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## **Long-term material and structural behavior of high strength concrete cantilever bridge: results of twenty years monitoring**

Eva O.L. Lantsoght<sup>1,2</sup>, Cor van der Veen<sup>3</sup>, Ane de Boer<sup>4</sup>, Herbert van der Ham<sup>5</sup>

1

### 2 **Summary**

3 In 1997, the Second Stichtse Bridge was built in the Netherlands using the balanced cantilever  
4 method. The use of high strength concrete was proposed. At that time, the long-term behaviour  
5 of this material was not known, and no code provisions were applicable. Therefore, it was  
6 proposed to monitor the material behaviour and the deflections of the bridge. To evaluate the  
7 development of the concrete compressive strength and the concrete splitting tensile strength over  
8 time, concrete cubes were cast at the same time as each of the cantilever segments, and stored  
9 inside the bridge. These samples have been tested at different points in time to study the  
10 development of the strength as a function of the elapsed time. Creep and shrinkage  
11 measurements were carried out on samples stored inside the bridge as well as in the laboratory.  
12 Temperature and moisture were monitored as well. The deflections of the bridge superstructure  
13 have been measured periodically. These measurements can be compared to predictions from  
14 finite element models. Based on the available data, it is found that the concrete compressive and

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1 splitting tensile strength remain constant. The deflections are small, and the advanced finite  
2 element models resulted in good predictions.

3

4 **Keywords:** balanced cantilever construction method; box girder bridge; deflections; high  
5 strength concrete; long-term behavior; material properties; monitoring

## 6 **1 Introduction**

7         The cantilever construction method has been used since the nineteenth century, with the  
8 Forth bridge as a notable example. In the twentieth century, the introduction of the new  
9 construction material prestressed concrete resulted in the use of the in-situ balanced cantilever  
10 method to build prestressed concrete bridges with a variable height and with a box girder cross-  
11 section. Until 1984, such bridges were built in the Netherlands by using lightweight concrete.  
12 The advantage of using lightweight concrete is the reduction in self-weight, which leads to  
13 smaller dimensions. However, in practice problems were observed. During construction, the high  
14 pressure used to prepare the ducts resulted at some locations in the propagation of cracks in the  
15 concrete, which led to spalling of the concrete. In some cases, the damage occurring during  
16 construction was considerable. The remaining option then was to use normal weight gravel  
17 aggregate concrete, which led to larger amounts of prestressing (1).

18         For the construction of the Second Stichtse Bridge, the option of using high strength  
19 concrete was explored in the 1990s. This bridge would parallel the First Stichtse Bridge, a  
20 lightweight concrete box girder bridge constructed using the balanced cantilever method, which  
21 opened in 1983. Therefore, a requirement for the Second Stichtse Bridge was that it would look  
22 similar to the First Stichtse Bridge. As the use of lightweight concrete was not allowed anymore,

1 and normal weight concrete would result in larger cross-sections, high strength concrete was  
2 selected for the construction of the Second Stichtse Bridge. The difficulty at that time, however,  
3 was that the governing codes did not include high strength concrete classes, and that no  
4 information was available about the long-term performance of these new concrete mixes. For this  
5 reason, it was decided to monitor the material properties and overall structural behavior of the  
6 Second Stichtse Bridge. The originally planned duration for the monitoring was ten years, but  
7 some parameters have been monitored for twenty years.

## 8 **2 Literature review**

9 Research on the structural applications of high strength concrete with cube compressive  
10 strengths of 100 MPa has been carried out, first on reinforced concrete beams, since the 1970s  
11 (2). The application to prestressed, pretensioned high strength concrete bridge girders was  
12 studied in the 1990s (3). The results of this study was that high strength concrete bridge girders  
13 can be expected to perform adequately over the long-term, but that further research into the creep  
14 and shrinkage characteristics of high strength concrete is required. A decade later, results of high  
15 performance lightweight concrete pretensioned bridge girders were reported (4).

16 The long-term material behavior of plain concrete samples of different strengths of  
17 concrete was studied in the 1980s (5), and led to the conclusion that the long-term sustained  
18 strength of high strength concrete is higher than that of low and medium strength concrete, but  
19 that it is not much greater. Additionally, it was found that while the influence of the concrete  
20 strength on total strain at failure depends upon the stress level, the creep strain, creep coefficient,  
21 and specific creep at failure are greater as the strength of concrete is lower. A similar  
22 observation was made in another study in which the sustained stresses were applied for 3 months

1 (6), as well as in a laboratory study which lasted two years (7). It must also be noted that the  
2 longest experiment from this series lasted 60 days, which is much shorter than the information  
3 required for the application to bridges. Other research (8) reported on the results of high strength  
4 concrete cylinders under creep load for up to 430 days. Again, longer periods of time are  
5 required for the application to bridges. The influence of the type of aggregate has also been  
6 studied in laboratory tests (9).

