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Publication date

2016

Document Version

Accepted author manuscript

Published in

fib symposium 2016, performance-based approaches for concrete structures

Citation (APA)

Lantsoght, E., van der Veen, C., & Boer, A. D. (2016). Shear capacity of the Ruytenschildt bridge. In H. Beushausen (Ed.), *fib symposium 2016, performance-based approaches for concrete structures : Cape Town, South Africa* (pp. 1-11)

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SHEAR CAPACITY OF THE RUYTENSCHILDT BRIDGE

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ABSTRACT

In August 2014, the Ruytenschildt Bridge, a reinforced concrete solid slab bridge (reinforced with plain bars) in the Friesland province in the Netherlands was tested until failure. One of the goals of proof loading and testing this bridge to failure, was to study the failure mode of existing slab bridges. The combination of smaller shear capacities as prescribed by the Eurocode in combination with the heavier live load models, has raised concerns with regard to a number of existing slab bridges in the Netherlands. As the shear capacity of existing bridges is under study, the results of testing an actual slab bridge until failure are used to compare to the results of testing half-scale slab specimens in the laboratory, and the conclusions resulting from those experiments. In this paper, the results of the predictions based on the first order of approximation rating procedure from the Netherlands for shear, the Quick Scan method, as well as based on predictions of the failure mode and the average predicted capacity are compared to the experimental results. The predictions show a possibility of shear failure in the second span of the bridge. The experiment showed that both spans of the bridge failed in flexure. The observed failure mode is important, as some of the results indicate that the solid slab bridges, currently under discussion with regard to their shear capacity, fail in flexure in reality. Flexural failure is considered a ductile failure compared to the brittle failure mode in case of a shear failure.

Keywords: bridges, field testing, load testing, shear, slabs.

1. Introduction

The majority of the bridges in The Netherlands were built during the decades following the Second World War. These bridges were designed for the live loads of that era, which are considerably lower than the current live loads (see NEN-EN 1991-2:2003 (CEN 2003)). In NEN-EN 1992-1-1:2005 (CEN 2005) the calculated shear capacity of a cross-section typically results in a lower value than according to the previously used NEN 6720:1995 (Code Committee 351001 1995). As a result, upon first assessment, a large number of existing Dutch reinforced concrete solid slab bridges are found to be insufficient for shear (Lantsoght et al. 2013b).

In the *fib* Model Code 2010 (fib 2012), the concept of Levels of Approximation is introduced. Increasing the Level of Approximation increases the computational time, but also results in an estimation of the capacity that is expected to be closer to the “real” capacity of the element under consideration. Levels of Approximation are also used in The Netherlands for the shear assessment of existing concrete structures, resulting in the so-called Levels of Assessment. The first Level of Assessment is the “Quick Scan” (Lantsoght et al. 2013b, Vergoossen et al. 2013), a conservative spreadsheet-based method that results in a “Unity Check”: the ratio of the sectional shear stress to the shear capacity. Currently, the Quick Scan only considers shear, but it will be extended for bending moment. If the Unity Check is larger than 1, the analysis has to be repeated at Level of Assessment II. At this Level, the shear stress distribution over the width of the support is determined with a linear elastic finite element program. The peak shear stress is then averaged over $4d_i$ (Lantsoght et al. 2013a, Lantsoght et al. 2014a) (with d_i the effective depth to the longitudinal reinforcement) and compared to the shear capacity. If this value is again larger than 1, the

procedure is repeated at Level of Assessment III, which uses probabilistic analyses. Level of Assessment IV contains advanced non-linear finite element calculations and proof loading.

2. Literature review

2.1 Shear capacity of reinforced concrete slabs

As the shear capacity of existing bridges is under study, the results of testing an actual slab bridge until failure will be used to compare to the results of testing half-scale slab specimens in the laboratory. These tests were carried out at Delft University of Technology on slab specimens under concentrated loads close to supports (Lantsoght et al. 2013c, Lantsoght et al. 2014b). It was concluded that slabs have additional capacity as a result of transverse load redistribution (Lantsoght et al. (in press)) when compared to beams.

