1. CANTILEVER CONCRETING TECHNOLOGY

Cantilever concreting technology in bridges was first used in 1951 by U. Finsterwalder during the building of a bridge over the Lahn Bulduinstein River. In these times, bridges constructed using this technology did not usually exceed half of the designed 100-year service life. Recently, several thousand bridges of this type have been made in the world e.g. [1, 2]. In Poland, in the years 1963–1973, cantilever concreting technology was used in three bridges (including two built as a cantilever assembly of precast concrete elements) [3]. Another large group of bridge objects [2, 5, 6, 7, 8] was created after constructing a bridge in Toruń [4] in 1998. Therefore, Polish experience in the use of cantilever concreting technology is only equal to a dozen or so years.

Cantilever concreting (or assembling) technology is one of the modern methods of constructing concrete bridges. Its main advantage is the savings made in materials, scaffolding costs and formwork, and above all, the possibility of building a span in many places at the same time. The latter, and especially the cyclicity of concreting individual segments, shortens the time of construction. Cantilever concreting technology in bridges is effective when a span length is between 50 and 250 m.

The typical feature of these bridges is their external appearance, which is shown in Figure 1. Their geometric characteristics are adapted to the adopted technology and load system in the construction phase. In these types of long-span prestressed concrete bridges, the static scheme that occurs during the construction of the cantilevers has the main influence on the system of internal forces. Therefore, these bridges have a classically shaped structural system in the form of a girder with a box cross-section of a variable height.

THE EFFECTS OF DEAD LOADS IN CANTILEVER CONCRETING BRIDGES

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Abstract
Cantilever concreting technology is one of the modern methods of constructing concrete long-span bridges. Characteristic feature of those bridges is the long-term span deflection resulting from the rheological processes in the concrete and in the pre-tensioning steel. It can also be caused by the material deterioration, e.g. concrete cracking, as well as the changes in the bridge structure, such as the support settlements. The aggregate result of bridge exploitation are the changes in its grade line, considered in this paper as the bridge span deflection line. The aim of the paper is the assessment of the internal forces on the basis of the bridge span deformation. Furthermore, an algorithm for the correction of the deflection function determined on the basis of surveying measurements (low precision measurements) is proposed. It is characterized by a significant improvement of the computational results, and it hardly “smoothen” the primary measurement results. The algorithm can be used to analyse the selected part of the bridge structure, e.g. the longest span. The paper proposes a universal coefficient of cantilever deflection, which is calculated on the basis of the cantilever joint moment when the final static scheme of the bridge is created. It can be used for the comparative analyses of various bridges. The value of the coefficient is dependent on the geometry of the cantilever box cross-section only.

Keywords: Cantilever bridge; Long-term deflection; Internal force.
A characteristic feature of many bridges, as objects of large spans made of prestressed concrete, are their excessive deflections [9–13]. An excessive deflection occurs when it exceeds the permissible value of the index $\omega = 1.25\%$, which is calculated using the following formula:

$$\omega = \frac{w}{L} \%,$$

where $w(t)$ [mm] is the displacement of the mid-point of the bridge span with length $L$ [m].

2. DEFLECTION OF A CANTILEVER DURING CONSTRUCTION

During the construction of a bridge, detailed calculations of the deflection of the cantilever span are carried out in order to obtain an appropriate grade line of a bridge object. These calculations include the self-weight of a structure and the effects of assembly prestressing. Technological loads are also important in the case of the deflection of a cantilever. The deformation analysis of the cantilever takes into account the intensive rheological processes occurring in concrete, and also the current climatic conditions. Such calculation results are necessary in order to control the assembly elevation of the entire facility, and in particular, of each segment being built. For the above-mentioned reasons, individual calculations are made, as well as a forecast of the bridge's grade line for each object.

In this section, a particular phase of construction is analyzed, as shown in Figure 2 – the moment of joining cantilevers, in which technological loads do not occur. Therefore, this is a special situation when the principle of the object’s operation changes – there is a transition from the static scheme of the cantilever into a continuous multi-span system. The simplified calculations of the deflection of a cantilever span that resulted from the self-weight of the structure, moment $M$ and the assembly prestressed force $S$, are presented below. It is assumed that these loads cause stresses on the top edge of the cantilever slab that are equal to zero, which gives the following strength condition.

$$\frac{S}{A} + \frac{S \cdot e}{I_x} v_g - \frac{M}{I_x} v_g = 0$$

(2)