7         In terms of the long-term behavior of high strength prestressed concrete box girder  
8 bridges, limited information is available, none of which was available at the time of design of the  
9 Second Stichtse Bridge. In China, the long-term behavior of a segmental concrete cantilever T-  
10 beam of 12.8 m in length has been studied (10) to simulate the construction process of the  
11 Sutong Bridge. The results of the scaled specimen in the field gave information about the losses  
12 that could be expected for the actual bridge, but it must be noted that the measurements were  
13 only carried out over 1 year. The reason for carefully estimating the prestress losses for the  
14 Sutong Bridge is that the collapse of the Koror-Babeldaob bridge in Palau has been attributed to  
15 excessive creep and prestress losses (11), and that the maximum deflection at midspan after  
16 seven years in service of the Huangshi Yangtze River Bridge in China was 32 cm higher than  
17 predicted. Similar problems have been observed in Switzerland, where long-term monitoring of  
18 prestressed concrete balanced cantilever bridges using hydrostatic leveling showed that these  
19 structures had undergone large deformations and required strengthening (12). It must be noted,  
20 however, that these bridges have a hinge at the connection of the cantilevering parts, which is not  
21 the case for the Second Stichtse Bridge.

22         Field testing in India of an existing cast-in-place box girder bridge span of normal  
23 strength concrete (13) showed that the measured net camber was closest predicted using the

1 Model Code 1990 (14) for creep and shrinkage after 5 years of measurements. In Canada, the  
2 Confederation Bridge, a prestressed concrete box girder balanced cantilever bridge completed in  
3 1997, and using 55 MPa concrete, is also being monitored (15). The deflections are found to be  
4 smaller than predicted (16).

5 In the Netherlands, the long-term deflections of prestressed concrete bridges constructed  
6 with the balanced cantilever method have been measured annually between the late 1960s and  
7 the early 1990s (17). It was found that no limit value of the deflections was reached after 25  
8 years, which is in contradiction with the expectation of reaching a limit after 10000 days. The  
9 available results from this time period are shown in Fig. 1 for normal weight normal strength  
10 concrete (NWNSC) cantilever bridges. The first bridge, the Wessem bridge, had a larger cross-  
11 sectional height than bridges that were designed and built later, such as the Maastricht bridge,  
12 which had a more economical design. The Grubbenvorst, Empel, and Heteren bridges are very  
13 similar structures. Regardless, the Grubbenvorst bridge had 30% larger deflections than the  
14 Empel and Heteren bridges. The Ravenstein bridge had the largest span. For these structures, the  
15 calculated deflections based on the Model Code 1978 (18) were about 20% smaller than the  
16 deflections measured in the field.

17 From this literature review, three main conclusions can be drawn. The first conclusion is  
18 that laboratory testing on girders showed that adequate performance of high strength girders is  
19 expected based on the experimental result, but that no long-term creep and shrinkage data are  
20 available. The second conclusion is that material testing indicated that for high strength concrete  
21 the effect of creep may be lower, but this conclusion is based on maximum 430 days of  
22 following of the properties. The third conclusion is that at the time of the design and construction  
23 of the Second Stichtse Bridge, no information was available about the long-term material and

1 structural behavior of high strength concrete box girder bridges. Even nowadays, the available  
2 data gathered over a long period of time are scarce. From the available monitoring results, it is  
3 concluded that bridges with a hinge at midspan (the cantilever connection) seem to have large  
4 deformations whereas for cases without a hinge the deflections are as predicted or smaller.  
5 Moreover, following the deflections of bridges constructed in the Netherlands with the balanced  
6 cantilever method showed that no limit value of the deflection was reached after 25 years. With  
7 this information in mind, the value of monitoring the material and structural behavior of a high  
8 strength concrete bridge built with the balanced cantilever method over a long period of time  
9 (twenty years) becomes clear. The data of this long program can be used to answer the questions  
10 that remain from the discussed literature review.