Laboratory tests are a schematization and simplification of a full bridge. Therefore, field testing of an existing slab bridge was planned. The Ruytenschildt Bridge, scheduled for demolition, was offered by the province of Friesland for a field test to failure.

2.2 Loading testing to failure

In the past, only a limited number of bridges have been tested to failure. An overview of those known by the authors is given in Table 1. It can be noted that the majority of these bridges were slab bridges, mostly resulting in a flexural failure. The testing of the Thurloxton underpass (Cullington et al. 1996) is not included in Table 1, because the test was carried out after applying saw cuts over 1 m, which does not give information about the behavior of the bridge as an entire structure.

Table 1. Overview of past testing to failure on bridges

Reference	Bridge name	Type of bridge	Failure mode
(Haritos et al. 2000)	Barr Creek	slab bridge	Flexural failure
(Azizinamini et al. 1994a, Azizinamini et al. 1994b)	Niobrara River	slab bridge	Flexural failure
(Aktan et al. 1992, Miller et al. 1994)	-	slab bridge	Punching failure
(Jorgenson and Larson 1976)	ND-18	slab bridge	Flexural failure
(Bagge et al. 2015a, Bagge et al. 2015b)	Kiruna	prestressed girder bridge	Punching failure

3. Description of the Ruytenschildt Bridge

3.1 Location and history

The Ruytenschildt Bridge is located in the province of Friesland (The Netherlands), over a waterway connecting the Tjeuker Lake to the Vierhuister Course. The bridge carries the national road N924, which connects the villages of Lemmer and Heerenveen. The bridge was built in 1962, and was scheduled for demolition and replacement by a bridge with a larger clearance, allowing for the passage of larger boats. The carriageway was divided into two lanes and a bike lane. In August 2014, the Ruytenschildt Bridge was tested until failure by a proof loading in situ.

3.2 Structural system

The Ruytenschildt Bridge is a solid slab bridge with five spans. The bridge is an integral bridge. At the supports, cross-beams are cast integrally onto the piers. The bridge had a skew angle of 18°. The geometry is shown in Fig. 1.

The availability of at least one motor lane and a bicycle lane was needed during demolition and reconstruction. Therefore, the demolition of the bridge was planned in stages. In the first stage, 7.365 m of the bridge was demolished and the remaining 4.635 m served as one traffic lane. A saw cut between both parts was made, and the eastern part of the bridge of 7.365 m wide was tested to failure in spans 1 and 2. A separate temporary bike bridge on pontoons was provided.

