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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[1,2], Cor van der Veen[3], Ane de Boer[4], Herbert van der Ham[5]

Summary

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

[1] Corresponding author. Politecnico, Universidad San Francisco de Quito. Diego de Robles y Pampite, Sector Cumbaya, Quito, Ecuador. elantsoght@usfq.edu.ec

[2] Delft University of Technology, Concrete Structures, E.O.L.Lantsoght@tudelft.nl

[3] Delft University of Technology, Concrete Structures, C.vanderveen@tudelft.nl

[4] Consultant, Arnhem, ane1deboer@gmail.com

[5] Ministry of Infrastructure and the Environment, Rijkswaterstaat. Herbert.vander.Ham@rws.nl

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

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

1 Introduction

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

For the construction of the Second Stichtse Bridge, the option of using high strength concrete was explored in the 1990s. This bridge would parallel the First Stichtse Bridge, a lightweight concrete box girder bridge constructed using the balanced cantilever method, which opened in 1983. Therefore, a requirement for the Second Stichtse Bridge was that it would look similar to the First Stichtse Bridge. As the use of lightweight concrete was not allowed anymore, and normal weight concrete would result in larger cross-sections, high strength concrete was selected for the construction of the Second Stichtse Bridge. The difficulty at that time, however, was that the governing codes did not include high strength concrete classes, and that no information was available about the long-term performance of these new concrete mixes. For this reason, it was decided to monitor the material properties and overall structural behavior of the Second Stichtse Bridge. The originally planned duration for the monitoring was ten years, but some parameters have been monitored for twenty years.

2 Literature review

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

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

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

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

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

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

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

From this literature review, three main conclusions can be drawn. The first conclusion is that laboratory testing on girders showed that adequate performance of high strength girders is expected based on the experimental result, but that no long-term creep and shrinkage data are available. The second conclusion is that material testing indicated that for high strength concrete the effect of creep may be lower, but this conclusion is based on maximum 430 days of following of the properties. The third conclusion is that at the time of the design and construction of the Second Stichtse Bridge, no information was available about the long-term material and structural behavior of high strength concrete box girder bridges. Even nowadays, the available data gathered over a long period of time are scarce. From the available monitoring results, it is concluded that bridges with a hinge at midspan (the cantilever connection) seem to have large deformations whereas for cases without a hinge the deflections are as predicted or smaller. Moreover, following the deflections of bridges constructed in the Netherlands with the balanced cantilever method showed that no limit value of the deflection was reached after 25 years. With this information in mind, the value of monitoring the material and structural behavior of a high strength concrete bridge built with the balanced cantilever method over a long period of time (twenty years) becomes clear. The data of this long program can be used to answer the questions that remain from the discussed literature review.

3 Description of Second Stichtse Bridge

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

The total length of the Second Stichtse Bridge is 320 m, over three spans: two end spans of 80 m and a main span of 160 m, see Fig. 3. In Fig. 3, the cross-sections are shown that are necessary when lightweight concrete ($f_{c,cu}$ = 40 MPa, with $f_{c,cu}$ the design concrete cube compressive strength), NWNSC ($f_{c,cu}$ = 55 MPa), and high strength concrete ($f_{c,cu}$ = 75 MPa) are used, respectively (1). The option with high strength concrete required 28% less concrete than lightweight concrete and 14% less concrete than NWNSC, and 11% less prestressing steel than lightweight concrete and 26% less prestressing steel than NWNSC. Since the dimensions and the weight of the high strength concrete option are smaller than for the normal strength option, larger segments could be developed and the erection time could be shortened by three months. The required amount of cement for high strength concrete was slightly higher (3%) than for light weight and NWNSC.

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

4 Long-term material behavior

4.1 Concrete compressive strength

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

The concrete compressive strength was determined on cubes from segments 5, 6, 7 and 12 after 20 years. The strength development is shown in Fig. 5. Each datapoint in Fig. 5 represents the average of the cubes associated with a single segment that were tested (usually three cubes). Unlike the results after 10 years, the results after 20 years show no decrease of the concrete compressive strength, and seem to indicate even a possible increase over time. To take into account the effect of the variability of the material, the lines of one standard deviation above and below the general average (all cubes tested at a certain point in time) are shown as well in Fig. 5. The standard deviation on the tested cubes is 5.67 MPa, which is slightly larger than the value of 4.88 MPa prescribed for cylinders in NEN-EN 1992-1-1:2005 (20). The results in Fig. 5 show that the average value plus one standard deviation after one year (138 MPa) is similar to the average value after twenty years (138 MPa). As the measured strength lies within one standard deviation from the average, it can be considered to be constant after one year. These results can also be used to reevaluate the lower strengths measured after 10 years. The measurements from cubes from segments 8 and 11 lie within one standard deviation of the average measured at one year. Only the results from segment 9 are lower. Given the performance after 20 years, the results from segment 9 can be considered as an outlier.