All static and geometric values are functions that are variable in relation to the central axis of box cross-section $x$. The eccentricity of prestressing $e$ is determined on the basis of the distance from the top surface of bridge slab $a$, as shown in Figure 2 and the following formula

$$e = v_g - a$$

(3)

The prestressing force is obtained from condition (2)

$$S = \frac{A \cdot v_g}{I_x + A \cdot v_g (v_g - a)} M$$

(4)

as a dependence between the geometry of the trans-
verse cross-section, which is included in the following characteristics: \( I_x \) – moment of inertia, \( A \) – cross-sectional area, and \( v_g \) – distance of the inertia axis from the top surface of the bridge slab. The total bending moment in the analyzed cross-section of the cantilever, which results from the self-weight, is reduced by prestressing and is equal to

\[
M_v = M - S \cdot (v_g - a)
\]  

(5)

Because formula (4) is only used to estimate the prestressing force \( S \) (in the construction of a cantilever it changes abruptly in subsequent segments), it is preferable to simplify that \( a \approx 0 \), and then a simple expression is obtained

\[
M_v = \frac{I_x}{I_x + A \cdot v_g} M
\]  

(6)

(7)

In this way, the independence of bending moments from prestressing is obtained in formula (7), although its share is included in the ratio of moments of inertia \( I_x/I_v \).

The deflection of the mid-point of span \( x = L/2 \), however, as a cantilever it can be calculated from the Mohr formula as

\[
w_c = \int_0^{L/2} M_v \cdot x \, dx = \int_0^{L/2} \frac{M \cdot x}{E I_v} \, dx = \frac{C_0}{E} \int_0^{L/2} \frac{M_0 \cdot x}{I_v} \, dx = \frac{C_0}{E} \frac{G L}{24}
\]  

(9)

In formula (9), the function of time is omitted, i.e. changes in the properties of concrete \( E(t) \) and prestressing \( S(t) \) at the end of the construction process of the cantilever are not taken into account. In turn, the concrete creep function \( \varphi(t) \), which refers to the construction technology of the cantilever, is important.

Due to the fact that the functions in formulas (8) and (9) are a complex dependence referring to coordinate \( x \), it is convenient to make calculations in the form of a matrix, as is the case in dependencies

\[
w_c = \frac{C_0}{24 E} m \cdot B \cdot p^T
\]  

(10)

where

\[
m = \begin{pmatrix} M_1 & M_2 & M_3 & \ldots & M_k & \ldots & M_n \end{pmatrix}
\]  

(11)

and

\[
p = \begin{pmatrix} 1 & 3 & 5 & \ldots & (2k-1) & \ldots & (2n-1) \end{pmatrix}
\]  

(12)

The deflection of the mid-point of span \( x = L/2 \) in Figure 2. In this way, the independence of bending moments from prestressing is obtained in formula (7), although its share is included in the ratio of moments of inertia \( I_x/I_v \).

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\]  

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\]  

(12)

In the compliance matrix there are moments of inertia \( I_i \) that are calculated according to formula (8)
The central expression of matrix $B$ concerns node $k$. The indexes of the moments of inertia in this matrix are the numbers of elements, as shown in Figure 2. In the case of vector $m$, the indexes are the numbers of nodes. Vector $p$ is made up of whole numbers, which after being multiplied by $c/2$ give the coordinates of the centers of segments $c$ (elements of the cantilever). Therefore, the overall multiplier from formula (10) was derived from the partial components given below

$$\frac{c^4C_0}{24E} = \frac{1}{2}c^2C_0 \cdot \frac{c}{6E} \cdot \frac{c}{2} \quad (14)$$

Therefore, the expression of vector $m$ as the bending moment in the selected cross-section $k$ is determined on the basis of the constant values of the volumetric weight of reinforced concrete $C_0$, the length of the divided span sections $c$ and the variable cross-section areas $A_i$, as in the formula

$$M_k = \sum_{j=1}^{k} (2j-1)A_j \quad (15)$$

The bending moments refer to nodes. Geometric characteristics $A_i, I, \nu_g$ are the values in the middle point of the elements. The division of the cantilever into calculation elements with the length $c$ does not correspond with the division of the span into constructed segments of the cantilever.