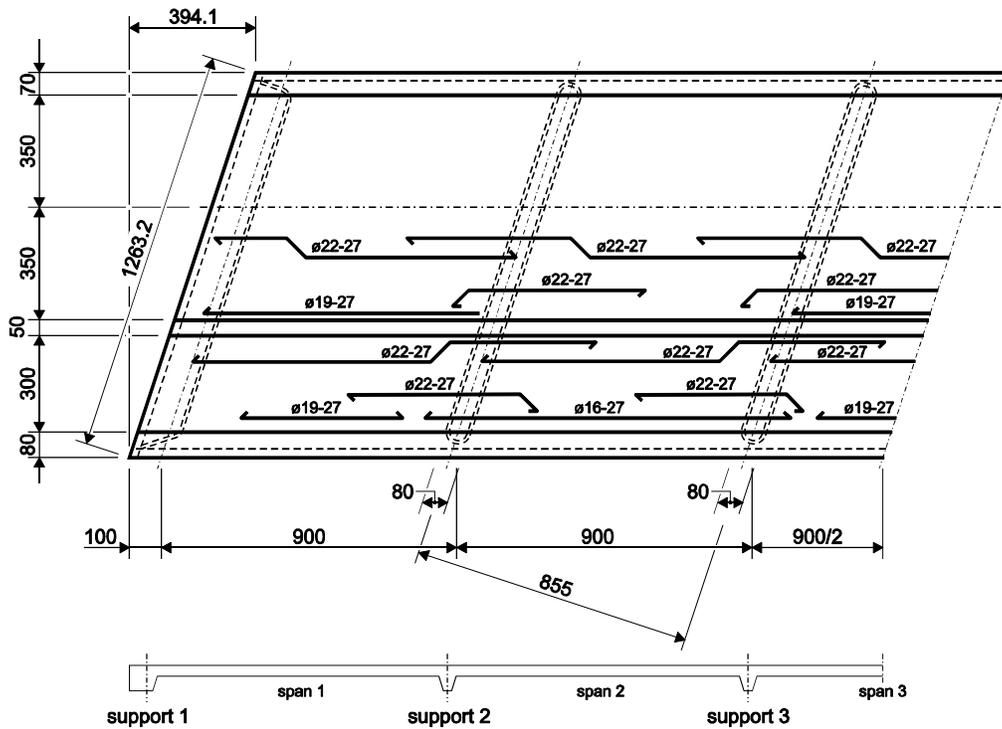


Fig. 2. Reinforcement drawing of Ruytenschildt Bridge, showing spans 1, 2 and half of span 3, and showing the location of the sawn beams. The structure is symmetric. Distance units in cm and bar diameters in mm.

3.3 Material properties

62 cores were drilled to test the concrete compressive strength, showing that the average cube compressive strength was $f_{cm} = 40$ MPa. The characteristic concrete compressive strength was $f_{ck} = 25.4$ MPa (Lantsoght 2015).

Additionally, 31 concrete cores were drilled from beams sawn from the bridge that were tested later in the laboratory for further research (Yang 2015). However, not all were drilled vertically. These additional tests provide a calibration factor for the poor surface treatment of the previously tested cores, resulting in $f_{cm} = 63$ MPa, which corresponds to a cylinder compressive strength $f_{cm,cyl} = 52$ MPa. On the cores drilled from the bridge, the thickness of the asphalt layer was measured as 51 mm. The thickness of the asphalt layer is necessary as input for the Quick Scan sheet and for determining the average predicted shear capacity.

Reinforcement steel QR24 was used, which is tabulated as having a characteristic yield strength $f_{yk} = 240$ MPa. Tensile tests on steel samples taken from the structure showed an average yield strength $f_y = 352$ MPa and an average tensile strength $f_t = 435$ MPa for the bars with a diameter \varnothing of 12 mm and $f_y = 309$ MPa and $f_t = 360$ MPa for $\varnothing = 22$ mm. The samples were taken from the bridge after testing, so that yielding of the steel could have occurred, influencing the measured material properties. Past testing of QR24 steel from a similar bridge that was not tested to failure resulted in a yield strength $f_y = 283$ MPa (Yang et al. 2010).

3.4 Reinforcement layout

The slab thickness was about 550 mm, as shown in Fig. 1. The reinforcement layout of one half the symmetric bridge structure is shown in Fig. 2. In span 1, the area of the bottom reinforcement is calculated as $A_s = 3866\text{mm}^2$. The top reinforcement at support 2 is not very clear due to the configuration with bent-up bars. Therefore, both $A_s = 1408\text{mm}^2$ and $A_s = 5632\text{mm}^2$ are used for the analysis.

4. Prediction of the shear capacity

4.1 Quick Scan method

Before the proof loading and testing until failure (Lantsoght 2014), the Quick Scan was determined as a function of the concrete compressive strength, since this value was unknown at that time. After the experiment, the material properties were determined. The analysis is carried out with the Excel spreadsheet of the Dutch Ministry of Infrastructure and the Environment (Rijkswaterstaat 2013a, Vergoossen et al. 2013). Here, the results for the Quick Scan are determined with the a posteriori determined material properties:

- $f_{ck} = 25,4$ MPa as determined from the cores
- $f_{yk} = 309$ MPa: as measured, but not enough measurements are available to find an average and standard deviation
- $f_{yd} = 267$ MPa: a material factor of 1,15 is taken into account

The skew of the viaduct is taken into account by increasing the effect of the load based on skew factors (Rijkswaterstaat 2013c). All calculations are carried out assuming a straight viaduct, and then the skew factors are added to increase the shear stresses. All load factors are taken according to NEN 8700:2011 (Code Committee 351001 2011).