11

### 12 **3 Description of Second Stichtse Bridge**

13 Construction of the Second Stichtse Bridge was finished in 1997. A photograph of the  
14 structure can be seen in Fig. 2. At its time of completion, the Second Stichtse Bridge was the first  
15 prestressed concrete bridge constructed with the balanced cantilever method using high strength  
16 concrete in the Netherlands. The First and Second Stichtse Bridge are part of the Dutch highway  
17 A27. The bridges connect the provinces Flevoland and Noord Holland over a canal that connects  
18 the Gooi lake and Eem lake. When the Second Stichtse Bridge was opened, the traffic capacity  
19 of this particular crossing was doubled.

20 The total length of the Second Stichtse Bridge is 320 m, over three spans: two end spans  
21 of 80 m and a main span of 160 m, see Fig. 3. In Fig. 3, the cross-sections are shown that are  
22 necessary when lightweight concrete ( $f_{c,cu} = 40$  MPa, with  $f_{c,cu}$  the design concrete cube

1 compressive strength), NWNNSC ( $f_{c,cu} = 55$  MPa), and high strength concrete ( $f_{c,cu} = 75$  MPa) are  
2 used, respectively (1). The option with high strength concrete required 28% less concrete than  
3 lightweight concrete and 14% less concrete than NWNNSC, and 11% less prestressing steel than  
4 lightweight concrete and 26% less prestressing steel than NWNNSC. Since the dimensions and the  
5 weight of the high strength concrete option are smaller than for the normal strength option, larger  
6 segments could be developed and the erection time could be shortened by three months. The  
7 required amount of cement for high strength concrete was slightly higher (3%) than for light  
8 weight and NWNNSC.

9         As mentioned previously, and as can be seen from the literature, at the time of design of  
10 the Second Stichtse Bridge in the 1990s, the information about the long-term material and  
11 structural behavior of high strength concrete was limited. Therefore, it was decided to monitor  
12 properties of this structure over time. In particular, the development of the concrete compressive  
13 and splitting tensile strength, the effect of creep and shrinkage (including autogenous shrinkage),  
14 and the resulting deflections over time needed further study. Estimating the elastic and time-  
15 dependent deflections during the construction stages is important to determine the required  
16 camber of the main span. To estimate the time-dependent deflections of the finished structure,  
17 the creep and shrinkage behavior of high strength concrete over several years needs to be known.  
18 To gather data about these unknown parameters, experiments have been carried out on samples  
19 that were cast during construction of the bridge and stored inside the box girder, as well as on  
20 samples that were cast during construction and then transported, stored, and tested in the Stevin  
21 II laboratory of Delft University of Technology. To assess the structural behavior of the bridge,  
22 the deflections of the bridge are measured periodically on site by a surveyor. The error on the  
23 surveying results was reported in 2016 as 0.3 mm.

## 1 **4 Long-term material behavior**

### 2 **4.1 Concrete compressive strength**

3           The concrete compressive strength has been measured for twenty years on cubes that  
4 were cast during the construction of the bridge and that were stored inside the respective bridge  
5 segments. The numbering of the segments, following the construction sequence, is shown in Fig.  
6 4. The specimens that are discussed in this paper were cast together with the main span  
7 segments, and stored inside of the main span of the bridge. In the reports of the concrete supplier,  
8 the average measured cube concrete compressive strength at 7 and 28 days is reported as 86.9  
9 MPa and 106.9 MPa respectively. It must be noted as well that cubes with a compressive  
10 strength higher than 115 MPa were not tested to failure. To determine the concrete compressive  
11 strength, a number of cubes were gathered from the bridge, transported to the laboratory, and  
12 tested there. Compressive strength testing was carried out on concrete with an age of 84 days,  
13 124 days, 1 year, 2 years, 5 years, 10 years, and 20 years. The individual results of all tested  
14 cubes can be found in the full measurement report (19). The results after following the  
15 development of the concrete compressive strength after 10 years showed a reduction in the  
16 strength as compared to the strength measured after 1 year of 5% for the cubes from segments 8  
17 and 9. At that time, it was concluded that a continued increase in the concrete compressive  
18 strength over time cannot be expected for high strength concrete, and it was recommended to  
19 repeat a set of measurements after 20 years.

