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Ensink, S. W.H.; Lantsoght, E. O.L.; Hendriks, M. A.N.

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S.W.H. Ensink
Concrete Structures, Delft University of Technology, Delft, The Netherlands

E.O.L. Lantsoght
Concrete Structures, Delft University of Technology, Delft, The Netherlands
Politécnico, Universidad San Francisco de Quito, Quito, Ecuador

M.A.N. Hendriks
Concrete Structures, Delft University of Technology, Delft, The Netherlands
NTNU, Trondheim, Norway

ABSTRACT: In the Netherlands, there are about 70 prestressed T-girder bridges with cast-in-between decks, constructed post-World War II. While they have low ratings upon assessment, inspections reveal no distress. This research aims to understand the structural behavior of the full system compared to isolated T-girders. Experiments were conducted on the original structure, as well as on the structure with the deck sawn to test individual girder behavior. The main insight is that the system behavior at the ultimate limit state is driven by compressive membrane action in the deck as well as compressive arching action in the girders, and that simplified distribution factors for shear derived from linear elastic behavior do not properly reflect the distribution and load path at the ultimate. These results can be used for a better assessment of the existing pre- stressed T-girder bridges.

1 INTRODUCTION

In the decades following the Second World War, the Dutch road network underwent a large ex- pansion. This expansion also involved the construction of a large number of bridges. These bridges are now reaching the end of their originally devised service life (Lantsoght et al. 2013). Upon assessment, engineers are confronted with the fact that the live loads that are currently prescribed by NEN-EN 1991-2:2003 (CEN 2003) are heavier than prescribed by the previous national codes, and that at the same time the shear capacity calculated from the current code pro- visions in NEN-EN 1992-1-1:2005 (CEN 2005) is lower than in the previous national codes (Dutch Institute for Normalization 1962). As a result, a large number of existing bridges are ana- lytically found to be insufficient for shear.

One subset of the Dutch bridge stock consists of prestressed T-girder bridges with cast-in-between decks. About 70 of these structures are present in the Netherlands. Even though the analytical assessment for shear results in the conclusion that these bridges are not sufficient, typically no signs of distress are found upon inspection. A possible explanation for this observation is that additional load-carrying mechanisms are activated, which are traditionally not considered in assessment. In particular, for prestressed T-girder bridges with cast-in-between decks, the structural capacity could benefit from the effect of compressive membrane action in the deck, compressive arching action in the girder, the effect of the diaphragms acting as intermediate supports, and of transverse distribution from the analysed girder to the adjacent girders.

Experimental data that quantify the transverse distribution for shear in prestressed T-girder bridges with cast-in-between decks is limited. Moreover, the available experimental data were obtained through diagnostic load tests, or the distribution factors were derived based on

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linear finite element analyses. For girder bridges, the test data are limited to slab-girder bridges, for which the deck is cast on top of the girders instead of in between the girders (Arockiasamy and Amer 1998, Jáuregui and Barr 2004, Wekezer et al. 2004, Mordak and Manko 2008, Idriss and Liang 2010). Additional difficulties for the assessment of the existing prestressed T-girder bridge with cast-in-between decks in the Netherlands are the very low shear reinforcement ratios, and the presence of diaphragms (Cai et al. 2002).

For design, the distribution factors from AASHTO §4.6.2.2 (AASHTO 2015) can be used.

Different distribution factors are determined for bending moment and shear, and for interior and exterior girders. The distribution factor (DF) for an interior girder for shear is as follows:

$$DF = 0.36 + \frac{S}{25.0} \tag{1}$$

with S the spacing of the beams in [ft]. Guidelines are available for flexure on how to use field tests to determine the distribution factors (ACI Committee 342 2016). However, it is not known if the distribution changes as the load is increased and the structural behaviour changes from linear elastic to the ultimate limit state. With this in mind, this paper will use the described test results on a prestressed concrete slab-between-girder bridge to evaluate if the distribution factors can be used to describe the system behaviour, and, if not, which mechanisms contribute to the system behaviour.