The spreadsheet calculated the Unity Check of every cross-section between the two supports of the considered span and accounts for continuity by modeling up to 5 spans of the structure most adjacent to the span under study. The reinforcement is modeled in a detailed way: for every layer of reinforcement the diameter, center-to-center distance and position of beginning and end of the bars need to be entered, and the sheet checks if an additional check for anchorage of the reinforcement should be made. The sheet then calculates the ratio of the top and bottom reinforcement for every cross-section, as shown in Fig. 3a.

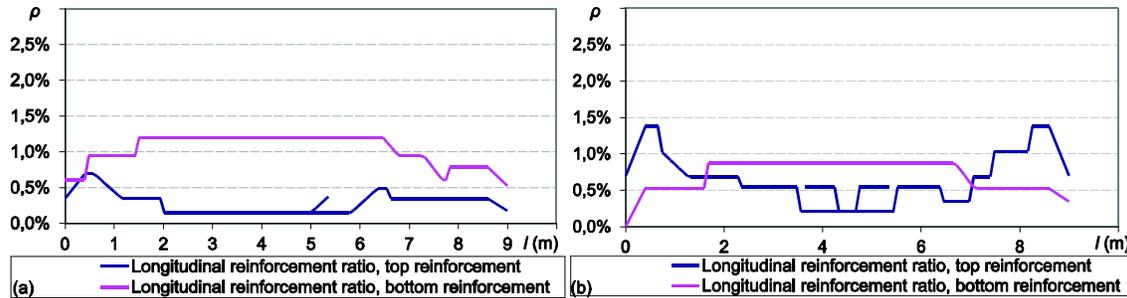


Fig. 3. Calculated reinforcement percentages: (a) Span 1; (b) Span 2.

The Unity Check for every cross-section assuming concrete class C35/45 is then calculated, Fig. 4. For Level of Assessment I, when material parameters are not available, the concrete class is always assumed to be C35/45 (Steenbergen and Vervuurt 2012). The values for the Unity Check for the lower concrete compressive strength as tested are also given in the sheet, but are not presented as a graph. The results show that the Unity Check of span 1 using the measured material parameters is largest close to support 2, resulting in $UC = 1.14$ (at $x = 8.6$ m; $UC = 0.88$ for C35/45). At the face of support 1, $UC = 0.81$. The anchorage detailing is insufficient.

Similar calculations are then carried out for span 2, with the reinforcement percentage along the length as shown in Fig. 3b and the resulting Unity Checks for C35/45 as shown in Fig. 4b. For the measured material properties, the maximum Unity Check equals 1.04 for $x = 1.3$ m ($UC = 0.78$ for C35/45). At support 2-3, $UC = 0.95$ and at support 3-2, $UC = 0.91$. The maximum Unity Check is now not found close to the support, because of the changing reinforcement layout.

Based on the results of the Quick Scan Excel, it is concluded that the shear capacity of the viaduct is insufficient and that further analyses with higher Levels of Assessment are necessary. These observations are in line with calculations of the bridge owner, the Province of Friesland, that the shear capacity of the viaduct is insufficient (Provincie Fryslân 2010).