20           The concrete compressive strength was determined on cubes from segments 5, 6, 7 and  
21 12 after 20 years. The strength development is shown in Fig. 5. Each datapoint in Fig. 5  
22 represents the average of the cubes associated with a single segment that were tested (usually  
23 three cubes). Unlike the results after 10 years, the results after 20 years show no decrease of the

1 concrete compressive strength, and seem to indicate even a possible increase over time. To take  
2 into account the effect of the variability of the material, the lines of one standard deviation above  
3 and below the general average (all cubes tested at a certain point in time) are shown as well in  
4 Fig. 5. The standard deviation on the tested cubes is 5.67 MPa, which is slightly larger than the  
5 value of 4.88 MPa prescribed for cylinders in NEN-EN 1992-1-1:2005 (20). The results in Fig. 5  
6 show that the average value plus one standard deviation after one year (138 MPa) is similar to  
7 the average value after twenty years (138 MPa). As the measured strength lies within one  
8 standard deviation from the average, it can be considered to be constant after one year. These  
9 results can also be used to reevaluate the lower strengths measured after 10 years. The  
10 measurements from cubes from segments 8 and 11 lie within one standard deviation of the  
11 average measured at one year. Only the results from segment 9 are lower. Given the performance  
12 after 20 years, the results from segment 9 can be considered as an outlier.

13         Similar results, indicating that the concrete compressive strength does not significantly  
14 increase after about 90 days, were found for cubes of concrete class C55/67 that were cast in the  
15 laboratory along with reinforced concrete slabs (21), see Fig. 6. These cubes were stored in the  
16 fog room, and compressive strength tests were carried out to study the strength development of  
17 this high strength concrete.

18         It is thus safe to conclude that the compressive strength of the studied high strength  
19 concrete mix does not decrease over time. However, for this high strength concrete with finely  
20 ground cement, no doubling or tripling of the concrete compressive strength over time can be  
21 assumed, which is the case for the concrete mixes with coarsely ground cement that were used in  
22 the Netherlands in the decades after the Second World War.

## 1 **4.2 Concrete splitting tensile strength**

2 Just like the concrete compressive strength, the concrete splitting tensile strength has  
3 been measured on cubes that were cast together with the segments, and stored inside the box  
4 girder bridge. After the first ten years of measurements, a splitting tensile strength of 7 MPa on  
5 average was found. After two years, temporarily lower values of 6 MPa were found. As the  
6 design was carried out assuming C55/67 concrete, the average tensile strength should be  $f_{ct,m} =$   
7 4.2 MPa. The value of 6 MPa after 2 years was thus never too low. After ten years of  
8 measurements, the conclusion was that the splitting tensile strength remained constant.

9 The results of the measurements after twenty years are now available, and these are  
10 shown together with all previous measurements in Fig. 7. After twenty years, the splitting tensile  
11 strength was determined on cubes cast with segments 5, 6 and 12. As can be seen in Fig. 7, the  
12 splitting tensile strength has remained constant and about 7 MPa.

13 Similar results from measuring over two years can be seen in Fig. 8. The strength  
14 development of a C55/67 concrete mix used for casting slab specimens was followed over two  
15 years. The cubes were stored in the fog room. It can be seen that the splitting tensile strength  
16 remains constant after 90 days. The general conclusion of measuring the splitting tensile strength  
17 of the high strength concrete mixture for twenty years is that the tensile strength remains  
18 constant over time. This conclusion is supported by the evidence of twenty years of  
19 measurements on the Second Stichtse Bridge, as well as on the cubes followed during two years  
20 in the laboratory.

## 21 **4.3 Shrinkage**

22 To evaluate the shrinkage of the high strength concrete mixture used in the Second  
23 Stichtse Bridge, measurements on specimens in the field and in the laboratory were taken. The

1 laboratory setup was kept for ten years, whereas the specimens in the field are stored inside the  
 2 box girder and remain available for measurements. The specimens tested in the laboratory, see  
 3 Fig. 9, were prisms with dimensions of 400 mm × 100 mm × 100 mm. The shrinkage strain  $\varepsilon_{sh}$  is  
 4 determined for concrete with an age of 14 days and with an age of 365 days. The specimens were  
 5 stored at a constant temperature of 20°C and at a constant relative humidity of 50%. For the  
 6 concrete exposed at 14 days until 10 years, a shrinkage strain of  $\varepsilon_{sh} = 0.35 \times 10^{-3}$  was found, and  
 7 for the concrete exposed at 365 days until 10 years, a shrinkage strain of  $\varepsilon_{sh} = 0.25 \times 10^{-3}$  was  
 8 found. The measurements on the concrete exposed at 14 days stabilized at 4 years, and for the  
 9 concrete exposed at 365 days at 8 years. For the measurements after 14 days, the shrinkage  
 10 increased continuously, whereas for the measurements after 365 days, a small reversal ( $\approx 0.01 \times$   
 11  $10^{-3}$ ) of the shrinkage between 4 and 8 years occurred. The shrinkage measured in the laboratory  
 12 was mostly drying shrinkage. Autogenous shrinkage was assumed to occur mostly in the first  
 13 weeks after casting.