The effectiveness of the proposed indicator is illustrated using examples of the analysis of bridges built in Poland – the results of calculations are presented in Table 1. The basic parameter of the analysis is value $G$, and the additional parameter is $\omega$, which is determined from formula (1). Both parameters give deflection $w_v$, which is calculated from formula (9) in the form of

$$\omega = 1000\frac{w_v}{L} = 1000\frac{C_0}{E}G \quad (16)$$

For calculating the value of $\omega_v$ in Table 1, the same physical characteristics of concrete – $C_o = 26$ kN/m³, $E = 36$ GPa/m² – were assumed. The comparison of $G$ values for the analyzed objects shows the increased susceptibility of the bridge in Kedzierzyn-Kozle, which was analyzed in paper [13]. A similar case also occurs in Zwierzyniecki Bridge in Cracow. The $G$ parameter can be considered as general – associated with the commonly used indicator $\omega$ [13] and the dependence that is included in formula (16). The $\omega_v$ values are similar to those found in constructed buildings.

![Figure 3. Scheme of Støvset Bridge and the increase of deflections during its operation [9]](image)
The deflection $w$ that was determined from formula (10) is not subjected to experimental verification. During construction, the fictitious middle point of the span of a structure $x = L/2$ does not exist – it only occurs after joining the cantilevers, and thus in the transition phase of the change in the static scheme. After joining the cantilevers, there is a secondary (final) prestressing of the span and an increase in the dead load due to bridge equipment. This results in a further change of the deflection line, however, this time it occurs in the static scheme of the operated object. The deflections generated during the construction phase are usually compensated in the executive lift.

3. DEFLECTION OF A SPAN DURING THE OPERATION OF A BRIDGE

The measurements of grade line changes of bridges’ spans made using the cantilever concreting method have been carried out for many years [6, 9, 11, 13, 14, 15, 16]. In the majority of these facilities there are no such operational problems as the ones that are considered in this paper. However, the phenomenon of large deflections that is analyzed in this paper is common, and until now not well investigated. A very good documented example of the analyzed problem is Støvset Bridge [9], which is shown in Figure 3a. LC55 lightweight concrete was used in the central part of this bridge, as was the case in the Stolma Bridge that has a record length of the central span $L = 301$ m. Therefore, it can be considered as an example of a bridge construction that enables large spans to be achieved. In this bridge, eight years since the end of its construction, the deflection of $w = 200$ mm exceeded the design value. The underestimated deformability of the lightweight concrete [9] was considered as the main reason for this deflection. The key in the diagrams shown in Figure 3b includes the designations of the time intervals between the reference (0) and analyzed (1–9) measurements.

A negative example of the reduction of an excessive deflection can be seen in the Koror-Babelthaupt Bridge with the span of $L = 241$ m. After 12 years of operation, the displacement in this bridge was equal to $w = 1200$ mm, and therefore $\omega = 4.98\%e$. After 18 years, it increased to the value of $w = 1390$ mm, which corresponds with $\omega = 5.77\%e$, and it exceeded the permissible value many times (1). Strengthening of the structure with the use of secondary post-tensioning did not work, and after a short period of operation ended in constructional failure [11].

Although concrete creeping tests have been carried out throughout the 20th century until now, the problem of large deflections of prestressed concrete bridges is still not solved. Therefore, it can be concluded that rheological processes do not reach a finite value during 100 years of bridge operation [17]. The phenomenon of large deflections is well recognized due to the monitoring of bridge objects [12].

The purpose of this paper is to determine changes in internal forces that result from deflections that are considered in a function of time $w(t)$, and which are obtained from the measurements of bridges built using cantilever concreting technology. Figure 4 shows the changes in the deflection that occurred in the middle of the length of a span that was built using cantilever concreting technology. These changes were presented in the form of a time dependency [9].

The waveforms of the deflections of the spans that were built using cantilever concreting technology can be shown in three time ranges [13]. In the initial period of several years after the completion of the construction phase, the increase in deflections is by far the largest. In the first year, the progress of the deflection is the highest and in the following years there is a slow stabilization. The second period involves the balanced growth of deflections, as can be seen in Figure 4. The third period, which is the longest period of bridge service life (about 3/4 of bridge service life), can only be a forecast, which is due to the lack of measurement data. The current results of measurements from a period of 30 years show an increase in deflections, which is a slow and thus unstable process [17].