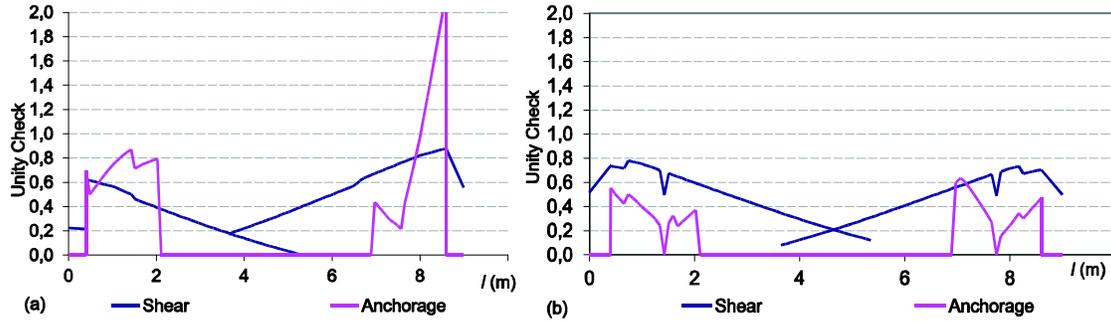


Fig. 4. Unity Check along the span length assuming C35/45: (a) for span 1; (b) for span 2.

4.2 Average predicted capacity

To predict the capacity of the tested cross-sections, calculations were performed with average material parameters. The characteristic shear capacity from NEN-EN 1992-1-1:2005 §6.2.2. (CEN 2005) can be converted into an average shear capacity by using f_{cm} and $C_{Rm,c} = 0.15$ (Yang and den Uijl 2011):

$$v_{R,c} = C_{Rm,c} k (100 \rho_l f_{cm})^{1/3} \quad (1)$$

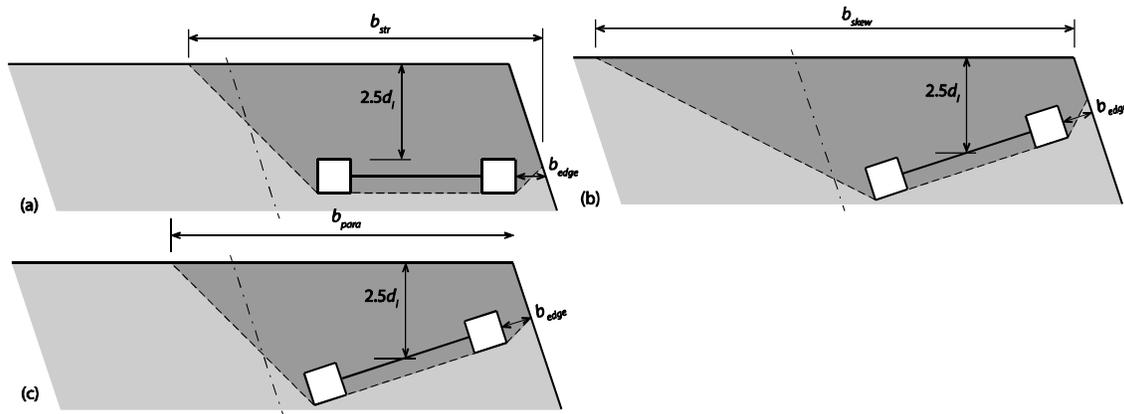


Fig. 5. Different possible effective widths for a skewed slab.

For skewed slabs, the determination of the effective width in shear is ambiguous. Three options have been studied, as shown in Fig. 5:

- b_{str} , the effective width for a straight slab,
- b_{skew} with horizontal load spreading under 45° from the far side of the wheel print to the face of the support (Lantsoght et al. in press), and
- b_{para} based on a parallel load spreading to the straight case, where the limitation because of the edge is considered.

The maximum total tandem load is then calculated assuming average material parameters and all load and resistance factors equal to 1. The maximum load is sought so that the resulting shear stress at the support from all occurring loads (dead load, superimposed load, proof load truck with 4 wheel prints) equals the shear capacity from Eq. (1). The maximum calculated tandem load is given as P_{tot} in Table 2. From the slab shear experiments (Lantsoght et al. 2013c), the ratio of the tested to predicted (Eq. 1) shear capacity was found to be 2.023 (Lantsoght et al. 2015), mainly caused by transverse load redistribution, resulting in the prediction $P_{tot,slab}$ in Table 2. This multiplication factor was derived in experiments on straight slabs.

Skewed slabs, on the other hand, have larger stress concentrations in the obtuse corner, which results in smaller capacities (Cope et al. 1983). Therefore, the shear capacity is estimated in between P_{tot} and $P_{tot,slab}$.

