.

Fig. 3 presents plots indicating the relationship between the shear reinforcement ratio and normalized shear capacity using Eurocode and RBK. The shear reinforcement ratio begins with the smallest shear reinforcement allowed by the code and similar to the previous analysis the plots are prepared for different levels of prestress.

The shear capacity in Eurocode can be determined by using the compression field angle that results in the crushing of the strut. Furthermore, the code restricts the calculated angle of the distributed struts to be between 21.8 to 45 degrees. Although the strut angles are evaluated for selected reinforcement ratios, in most cases the calculated angle is less than 21.8 degrees and the minimum limit will govern the assessment. The implication of the minimum imposed limits on the capacity prediction is presented in Figure 3a. The plot presents two analyses, one considering the limit ($\theta \geq 21.8$), while the other is without considering the code limitation.

As it can be seen from Fig. 3a, when the code restriction is considered, the prestress level will not have any effect on members with small amount of shear reinforcement. On the contrary, if the limit is not followed and smaller angles($\theta < 21.8$) are allowed, the prestress slightly affects the response. In addition, when compared with the analysis with the strut angle limit, it gives a higher shear capacity.

Another interesting observation can be found by comparing Figure 3a with Figure 2a. Consider a member reinforced with the minimum shear reinforcement, 1.5% longitudinal rebar, and presstress level 6 MPa. According to Eurocode analysis with limit, the normalized capacity is 0.13. If the shear reinforcement is not considered, Fig 2a indicates the normalized capacity of the member is 0.225, which is more than 70% higher. According to the comparison, in some instances the code demonstrates a member without shear reinforcement can have a higher capacity than a member with at least the minimum reinforcement. This behaviour conflicts with the rationale behind providing minimum shear reinforcement. Observing the analysis for the member with and without limit (see Fig 3a), it will be obvious that the reason for this discontinuity is the code's restriction on minimum strut inclination. While the limitation serves other purposes, it also creates discontinuity for such situations.

Fig. 3b compares Eurocode and RBK for members with at least minimum shear reinforcement. The plot is prepared by considering a longitudinal reinforcement ratio of 1.5%. Since the concrete contribution is included, the RBK gives a higher capacity for members with minimum reinforcement, and the capacity increases with the increase of the prestress. For higher shear reinforcement ratio, the Eurocode gives higher capacity regardless of the prestress level. This decline in the capacity of the RBK is mainly attributed to the fixed angle that is used for strut inclination (see Equation 4).

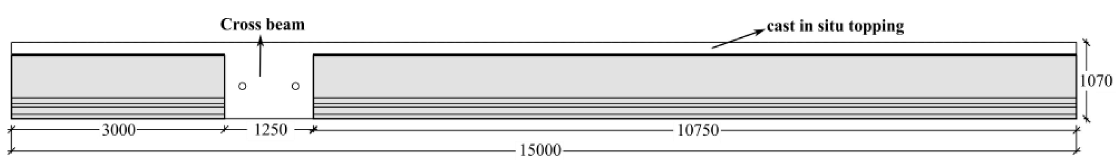
Even though the prestressing strands are located in the flexural compression zone, it is plausible to assume that there still is certain positive effect from the prestress. However, the empirical nature of the code formulation on the other hand makes it difficult to analyse this type of situation rationally. The difficulty of shear assessment is also further exacerbated by the scarcity of representative full-scale experiments. There are limited sets of experiments done on continuous girders to study the behavior at the intermediate support. These investigations are mainly focused on either the flexural behavior of the system [5] or addressed specimens with sufficient amount of shear reinforcement [6]. While these seminal works enhanced the understanding of continuous girders, the experiments are not comparable to the current existing precast girders with limited shear reinforcement.

As demonstrated from the above comparisons, the shear assessment of the continuous girders at the intermediate support is complicated and this calls for further investigation. Therefore, to address the difficulties and knowledge gaps, a comprehensive experimental campaign is currently underway on precast continuous girders with limited amount of shear reinforcement.

3 Challenges of conducting large scale experiments on continuous girder

3.1 General description of the specimens

The shear behavior of continuous precast girders is being investigated using full-scale specimens. The specimens are inverted T-girders with a web width of 300 mm and a depth of 900 mm. When the continuity is created, the total length of the specimen is 15 m, and they weigh more than 26 tons (See Fig. 4a and b).



(a)

(b)

Fig. 4 (a) Specimen details (all dimensions are in mm) (b) Specimens before connection (note the location of the strands and the hairpins)

The continuity at the intermediate support is created using a cast in situ topping layer with a thickness of 160 mm and a crossbeam with a width of 1250 mm (see Fig. 4a). Consistent with past practices, the interface shear transfer is maintained by using a rough surface and hairpin reinforcement. Since the prestressing strands are located in the bottom flange, once the continuity is formed at the intermediate zone, the prestressing effect will be on the flexural compression zone.

The experimental campaign aims to investigate the shear responses of these continuous members with and without sufficient shear reinforcement. Since the main aim of the current paper is to describe the challenges of designing and executing large-scale experiments, particular details pertaining to each specimen will not be discussed here. Rather, the unique challenge presented by the specimen and the difficulties and insights gained from the design and execution of the experiments will be emphasized. The unique challenges are divided into aspects during the design of the specimens and execution. Both challenges will be discussed in the following section.

3.2 Design challenges

In the experiment, the main investigated region is located between the intermediate support and span loading point (see Fig. 5). The loading will be applied using two hydraulic jacks located in the main span and the cantilever region. During the experiment, besides the shear failure at the support region, other premature failures may arise in the continuous girder. These include flexural and shear failure near the mid-span and flexural failure in the support region. During the design of these type members, sufficient over strength against premature failure must be provided to ensure the anticipated shear failure near the intermediate support region.