<table>
<thead>
<tr>
<th>No</th>
<th>Location</th>
<th>$G$ [m]</th>
<th>$\omega$ [%]</th>
<th>$L$ [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Grudziądz</td>
<td>1174</td>
<td>0.848</td>
<td>180</td>
</tr>
<tr>
<td>2</td>
<td>Płock - projekt</td>
<td>1297</td>
<td>0.937</td>
<td>148.6</td>
</tr>
<tr>
<td>3</td>
<td>Brzeg Dolny</td>
<td>1189</td>
<td>0.859</td>
<td>140</td>
</tr>
<tr>
<td>4</td>
<td>Kędzierzyn-Koźle</td>
<td>1796</td>
<td>1.297</td>
<td>140</td>
</tr>
<tr>
<td>5</td>
<td>Kraków</td>
<td>1583</td>
<td>1.143</td>
<td>132</td>
</tr>
<tr>
<td>6</td>
<td>Łany k. Wrocławia</td>
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<td>0.857</td>
<td>120</td>
</tr>
<tr>
<td>7</td>
<td>Miłówka k. Wisły</td>
<td>1059</td>
<td>0.765</td>
<td>82</td>
</tr>
</tbody>
</table>

(*) a bridge with a frame scheme and $L = 200$ m.
4. FUNCTIONS OF DEFLECTION'S CURVATURE

The effect of changes in the bridge’s grade line that come from the results of measurements of ordinates \( r(x, t_p) \) and \( r(x, t_k) \), which were obtained in two considered time periods \( t_p < \tau < t_k \), is treated as the function of deflection

\[
w(x, t) = r(x, t_k) - r(x, t_p)
\]  

(17)

This deflection is not caused by moving loads, but instead by the dead loads of the bridge: the self-weight of the structure and equipment, as well as prestressing. During the deflection of the span, the modulus of the concrete’s deformation \( E(t) \) changes and the moment of inertia \( I(x) \) depends on the position of the considered cross-section of the beam. When the deflection of the span is considered as an extemporary effect, the bending moment occurs as in dependence

\[
M(x, t) = E(t) \cdot I(x) \cdot \frac{d^2 w}{dx^2} = E(t) \cdot I(x) \cdot \kappa(x, t)
\]  

(18)

If homogeneity of the concrete in the segments is assumed, then the bending stiffness \( EI_c(t, x) \) in equation (18) can be adopted as the total value. In the case of complex systems made of various concretes (ordinary and light) – as is the case in the example of the analyzed object [9] – it is justified to use sections with separate material features.

From the form of solution (18), it is possible to analyze the section of the structure that is separated from the system. In the selected part of the deflection function there are no boundary conditions, e.g. the method of supporting the span or the type of load. Formula (17) shows the possibility of analyzing the changes in the grade line for any chosen time interval. Therefore, it is possible to analyze the state of the structure from the reference measurement to any chosen measurement of deflection in the key, as shown in Figure 4. The deflection can also be treated as a dependence between any measurements, e.g. 3 and 7 from Figure 3.

In formula (18), there is a second derivative of the deflection that results in the curvature of the beam in the considered point. In practice, the curvature of the beam in point \( j \) can be obtained on the basis of deflections in adjacent points \( i \) and \( k \), as in the differential equation

\[
\kappa_j = \frac{1}{E(x) c^2} \left( w_i - 2w_j + w_k \right)
\]  

(19)

For this purpose, the measurement of the grade line and deflections at regularly located points along the length of the assessed element with the value \( c \), which were calculated from (17), are used. Thus, the curvature calculated from dependence (19) is determined on the basis of the deformation of the beam on section \( 2c \). In the case of using the deflection function \( w(x) \), the value of \( \kappa(x, t) \) is a derivative in the analyzed point \( j \). The mathematical comparison of both values shows the approximation of their values when the \( c \)-section tends to zero. However, in the case of measurements on an object, the differences in the value \( w \) in points \( i, j, k \) also tend to zero. Therefore, in practice, the precision (accuracy) of measurements may be of great importance in the selection of \( c \).

Figure 5 presents function \( \kappa(x, t) \), which was calculated on the basis of the diagrams \( w(x, t) \) shown in
5. CORRECTION OF THE CURVATURE DETERMINED FROM MEASUREMENTS

An algorithm of re-calculating the deflection using the Mohr relationship, which is used in structural mechanics for modified bar systems, is proposed in the paper in order to improve (smooth out) the curve of function $\kappa(x, t)$

$$w_j = \int \kappa(x, t) M_j(x) dx \quad (20)$$

In formula (20), the deflection of point $j$ is calculated using the deflection function $M_j$ that was formed due to the bending moments resulting from the unit force in point $j$, which was determined in a convenient statically determinable scheme, e.g. a simply supported beam. If the curvature had been calculated within the differential approach as in (18), the obtained value $w_j$ would be identical to the initial one. Of course, this is the case when calculations are made using a uniform FEM model.