14 The specimens stored inside the box girder, see Fig. 10a, are concrete blocks of 320 mm  
 15 × 600 mm × 1000 mm. These were cast and then stored together with segments 13 and 15. The  
 16 thickness, 320 mm, is the same as the thickness of the web of the box girder. On these concrete  
 17 specimens, deformations are measured at three position (two on the top of the specimen and one  
 18 on the bottom) over a length of 800 mm. A detail of the locations of the measurement points is  
 19 shown in Fig. 10b. The total deformation is caused by changes in temperature and shrinkage.  
 20 Therefore, to find the strain caused by shrinkage, the strain caused by changes in temperature  
 21  $\varepsilon_{temp}$  needs to be corrected for. The value of  $\varepsilon_{temp}$  is calculated as:

$$22 \quad \varepsilon_{temp} = \alpha (T_{meas} - T_{ref}) l_{meas} \quad (1)$$

1 with  $\alpha$  the coefficient of thermal expansion,  $\alpha = 12 \times 10^{-6}/^{\circ}\text{C} \times \text{m}$ ,  $T_{meas}$  the temperature when the  
2 measurement is taken,  $T_{ref}$  the reference temperature of  $9.7^{\circ}\text{C}$  on April 11<sup>th</sup> 1997 at 11 am when  
3 the measurements were started, and  $l_{meas}$  the length over which is measured,  $l_{meas} = 800$  mm. The  
4 resulting measured values for the shrinkage strains are shown in Fig. 11a for the specimen cast  
5 with segment 13 and in Fig. 11b for the specimen cast with segment 15. The results of the  
6 measurements on top and bottom of the same specimen are quite similar. From the results in Fig.  
7 11 it can be seen that during the first 18 months, a clear increase of the shrinkage strains was  
8 observed. Subsequently, a period of swelling was observed until three years after casting the  
9 concrete. For the time period between three and ten years, continuous shrinkage is observed in  
10 all specimens. The maximum observed shrinkage strain for the specimen cast with segment 13 is  
11 less than  $\varepsilon_{sh} = 0.6 \times 10^{-3}$  and for the specimen cast with segment 15 less than  $\varepsilon_{sh} = 0.5 \times 10^{-3}$ .  
12 These values for the shrinkage strains are larger than the results from the specimens monitored in  
13 the laboratory, since the conditions are not the same.

14         During the first ten years of monitoring, the temperature and humidity inside and outside  
15 of the box girder were measured continuously. These measurements are important for the  
16 correction of the total deformation with the effect of temperature. However, these measurements  
17 were not continued after ten years. Therefore, the temperature after twenty years of  
18 measurements was estimated based on the observations of the Royal Dutch Meteorological  
19 Institute (22). The nearest observation stations (Schiphol, Berkhout, De Bilt, Lelystad) reported  
20 temperatures on April 26<sup>th</sup> 2017 at 2 pm, the moment of data collection, ranging between  $8.3^{\circ}\text{C}$   
21 (De Bilt) and  $9.7^{\circ}\text{C}$  (Schiphol). Therefore, an exterior temperature of  $9^{\circ}\text{C}$  can be assumed. This  
22 temperature was used for the results shown in Fig. 11. However, the specimen on which the  
23 deformation measurements are taken is stored inside the box girder, so that the temperature of

1 the specimen may have been larger than the exterior temperature. The average difference  
2 between the temperature inside and outside the box girder was determined based on the  
3 measurements of temperature of the first ten years as 1.5°C (23). The largest temperature  
4 differences were observed during the winter and summer seasons, and are maximum 9°C. During  
5 the spring months, such as the time period when the last set of deformation measurements were  
6 taken, the largest observed differences between the temperature inside the girder and outside are  
7 5°C. Therefore, it is assumed that the temperature inside the box girder can have been maximum  
8 14°C when the last set of deformation measurements were taken. These results are shown in Fig.  
9 12, showing that the maximum shrinkage strain is still less than  $\varepsilon_{sh} = 0.6 \times 10^{-3}$ .