Table 2. Shear capacity of span 1 and span 2 of Ruytenschildt Bridge for three different load spreading methods.

Span	Span 1		Span 2	
	P_{tot} (kN)	$P_{tot,slab}$ (kN)	P_{tot} (kN)	$P_{tot,slab}$ (kN)
b_{str}	3760	7606	4020	8132
b_{para}	3236	6546	3432	6943
b_{skew}	4804	9718	5328	10779

5. Testing of the Ruytenschildt Bridge and results

5.1 Test setup

The geometry of the tandem load of NEN-EN 1991-2:2003 (CEN 2003) was used for the test, with wheel prints of 400 mm × 400 mm, a distance along the width between the wheels of 2 m and an axle distance of 1.2 m. The face-to-face distance between axle and cross-beam was $2.5d_l$, which is the critical position for shear failure (Lantsoght et al. 2013b, Rijkswaterstaat 2013b). The distance between the saw cut line and the first wheel was 800 mm in span 1 and 600 mm in span 2. The tandem was placed in the obtuse angle, since this location is critical for shear (Cope 1985).

For a safe execution of the experiment, a steel load spreader, see Fig. 6, was applied over the span. Before the experiment, ballast blocks were placed on the load spreader, so that the slab was not yet loaded. During the experiment, the load was gradually transferred from the load spreader to the wheel prints in the deck by hydraulic jacks. If large deformations caused by failure would occur, the load on the jacks would decrease again thanks to this structure.

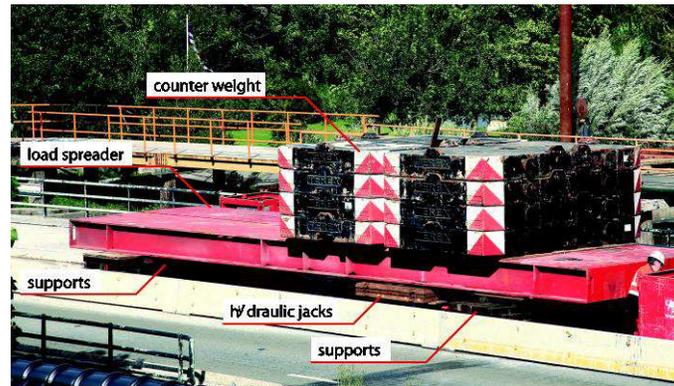


Fig. 6. Overview of test setup on top of Ruytenschildt bridge

5.2 Results

Several load cycles were applied, but only the last cycles of loading until the ultimate capacity are discussed here. The loading scheme for the final step for span 1 is given in Fig. 7a, and for span 2 in Fig. 7b.

The maximum load during the test on span 1 was 3049 kN, but failure was not achieved as the maximum load was determined by the maximum available counter weight. Flexural cracking was observed.

For testing span 2, additional load was ordered. The maximum load was 3991 kN. Flexural failure was achieved. A settlement at the pier of 15 mm right after achieving the maximum load was observed. After removing all equipment, delayed recovery resulted in a residual settlement of 8 mm.

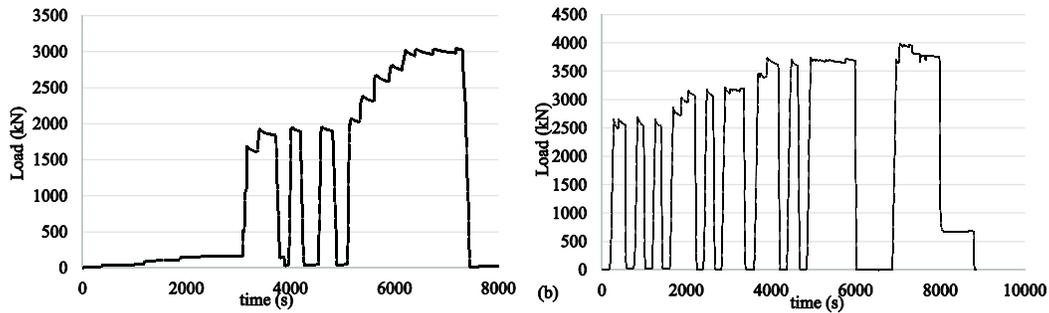


Fig. 7. Loading scheme to the ultimate: (a) Span 1; (b) Span 2.