In the case of using the curvature that is determined from the measurements and formula (19) instead of using the integral approach as in (20), it is convenient to use the matrix algorithm as in equation

$$w_j = \frac{c}{6} \left( \kappa^T \cdot B \cdot m_j \right) \quad (21)$$

In this approach, when the beam line is divided into a sequence of segments with length $c$, it is possible to create vectors $\kappa^T$ and $m_j$ from the curvatures and functions found in (20). These are the values of these functions at the beam measuring points. Assuming that the functions $\kappa(x, t)$ and $M_j(x)$ are continuous, you can use the matrix form $B$ in the calculations as in the table

$$B = \begin{bmatrix} . & . & 4 & 1 \ . & 1 & 4 & 1 \ . & . & . & . \end{bmatrix} \quad (22)$$

From formula (21), a slightly different value of deflection $w_j$ than from the measurements is obtained. This is due to the use of $\kappa(x, t)$, which is calculated as a differential approach with a form as in (19). In this case, the accuracy of the measurements of the grade line, as well as the deflections calculated using (17) as in Figure 3, is of paramount importance. The procedure that is presented in (21) can be repeated multiple times using the previous calculation result in order to create $\kappa(x, t)$. In this way, the subsequent form of the function is created from the recalculation of function $w(x, t)$. Therefore, this is a procedure of subsequent approximations. Figure 6 shows the course of the smoothing of function $\kappa(x, t)$ with the assumption of deflections from the final measurement 9, as in Figure 3. An important element
of the iterative process is the tracking of changes in the deflection function after subsequent shape corrections (marked in the key as kor1–kor4), as in Figure 7.

It is important in the iteration process that the deflection functions do not differ significantly from the initial form that was obtained from the measurements. However, it is assumed that the measurement is carried out correctly and does not contain an erroneous reading. Figures 6 and 7 present the results after the third correction. Further corrections of these diagrams may be inappropriate with regards to the description of the course of the phenomenon. The change in curvature does not have to be an absolutely smooth function, as is the case in a steel rolled beam that is loaded with a uniformly distributed force. Local disturbances of function \( \kappa(x,t) \) can also be a result of changes in the structure of concrete, e.g. scratches or local slipping of cables.

In the interpretation of the results it is necessary to realize that the result of calculations in the form of function \( \kappa(x,t) \) refers to a situation from the analyzed time period. Therefore, the current state of the analyzed element is not considered because the result does not refer to the initial value in the unknown situation being analyzed. From the diagram in Figure 7, it can be concluded that the changes in the curvature in the support zone are clearly reduced and do not increase as is the case in the fixed beam.

6. CHANGES OF BENDING MOMENTS AND STRESSES

The general relationship between unit strains \( \varepsilon(t) \) and normal stresses \( \sigma(t) \) in the rheological model of concrete according to Trost [18] is determined by the equation
In this paper, a transformed formula (23) is used in the form of a relation between curvature $\kappa(t)$ and bending moment $M(t)$, as in formula

$$\varepsilon_t = \frac{1}{E} \left[ \sigma_p(1+\phi_t) + \left( \sigma_k - \sigma_p \right)(1 + \rho\phi_t) \right] + \varepsilon_{sk} \quad (23)$$

In the initial situation, the curvature of the beam that is subjected to bending with moment $M_p$ is equal to

$$\kappa_p = \frac{M_p}{EI} \quad (25)$$

In the final phase of the measurements, according to formula (24), the curvature changes into the value

$$\kappa_k = \frac{1}{EI} \left[ M_p(1+\phi_k) + (M_k - M_p)(1 + \rho\phi_k) \right] \quad (26)$$

The process of a change in $\kappa(t)$ and $M(t)$ is taken into account according to (24) in the relaxation index [18], and $EI$ is the bending stiffness of a bar in the analyzed cross-section.

Figure 8 shows the general dependence between the change in curvature and bending moments as a function of time in the approach considered in this paper. On the horizontal axis, three characteristic situations were identified and defined by the following points:

$\phi$ – beginning of loading, $p$ – beginning of measuring the bridge’s grade line, $k$ – completion of the tests. In the analyzed moments of time, curvature and bending moments assume the values given in Figure 8. In the considered time interval, the creep function takes the values of $\phi_t$, with the values given on the horizontal axis. Therefore, the ordinates from the diagrams in Figure 8 determine the flow of time, but are included as a function of the concrete’s creep.