10 When a temperature inside the box girder of 9°C is assumed (Fig. 11), a small amount of  
11 swelling of the specimens can be observed after twenty years in comparison with the  
12 measurements after ten years. When a temperature inside the box girder of 14°C is assumed (Fig.  
13 12), no difference between the shrinkage strain after ten and twenty years is found. In general, it  
14 can be concluded that no further increase in shrinkage is observed after twenty years and that the  
15 time-dependent deformations have stabilized in the observations between ten and twenty years.

#### 16 **4.4 Creep**

17 To evaluate the creep of the high strength concrete mixture used in the Second Stichtse  
18 Bridge, measurements on specimens in the field and in the laboratory were taken during the first  
19 ten years after construction (24). The creep tests in the laboratory are carried out on prisms of  
20 100 mm × 100 mm × 400 mm. The deformations are measured over 200 mm by four LVDTs.  
21 The specimens are not sealed, so that the measurements have to be corrected for the effect of  
22 shrinkage. The setup is placed in a room with a constant temperature of 20°C and a constant  
23 relative humidity of 50%. A photograph of the test setup is shown in Fig. 13.

1           The different specimens are loaded at three different ages of the concrete: 14, 28, and 365  
2 days. For the specimens loaded at 14 and 28 days, a stress of 10 MPa was used. For the specimen  
3 loaded at 365 days, a stress of 20 MPa was used. This load level was then kept constant for 10  
4 years. The laboratory measurements led to the conclusion that a small creep factor (defined as  
5 related to the elastic deformation at 28 days) of between 0.5 and 1.0 is observed for the studied  
6 high strength concrete for all ages of loading of the concrete.

7           After ten years, the load was removed from the creep specimens. The deformations were  
8 measured as well during unloading. Afterwards, the specimens were loaded to a stress level of 35  
9 MPa (about 30% of the actual measured compressive strength). The behavior during loading was  
10 linear elastic and could be used to determine the modulus of elasticity. This value was found to  
11 increase as the age of the concrete at first loading increased. The modulus of elasticity is  
12 influenced by the load level and the age of first loading. After unloading, the delayed elastic  
13 deformations occurred within 60 minutes after removing the load. These observations are  
14 important for the assessment of existing bridges, where the effect of prestressing on the concrete  
15 needs to be determined.

16           In addition to the laboratory tests, measurements were taken on the web of the box girder  
17 for segments 2, 13, and 15. Since a specimen inside the box girder (Fig. 12) was monitored for  
18 shrinkage, the difference in behaviour between the web and the shrinkage specimen can be  
19 attributed to creep. Therefore, the measurements on the web have to be corrected for the effect of  
20 shrinkage (determined based on the specimen stored inside the bridge), and the effect of  
21 temperature to find the effect of creep. The results for segment 13 for ten years of monitoring are  
22 shown in Fig. 14. The large increase in the strains at 120 days of age were caused by the casting  
23 of the asphalt layer. These results can be used for calculations with for example finite element

1 models. In such a model, a discrete creep function can be implemented as a visco-elastic process.  
2 A Kelvin chain, consisting of a series of parallel springs and dashpots, can be used. The creep  
3 strains are of the same order of magnitude as the measured shrinkage strains.

4

## 5 **5 Long-term structural behavior**

### 6 **5.1 Deflection measurements**

7 The deflections of the Second Stichtse Bridge have been measured at several points in  
8 time by a surveyor. These sets of measurements are taken over two profiles at 13 positions. The  
9 measured values at supports 2 and 3 and at midspan of the central span, as well as the reported  
10 temperature at the time of measurement collection are given in Table 1 and Table 2. The full  
11 measured profiles are shown in Fig. 15, where negative values denote sagging and positive  
12 values hogging. It can immediately be observed that the first support is undergoing a settlement.  
13 In 2016, the measured value of the settlement was -22.3 mm. This settlement influences the  
14 midspan deflection of the central span.

15 To find the net midspan deflection of the central span, without the effect of the  
16 settlements at the support, the effect of all support settlements should be taken into account. To  
17 find the deflection at midspan of the central span caused by all support settlements, a beam  
18 model is used of the box girder modeled as a girder with a variable moment of inertia  $I$  resulting  
19 from the changes in the height of the box girder. The calculated deflection can then be used to  
20 correct for the effect of the settlements, and find the net midspan deflection of the central span  
21 over time.