2 DESCRIPTION OF VECHT BRIDGE

2.1 *Location, structural system, and geometry*

One example of a T-girder prestressed concrete bridge with cast-in-between decks is the Vecht Bridge near the town of Muiden in the province of Noord Holland in the Netherlands. This bridge was built in 1962, and consisted of nine simply supported spans of which one span contains a movable bridge over the Vecht river. Two parallel bridges were built, one per driving direction. The bridge was situated in the A1 highway. This highway is rerouted, so that it became possible to test the bridge to failure in two spans prior to its demolition. A partial overview of the bridge structure is given in Figure 1 showing the western approach ramp. Figure 1 also marks the inves- tigated spans. On span 3 (see Figure 1) an extensive investigation into the material properties of the concrete (drilled cylinders) and the prestressing tendons has taken place.

All spans are 24 m long, and consist of 15 identical prestressed girders with diaphragms at 8 m intervals. An overview of the transverse cross-section is given in Figure 2. The center-to-center distance between the girders is 1225 mm. The thickness of the cast-in-between decks is equal to the height of the top flange of 180 mm. The girders were prefabricated on-site and pretensioned after which the ducts were grouted.

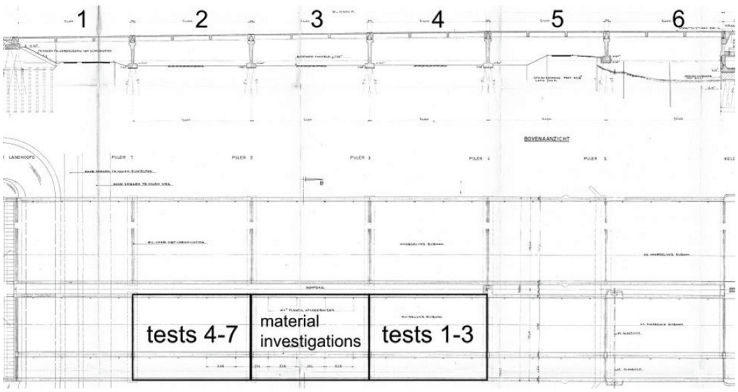

Figure 1. Partial overview of Vecht Bridge, longitudinal section (top) and top view (bottom).

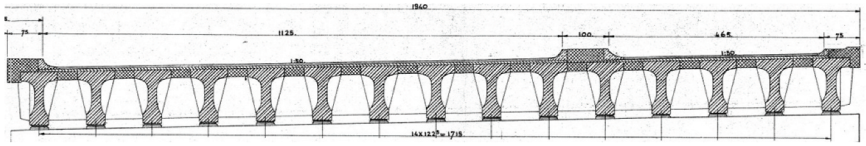

Figure 2. Cross-section of Vecht Bridge (measurements in cm).

2.2 *Material properties and reinforcement*

The longitudinal prestressing in the girders consists of seven 12ø7 mm (A_p = 462 mm^2) tendons per girder. The transverse prestressing consists of 12ø7 mm (A_p = 462 mm^2) per tendon, with a total of 59 transverse prestressing tendons per span. A detail of the curved prestressing tendons close to the support is shown in Figure 3. Prestressing steel qualities QP150 and QP170 were used during the era of construction of the Vecht Bridge. Documents from the archives suggest the use of QP170, so that f_{pk} is estimated as 1754 MPa. Extensive material research in span 3 identified the yield strength of the prestressing steel as 1505.4 MPa and the ultimate strength as 1769.5 MPa. Mild steel reinforcement is present in the deck and girders. In the bottom flange of the girders, 3ø10 mm bars are present at 1107 mm from the face of the top flange and 2ø10 mm at 980 mm. In the web, 2ø10 mm are present at 785 mm, 560 mm and 300 mm, see also Figure 4. The shear reinforcement consists of ø8 mm at 500 mm on center. The mild steel is QR24 with a character- istic yield strength f_{yk} estimated at 280 MPa and confirmed with the extensive material investigation as 287.7 MPa. With the exception of the transverse prestressing, there is no steel reinforce-ment connecting the cast in-between slab to the girder.