In formulas (23–28), one cross-section is considered along the length of the span. Therefore, there are moments $M_p(x)$ and $M_k(x)$, as well as different changes in curvature in the form of functions $\kappa(x)$ in each of the analyzed cross-sections, as in Figure 6.

Moment function $M(x)$ refers to the bending intensity along the span, but its values are difficult to interpret. A better measure is the stresses $\sigma(x)$, which can be related, e.g. to the strength of concrete. Therefore, the stress function is determined based on the bending moment calculated in (28)

$$\sigma(x) = \frac{M(x) \cdot \kappa(x) + M_p(x) \cdot \left[ 1 - (1-\rho)\phi_p \right]}{1 - \rho\phi_p} \quad (29)$$

where $v_i$ is the distance of the analyzed point of the cross-section (edge) from the axis of inertia.

Figures 9 and 10 show the results of calculations and tests of the bridge that is presented in Figure 1 in an analogical approach to the previously analyzed Støvset Bridge. Figure 9 shows the change in the grade line of the bridge in Kedzierzyn-Kozle, which...
occurred during an observation period of 76 months (with the initial measurement after 30 months from the time of joining the cantilever spans). The measurement result (pom) was adapted to the support conditions of the span (trans). This is based on the transformation of the original diagram in order for the deflection ordinates to be zero at the support points, i.e. when $x = 0$ and $x = L = 140$ m. As a result
of correcting the curvature, the results of which are summarized in Figure 10, the diagram marked as (obl) was created.

Figure 11 shows the diagram as the values resulting from the first component of formula (29), and therefore only shows the effect of the change in curvature

\[ \sigma_i(x) = \frac{E}{1 - \rho \phi \sigma} \kappa(x) \cdot \nu_i(x) \]  

(30)
The second component of formula (29) takes into account all the information about the construction technology of the span, laying equipment and, above all, data on the rheological process. When creating the diagram given in Figure 11, the value of \( E = 25000 \text{ MN/m}^2 \) was adopted from formula (30) without additional justification. This is because the accurate reproduction of rheological processes during the construction process, as well as the real state of concrete strain during the operation of the object, was not considered. The estimated negative values (tensile stresses) in Figure 11 are of significant importance.

6. CONCLUSION

A characteristic feature of bridges as large span objects made using cantilever concreting technology is the formation of excessive deflections (\( w > L/800 \)), which result from the rheological processes occurring in concrete and prestressing steel [10]. Moreover, deflections in these objects may be a result of material destruction, such as cracks or a change in the construction’s load scheme, e.g. subsidence of supports. The total effect of exploitation is the changes in the bridge's grade line that are observed on site in the results of the conducted surveying measurements. The difference in grade lines between two selected observation times (measurements) is treated in the paper as the deflection line of the span. Deformation of the span is accompanied by a change in the internal forces and support reactions in the structure, which is treated in this paper as a beam.

The surveying measurements result in the characteristic functions of the span’s deflection with a parabolic shape. Due to the accuracy of surveying measurements, it is not possible to calculate the internal forces using derivatives of function \( w(x) \) from the deflection diagrams. Therefore, an algorithm for correcting the deflection function was proposed in this paper. It is characterized by a significant improvement of the results of calculations and, to a small extent, leads to smoothing of the original measurement results. The algorithm is adapted to the local analysis of the selected section of a structure, e.g. the span with the largest length that was built using cantilever concreting technology. The advantage of the algorithm is its ability to analyze the structure in any chosen time interval (between the two considered measurements). The results of such analyzes are not related to the initial state, e.g. the moment of joining the structure.

A separate issue in concrete cantilever bridges is the construction phase. Its feature is the large dispersion of measurement results of deflection, which is caused by many factors with random characteristics, such as: construction technology, construction time, concreting time, climate, concrete strength, used aggregate, reinforcement grade, prestressing ratio, and the most important – rheological processes. Therefore, calculations for the construction phase must be conducted individually. The paper proposes a general deflection coefficient (16), which is calculated for the moment of joining the cantilevers, as well as the creation of an operating scheme. It can be used in comparative analyzes of various objects. Its value depends on the geometric indicators of the box cross-section of a cantilever while fulfilling a condition regarding prestressing (2).

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