6. Comparison between tested and predicted results

For span 1, the maximum experimental load was smaller than the predicted maximum P_{tot} . If the slab effect is considered, then the maximum calculated load $P_{tot,slab}$ is significantly higher than the experimental load. For span 2, similar conclusions can be drawn. Since the slab failed in flexure, there is no indication of its ultimate shear capacity. The predictions show a possibility of shear failure in the second span of the bridge. The experiment showed that both spans of the bridge failed in flexure. The observed failure mode is important, as some of the results indicate that the solid slab bridges, currently under discussion with regard to their shear capacity, fail in flexure in reality. Flexural failure is a much less brittle failure than shear failure. These results show that an assessment for both shear and flexure is important. For Level of Assessment I, the Quick Scan method is being extended to include assessment for flexure.

7. Discussion

The uncertainties with regard to the effect of the skew on the effective width result in difficulties predicting the shear capacity of the bridge. A range of values was given, from which it could be seen that the lower bound leads to conservative estimates. For rating of reinforced concrete slab bridges, the procedure that is used in The Netherlands for Level of Assessment I, the Quick Scan, was found to be conservative. If further information about a certain bridge is necessary, a higher Level of Assessment should be used. Level of Assessment IV includes proof loading of bridges. To study the failure modes and their transition of slab bridges subjected to concentrated live loads, future research on the shear capacity of beams with low amounts of reinforcement is prepared. Moreover, experiments on skewed slabs could give more information on the effective width for skewed slabs.

8. Summary and conclusions

Existing bridges in The Netherlands are currently analyzed for different Levels of Assessment. This assessment takes into account previous knowledge from slab shear tests.

To have more information about the behavior of bridges that have been in service for several decades, proof loading and load testing until failure is interesting. Very few examples of bridges that were tested to failure were found in the literature. Therefore, the Ruytenschildt Bridge in The Netherlands is tested to failure in two spans. This bridge is a reinforced concrete solid slab bridge located in the Province of Friesland, the Netherlands.

The material properties were determined before and after testing of the viaduct, and showed that attention needs to be paid to the surface treatment of cores (note that not all cores were drilled vertically), and to the effect of previous yielding on the ultimate strength of the steel reinforcement.

The shear assessment of the Ruytenschildt Bridge was carried out at Level of Assessment I with the Quick Scan method of the Dutch Ministry of Infrastructure and the Environment. It was found that the bridge rates insufficient for shear with the a priori determined material parameters.

For the determination of the expected average shear capacity of the bridge, the uncertainties related to the skewness of the bridge resulted in a range of predictions for the capacity. The uncertainties are related to the increase in shear capacity of slabs with respect to beams, which could be smaller in skewed slabs

because of the effect of stress concentrations in the corners. Moreover, the determination of the effective width in shear for skewed slabs is not clear.

The proof loading and testing to failure were carried out by using hydraulic jacks and a load spreader beam. It was found that the second span had flexural failure as their failure mode. Failure was not achieved in the first span. The flexural failure is positive, since this failure mode is less brittle than shear failure. The failure load in the second span was higher than predicted based on the Quick Scan method, and lower than predicted based on the extrapolation of laboratory test results on virgin specimens without material degradation.

9. Acknowledgements

The authors wish to express their gratitude and sincere appreciation to the Province of Friesland and the Dutch Ministry of Infrastructure and the Environment (Rijkswaterstaat) for financing this research work. The contributions and help of our colleagues A. Bosman, S. Fennis, D. Hordijk P. van Hemert and Y. Yang, of the contractor de Boer en de Groot and of Mammoet, responsible for applying the load, are also gratefully acknowledged.

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