The average concrete cube compressive strength $f_{ck,cube}$ on the original drawings is 50 MPa. From this value the average cylinder strength f_{cm} is estimated as 64 MPa. From the extensive material research, the average cylinder strength was found as 87.7 MPa (based on 30 cores) in the T-beams and 60.3 MPa in the integrated deck slab.

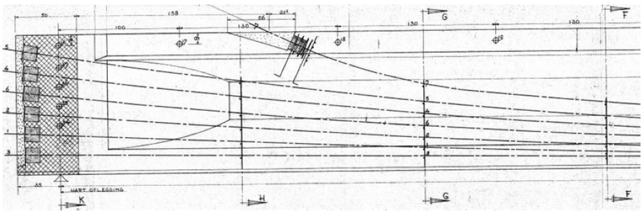

Figure 3. Detail of curved prestressing tendons in T-girder close to the support.

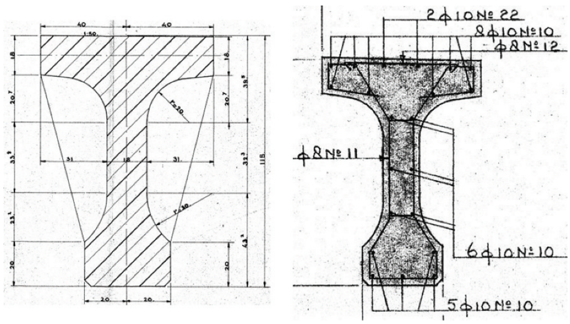

Figure 4. Cross-section of girder (left) and steel reinforcement (right) (measurements in cm).

3 TEST SETUP

3.1 *Load positions*

Three experiments were carried out on span 4, and four experiments in span 2, see also Figure 1 and Table 1. In span 4, the original structural system is unchanged. In test 1 and 2 at least four girders are present between the tested girder and the edge of the bridge. Therefore, in these tests a significant load distribution to the adjacent girders will take place. In test 3 the edge girder itself is loaded. Finally, in span 2, the deck is sawn, so that the individual behaviour of the girders could be tested. Also three of the four cross-beams, with the exception of the one on the opposite side, are sawn. The load position at 4000 mm is chosen because of the position of the cross-beams. The load position at 2250 mm is chosen as the weakest cross-section for an individual girder because of the position of the prestressing tendon anchored in the top flange (see Figure 3). By comparing the structural system to the individual girder behaviour, the system behaviour can be observed.

Table 1. Overview of tests.

Test	Span	Distance from support [mm]	Structural system	Type of test
1	4	4000	not changed	Intermediate girder
2	4	2250	not changed	Intermediate girder
3	4	2250	not changed	Edge girder
4	2	2250	sawn	Individual girder
5	2	2250	sawn	Individual girder
6	2	2250	sawn	Individual girder
7	2	4000	sawn	Individual girder

3.2 *Loading system*

The load is applied through a single concentrated load of 400 mm × 400 mm. The full loading system consist of a 25 m steel spreader beam with ballast on top. The load is then gradually applied from the steel spreader beam to the bridge by using a hydraulic jack. Additionally, a sliding system was provided to easily change the loading position, given the large number of tests planned on site. This system is shown in Figure 5. For the isolated beams in span 2, holes were drilled through the deck through which steel chains were placed and anchored in place by a steel beam, so that the beam would fall into the chains at failure instead of onto the ground.

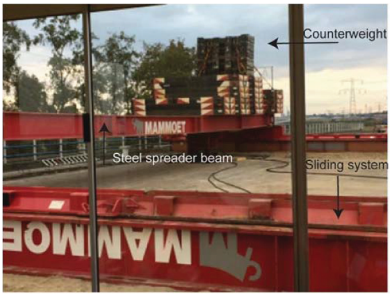

Figure 5. Load application system.

4 TEST RESULTS

4.1 *Failure loads*

Table 2 gives an overview of the results of the seven experiments. The table includes the test number and span of the experiment, with experiments in span 4 on the connected system and in span 2 of the disconnected girders. The *a*/*d* distance is the ratio of the shear span *a* to the effective depth *d* of the girder. The maximum load in the experiment is indicated as F_{max} and the maximum deflection in the experiment is indicated as δ_u. The shear at failure is calculated by considering the sectional shear due to the dead weight, the prestressing, and the externally applied load.