22 For the end spans, the height varies parabolically between 2.5 m and 6.75 m. The  
23 expression of the parabola is as follows:

$$1 \quad h(x) = 6.64 \times 10^{-4} \frac{1}{m} x^2 + 2.5m \quad (2)$$

2 For the central span, the height also varies parabolically between the supports and the midspan:

$$3 \quad g(x) = 6.64 \times 10^{-4} \frac{1}{m} x^2 - 0.10625 \times x + 6.75m \quad (3)$$

4 The dimensions of the cross-section used to calculate  $I$  are shown in Fig. 3. The value of the  
5 modulus of elasticity of the concrete is taken as  $E = 46725$  MPa, as measured after one year (23).

6 In the beam model, elements with a length of 5 m (the largest length of a segment that was used  
7 during construction) are used. Each element has a constant value of  $I$ , which differs from the  
8 adjacent elements. When the support settlements measured in 2016 are implemented in the beam  
9 model, a hogging deflection at midspan of the central span of 11.35 mm is found. Since a  
10 sagging deflection of -52.25 mm was measured, it can be found that the net deflection equals -  
11 63.6 mm. This calculation of the deflection caused by the support settlements is carried out for  
12 all sets of measurements and given in Table 3. The resulting net deflection at midspan of the  
13 central span is then given in

14 Table 4. These results are shown graphically as a function of time in Fig. 16. This figure also  
15 shows that the increase in deflections between the structure with an age of five years and with an  
16 age of twenty years is limited.

17 In 2008 (23) it was mentioned that the deflections measured in 1999, at a temperature of  
18 8°C were lower and below the trend of the measurements taken at higher ambient temperatures.  
19 Similarly, it can be seen that the deflections measured in 2010, at a temperature of 8-12°C were  
20 lower as well. Certainly, the influence of the temperature on the measured deflections is clear.  
21 This observation should be kept in mind when evaluating the measurement results. Since the

1 majority of the measurements were carried out at high temperatures, and the results show that the  
2 deflections are larger at larger temperatures, the measurements are on the safe side.

3         The net deflection at midspan of the central span after twenty years equals -63.6 mm.  
4 This value can be expressed as a fraction of the span length of 160 m. The required fraction is  
5 then 1/2516. It can thus be concluded that the net deflection at midspan is small.

6

## 7 **5.2 Comparison to predictions**

8         During the design stage and after construction of the Second Stichtse Bridge, several  
9 predictions have been made of the long-term deflections. These predictions have been used to  
10 evaluate different creep and shrinkage models as presented in different codes (24).

11         The most recent, and most advanced calculations (25), were carried out in TNO Diana  
12 Release 8 (26). For these calculations, staged execution of the structure was used. The modulus  
13 of elasticity was entered as a function of the time, and the shrinkage was modeled dependent on  
14 the time of removing the formwork. The creep was taken as a function of the time and load,  
15 related to the actual construction and load scheme of the bridge. The creep behavior was  
16 modeled with 10 Kelvin chains and 8 time steps.

17         The creep and shrinkage models of NEN-EN 1992-1-1:2005 (20), Model Code 1990 (14)  
18 and Han (27), a modified model of the Model Code 1990 with the inclusion of parameters for  
19 high strength concrete, have been studied. The results for the predictions for 100 years are given  
20 in Fig. 17a. These are then compared to the measurements in Fig. 17b. From this comparison, it  
21 can be seen that the predictions using the creep and shrinkage methods of Eurocode 2 and Han  
22 give better results than Model Code 1990, which overestimates the long-term deflection. It can  
23 also be concluded that the finite element model can give a good prediction of the long-term

1 behavior of a prestressed concrete bridge constructed using the balanced cantilever method. This  
2 conclusion is important for the design of this type of bridges.

3

#### 4 **6 Discussion**

5 The presented results contain unique data with regard to the long-term material and  
6 structural behavior of high strength concrete box girder bridges. These results can be used to  
7 compare to the strains caused by creep and shrinkage as prescribed by the governing codes,  
8 which are determined based on laboratory test results only. Since the presented results are  
9 determined based on laboratory specimens as well as on an actual structure, they can be used to  
10 improve the assessment practice for existing prestressed high strength concrete bridges. These  
11 field data can also be used to optimize the design of high strength concrete balanced cantilever  
12 bridges. Future analysis of the bridge can include the material models of the *fib* Model Code  
13 2010 (28), and/or using a plane stress model to study shear.