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

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

4.2 Concrete splitting tensile strength

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

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

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

4.3 Shrinkage

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

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

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

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

During the first ten years of monitoring, the temperature and humidity inside and outside of the box girder were measured continuously. These measurements are important for the correction of the total deformation with the effect of temperature. However, these measurements were not continued after ten years. Therefore, the temperature after twenty years of measurements was estimated based on the observations of the Royal Dutch Meteorological Institute (22). The nearest observation stations (Schiphol, Berkhout, De Bilt, Lelystad) reported temperatures on April 26th 2017 at 2 pm, the moment of data collection, ranging between 8.3^{o}C (De Bilt) and 9.7^{o}C (Schiphol). Therefore, an exterior temperature of 9^{o}C can be assumed. This temperature was used for the results shown in Fig. 11. However, the specimen on which the deformation measurements are taken is stored inside the box girder, so that the temperature of the specimen may have been larger than the exterior temperature. The average difference between the temperature inside and outside the box girder was determined based on the measurements of temperature of the first ten years as 1.5^{o}C (23). The largest temperature differences were observed during the winter and summer seasons, and are maximum 9^{o}C. During the spring months, such as the time period when the last set of deformation measurements were taken, the largest observed differences between the temperature inside the girder and outside are 5^{o}C. Therefore, it is assumed that the temperature inside the box girder can have been maximum 14^{o}C when the last set of deformation measurements were taken. These results are shown in Fig. 12, showing that the maximum shrinkage strain is still less than $\varepsilon_{sh} = 0.6 \times 10^{-3}$.

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

4.4 Creep

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

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

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

In addition to the laboratory tests, measurements were taken on the web of the box girder for segments 2, 13, and 15. Since a specimen inside the box girder (Fig. 12) was monitored for shrinkage, the difference in behaviour between the web and the shrinkage specimen can be attributed to creep. Therefore, the measurements on the web have to be corrected for the effect of shrinkage (determined based on the specimen stored inside the bridge), and the effect of temperature to find the effect of creep. The results for segment 13 for ten years of monitoring are shown in Fig. 14. The large increase in the strains at 120 days of age were caused by the casting of the asphalt layer. These results can be used for calculations with for example finite element models. In such a model, a discrete creep function can be implemented as a visco-elastic process. A Kelvin chain, consisting of a series of parallel springs and dashpots, can be used. The creep strains are of the same order of magnitude as the measured shrinkage strains.

5 Long-term structural behavior

5.1 Deflection measurements

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

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

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

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

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

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

The dimensions of the cross-section used to calculate *I* are shown in Fig. 3. The value of the modulus of elasticity of the concrete is taken as *E* = 46725 MPa, as measured after one year (23). In the beam model, elements with a length of 5 m (the largest length of a segment that was used during construction) are used. Each element has a constant value of *I*, which differs from the adjacent elements. When the support settlements measured in 2016 are implemented in the beam model, a hogging deflection at midspan of the central span of 11.35 mm is found. Since a sagging deflection of -52.25 mm was measured, it can be found that the net deflection equals -63.6 mm. This calculation of the deflection caused by the support settlements is carried out for all sets of measurements and given in Table 3. The resulting net deflection at midspan of the central span is then given in

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

In 2008 (23) it was mentioned that the deflections measured in 1999, at a temperature of 8^{o}C were lower and below the trend of the measurements taken at higher ambient temperatures. Similarly, it can be seen that the deflections measured in 2010, at a temperature of 8-12^{o}C were lower as well. Certainly, the influence of the temperature on the measured deflections is clear. This observation should be kept in mind when evaluating the measurement results. Since the majority of the measurements were carried out at high temperatures, and the results show that the deflections are larger at larger temperatures, the measurements are on the safe side.

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

5.2 Comparison to predictions

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

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

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

6 Discussion

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

7 Summary and conclusions

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

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

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

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

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Tables

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

Date	Sup 2 (mm)	Span (mm)	Sup 3 (mm)	*T* (°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

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

Date	Sup 2 (mm)	Span (mm)	Sup 3 (mm)	*T* (°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

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

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