The table also gives the Failure mode. For tests 1 and 2, Figure 6 illustrates the failure in the girder and Figure 7 illustrates the failure in the deck, which is the initial failure. Test 3 was an experiment on an edge girder. As there were safety concerns regarding the stability of the tower of counterweight upon failure of the edge girder, this experiment was limited to a maximum load of 250 tons. All experiments on the disconnected T-beams resulted in a flexural shear failure, see Figure 8. The load-deflection diagram of the connected girders is shown in Figure 9 and of the disconnected girders in Figure 10.

Table 2. Results of experiments.

Test	Span	a/d	F_{max} (kN)	V_R (kN)	δ_u (mm)	Failure mode
1	4	2.8	3004	1188	21	Punching of deck and shear failure of girder
2	4	4.8	3444	1525	14	Punching of deck and shear failure of girder
3	4	2.8	2506	998	11	Not tested to failure
4	2	2.8	1678	1301	79	Flexural shear T-girder
5	2	2.8	1703	1323	65	Flexural shear T-girder
6	2	2.8	1774	1385	74	Flexural shear T-girder
7	2	4.8	1022	774	132	Flexural shear T-girder

Figure 6. Shear failure of T-girder in test 1. The diaphragm is located on the left side and the end support on the right side.


Figure 7. Punching failure of the deck slab in test 1, showing top view and measurements in cm (left) and bottom view (right).

4.2 *Comparison of connected and disconnected girders*

It is not straightforward to compare the results of the shear at failure of the disconnected and connected girders, see Figure 11. A first reason why comparison is not simple is related to the load distribution. In the connected girders, load distribution results in a change of failure

Figure 8. Flexural shear failures in test 4 and 5.

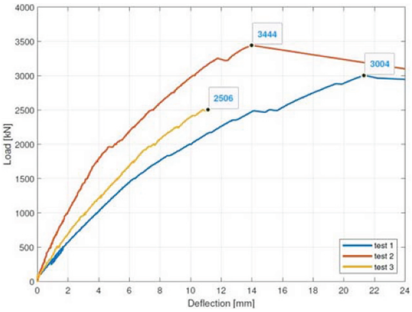

Figure 9. Load-deflection diagrams of girders in connected system.

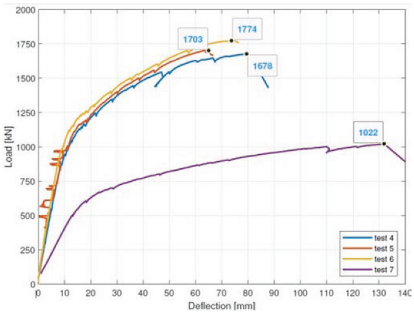

Figure 10. Load-deflection diagrams of girders in disconnected system.

mode from flexural shear for the individual girders to shear-tension for the case of the connected girders. Moreover, an important difference between the connected and disconnected girders can be at- tributed to system behavior represented by arching action in the girder and the deck. For the con- nected girders, the constraints provided by the surrounding members result in significant addi- tional compressive normal forces in the girder and in the development of compressive membrane action in the deck slab (Taylor et al. 2007, Collings and Sagaseta 2015, Amir et al. 2016). Finally, the presence of the diaphragms increases the capacity, as the diaphragm works almost as an in- ternal support.

In addition, using the shear distribution factor from Eq. (1), a factor of 0.52 is found for the interior girders of the Vecht bridge. For an a/d distance of 2.8, the maximum load in the connected system is 3004 kN, which would correspond to a maximum load on the tested girder of 1562 kN according to this distribution factor. This value can be compared to the average failure load of the three experiments of disconnected girders with a/d = 2.8 of 1718 kN. For a/d = 4.8 the maximum load in the connected system is 3444 kN, which would give a maximum load on the tested girder of 1791 kN. The failure load in test 7 was 1022 kN. This brief comparison shows that the AASHTO distribution factors cannot capture the complex system behavior and different shear failure mechanisms that occur in the system at the ultimate limit state.