14

#### 15 **7 Summary and conclusions**

16 When the Second Stichtse Bridge was designed in the 1990s, it was the first high strength  
17 prestressed concrete bridge in the Netherlands constructed with the balanced cantilever method.  
18 As can be seen from the literature, the information about the long-term material properties of  
19 high strength concrete were not well known at the time of design. Therefore, the calculation of  
20 the long-term deflections was a challenge. For this purpose, it was decided to monitor certain  
21 material parameters and the deflections over a large period of time.

1           The concrete compressive strength was determined on cubes cast together with segments  
2 of the box girder, and stored inside these segments. Whereas the test results after 10 years  
3 indicated that the concrete compressive strength could be slightly reduced in comparison with  
4 the strength after one year, the results after twenty years show that the concrete compressive  
5 strength remains constant. It must be noted though that the concrete compressive strength does  
6 not double or triple over time, as is the case for concrete mixtures with coarsely ground cement  
7 used in the past in the Netherlands. The concrete splitting tensile strength has been determined in  
8 the same way as the compressive strength, and was found to remain constant over time as well.

9           During the first ten years after construction, the shrinkage and creep deformations have  
10 been monitored on specimens inside the box girder for shrinkage and on the web of the box  
11 girder for creep, and on specimens in the laboratory for both creep and shrinkage. It was found  
12 that the time-dependent deformations have stabilized. The shrinkage strain in the field was larger  
13 than the strain measured in the laboratory, which is a more controlled environment. A small  
14 creep factor was found in the laboratory and the field.

15           The Second Stichtse Bridge is surveyed frequently. These measurements show that the  
16 deflections still slightly increase over time and that a settlement occurs at the first support. The  
17 net displacement at midspan of the central span can be used for comparison with the results of a  
18 finite element model that was used to predict this deflection. Different finite element models,  
19 based on different material models for the long-term deflections were studied. It was found that  
20 the modified model for high strength concrete from Han based on Model Code 1990 and  
21 Eurocode 2 both resulted in the predictions closest to the measured deflections. The developed  
22 model is thus valid, and can be used for the future design of high strength prestressed concrete  
23 bridges constructed with the balanced cantilever method.

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- 14
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1 **Tables**

2 Table 1. Measurements reported in 2016, profile 101-113 (29)

Date	Sup 2 (mm)	Span (mm)	Sup 3 (mm)	<i>T</i> (°C)
04-08-1997	0.0	0.0	0.0	21.7
07-09-1998	-1.0	-17.0	0.0	17.4
15-11-1999	-3.0	-14.0	-2.0	8
17-11-2003	-0.4	-47.3	1.6	23-25
11-09-2006	2.7	-41.4	2.3	25
18-11-2010	-0.4	-38.1	0.0	8-12
06-09-2016	3.6	-52.3	4.3	20

3

4 Table 2. Measurements reported in 2016, profile 401-413 (29)

Date	Sup 2 (mm)	Span (mm)	Sup 3 (mm)	<i>T</i> (°C)
04-08-1997	0.0	0.0	0.0	21.7
07-09-1998	1.0	-19.0	-2.0	17.4
15-11-1999	1.0	-16.0	-3.0	8
17-11-2003	-0.8	-51.8	-1.7	23-25
11-09-2006	0.2	-47.1	-1.0	25
18-11-2010	0.0	-41.4	-3.4	8-12
06-09-2016	4.3	-52.2	1.6	20

5

6 Table 3. Average support settlements and midspan deflection

Date DD-MM-YYYY	Sup 1 (mm)	Sup 2 (mm)	Span (mm)	Sup 3 (mm)	Sup 4 (mm)
04-08-1997	0.0	0.0	0.0	0.0	0.0
07-09-1998	-2.0	0.0	-18.0	-1.0	0.5
15-11-1999	-2.5	-1.0	-15.0	-2.5	0.5
17-11-2003	-11.2	-0.6	-49.6	-0.1	0.7
11-09-2006	-13.1	1.5	-44.3	0.7	1.6
18-11-2010	-16.8	-0.4	-39.8	-1.7	1.6
06-09-2016	-22.3	4.0	-52.3	3.0	2.0

7

8

1 Table 4. Calculated effect of support settlements and resulting net deflection at midspan

Date DD-MM-YYYY	Effect (mm)	Net (mm)
04-08-1997	0.0	0.0
07-09-1998	-0.4	-17.7
15-11-1999	-2.2	-12.8
17-11-2003	2.5	-52.1
11-09-2006	4.4	-48.7
18-11-2010	2.7	-42.5
06-09-2016	11.4	-63.6

2