The investigated region will have a constant shear force during the experiment. This may result in competing shear failures that may occur near the support or close to the main span loading zone (see Fig. 5). Because of the position of the prestressing strands, the shear capacity of the mid-span zone is expected to be higher than the support end. On the contrary, the presence of the hairpin near the support region may increase the member's shear capacity. Although the hairpins are provided for interface shear strength, a study on dowel spliting by Chana [7] indicated suppressing the dowel splitting can enhance the shear capacity of members. Therefore, to avoid this type of shifting in shear failure from support to the span, a detailed nonlinear analysis is conducted for the specimens. This type of prior analysis helps to anticipate and propose a solution for possible premature failure during the experiment.

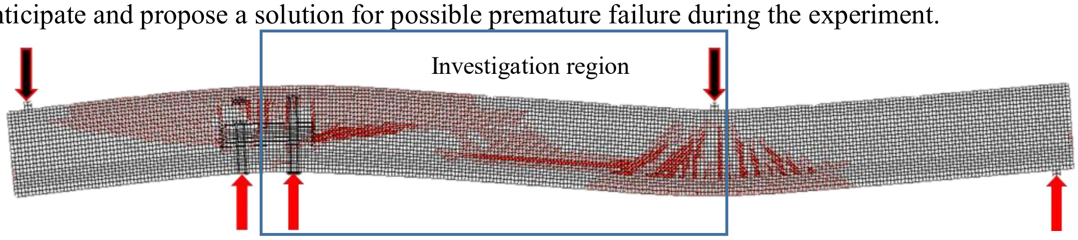


Fig. 5 Competing shear failure modes on the continuous girder

Generally speaking, designing large-scale experiments requires satisfying many constraints. The primary challenge is to accurately represent the characteristics of the existing girders. Since it is not possible to replicate the whole behaviour, on a scaled experiment, different competing failure modes may occur that are unique to the laboratory specimen. These competing failure modes shall be adequately assessed before the detailed design and casting of the specimens.

3.3 Construction and execution challenges

The inverted T-girder specimens are individually cast in a precast factory and later they are connected after a certain period. Between the girder casting and testing in the laboratory, there will be time-dependent prestress losses. Previously, researchers utilized concrete strain gauges on precast girders to monitor the concrete strain during prestress transfer [8]. The strain gauges are usually attached to the concrete surface and during the prestress release, the strain variation along the length can be monitored. Although the method is suitable to monitor transfer length, it would be difficult to monitor the prestress loss during the time between casting and testing. Furthermore, this method can only be reliably used in a laboratory setting for relatively smaller specimens. For specimens that are cast in a precast factory, in most cases, the surface of the specimens is not accessible to monitor using surface strain gauges.

In the current research, to monitor both the prestress transfer and time-dependent loss, smart rebar with fiber optics sensors are embedded in the specimens. The smart rebar can be used to monitor the strain change in the concrete resulting from mechanical or time-dependent actions. Because the sensors are embedded, they are also well protected from the damages arising from the subsequent process such as lifting, transportation, and continuity creation.

Before establishing continuity at the intermediate region, special attention must be given to the support condition. The boundary conditions of the test specimen should be similar to those of existing bridges. Furthermore, the boundary conditions that are used during continuity (casting) should also be maintained until the specimens are tested in the laboratory. The main reason for maintaining the boundary conditions is the interface that exists between the girders and the topping layer. The interface behaviour highly affects the in-plane shear response of the specimens near the intermediate support. Accidental or intentional changes in support conditions can result in unmonitored cracking of the interface. To avoid interface cracking, a special frame is designed and constructed. The steel frame is designed to protect the specimens during the lifting, and transportation of the specimens from the casting to the laboratory support location.

The protection frame will be clamped to the girder at the support location near the cross beam and outside of the investigated region with high-strength prestressing bars. The frame is also constructed with an additional clamp in the middle of the girder. The middle clamp will only be used after the testing of the specimens to assist in the safe disposal of the specimens.

The lifting of the specimen will be done through additional bottom supports that are placed between the bottom clamping sections. The frame together with the continuous girders is shown in Fig. 6 and by employing this method the specimens can be safely lifted, transported, and tested while maintaining the same boundary condition.

Fig. 6 Specimen with the protection frame

4 Conclusion

The shear assessment of continuous members is increasingly becoming very important. Although the application of the current Eurocode and RBK codes seem straightforward, the assessment of the intermediate region is complicated by several factors. Since the codes are empirical by their nature, gross assumption is usually taken during the assessments of these types of members. The outcome of the assessment usually indicates the member to be shear critical. However, without a proper understanding of the shear behaviour, the analysis obtained from the code predictions is unreliable. To address the knowledge gaps, a comprehensive experimental campaign on large-scale members is currently underway. Some of the conclusions on lessons learned during the assessment of the design codes and execution of the full-scale experiments include:

- The Eurocode and RBK are not suitable to assess the intermediate region of precast continues members when the strands are located in the flexural compression zone
- In some extreme situations, Eurocode provides an irrational higher shear capacity for a prestressed member without shear reinforcement than for the same member with minimum shear reinforcement.
- For continuous precast experimental specimens, the presence of an interface necessitates maintaining the same boundary condition from continuity until the testing phase. An external protection device that is used in the current campaign can reduce uncertainties that may affect the shear behavior of the specimen.

Acknowledgements

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