5 DISCUSSION

This article describes the seven experiments to failure carried out on the Vecht Bridge. As can be seen from the comparison of the maximum load and the sectional shear at failure for the connected bridge system and the disconnected girders, a one-on-one comparison between the two cases is not straightforward. The reasons for the complexity of the comparison is the system behaviour of the connected system, in which compressive membrane action in the deck occurs and compressive arch action in the prestressed girder. Moreover, these two arching mechanisms need to be analysed

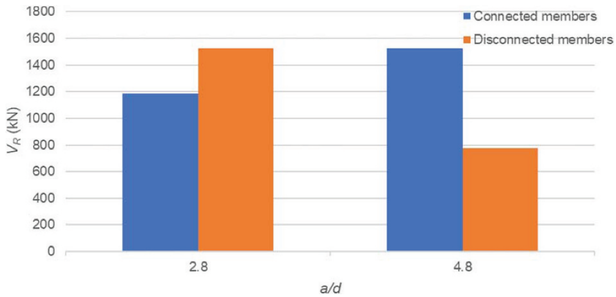

Figure 11. Comparison between sectional shear force at failure for connected and disconnected members from Table 2. Note that the average result of tests 4-6 is used in this comparison.

together. As a result, the flow of forces and the failure mode is significantly different in the connected system as compared to in the individual girders. A full analysis of system behaviour applied to this case study, as well as linear elastic finite element calculations typically used in assessment and non-linear finite element calculations used to further study the experiment can be found in (Ensink 2024).

One important aspect that can already be highlighted here with regard to the system behaviour is that in these experiments, compressive membrane action occurred, even though the load was placed right over the girder. These experiments are the first experiments that demonstrate that compressive membrane action can also be activated when the load is not in the middle of the deck, but right over the girder.

The comparison of the maximum loads also shows that the AASHTO shear distribution factors do not give good results at the ultimate limit state, as these factors are based on linear elastic analyses. The relation between the transverse distribution and the load level requires further re- search. It needs to be verified if the simplified load distribution factors gave good results for the lower load levels in the experiments on the Vecht Bridge, and it can be recommended to carry out research into transverse distribution factors that are representative of the ultimate limit state.

While collapse testing of bridges is expensive (order of magnitude of 400k€), the cost savings for the service life extension of the other slab-between-girder bridges (where replacement would cost over 10m€) is significant (approximately 45% cost savings on a 100-year time horizon when comparing annual equivalent costs). Moreover, indirect cost savings from avoid traffic delays and ecological cost savings related to material use and transportation are obtained.

A final step of this research, which is foreseen for future work, is the extension of this study to encompass all geometries encountered in the slab-between-girder bridges in the Netherlands. For these geometries, the system behaviour needs to be studied in more detail. Once this information is available, a method can be developed to improve the assessment of the existing prestressed T- girder bridges in the Netherlands.

6 SUMMARY AND CONCLUSIONS

This paper introduces seven experiments that were carried out on the Vecht Bridge, a slab-between-girder bridge in the Netherlands with diaphragms at the end and at one third of the span, and with post-tensioned girders. Three experiments were carried out on the connected system and four experiments were carried out on the bridge in which the deck and diaphragms were cut so as to study the behavior of the disconnected girder. The goal of the experiments was to compare the behavior of the connected system and individual members. Based on the described experiments, we can draw the following conclusions:

- The overall system behaves in a much stiffer way than the individual girders.
- The maximum load in the case of the overall system is significantly larger, indicating system behavior in these bridges.

– The failure mode changes from flexure-shear in the individual girders to punching of the deck with a shear-tension failure of the girder for the connected system.
– The system behavior can be attributed to compressive arch action in the girders and compressive membrane action in the deck, and cannot be represented properly by the AASHTO shear distribution factors.

Overall, these failure tests on a real bridge, with all its original details, has provided unique in- sights into the behavior and failure mode of existing slab-between-girder bridges.

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