Metal deck forms as lateral bracing of composite bridges with trapezoidal cross sections

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Permanent form av plåt som sidostagning av samverkansbroar med trapetsformat tvärsnitt

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Emelie Ohlin
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Abstract

A critical stage during construction of composite bridge is when casting the concrete on top of the steel girders. The bare girders carry the load from wet concrete, formwork, construction workers etcetera and are exposed to the risk of lateral torsional buckling.

This failure mode is when the cross section is both twisting and moving laterally, and can be prevented by lateral restraints. Commonly used formwork is performed by trapezoidal shaped metal decks spanning between the top flanges of the girders. By taking in account for the relatively high in-plane shear stiffness of metal sheets, they can be utilized also as the lateral restraint (Eglimez et al, 2005). Due to efficiency and cheaper constructions this performance is developing within the bridge industry.

In June 2002 the Y1504 Bridge collapsed in north of Sweden during the deck concreting. The bridge was supplied with trapezoidal metal decks, supposed to work both as permanent formwork and as lateral restraint. However, a combination of changed performance of the bridge where e.g. another metal deck than the prescribed one was used along with lacking of design controls for the deck, the bridge collapsed due to lateral torsional buckling.

In this thesis the failure reports of the bridge was examined, showing the lack of knowledge at the time of failure of designing a metal deck working as lateral restraint. Theory performed by Helwig et al. after the time of failure was also studied and the required stiffness for the metal deck to act as proper bracing for Y1504 Bridge was calculated. The results were compared to the actual stiffnesses calculated according to SDI, showing that both the prescribed and the used metal deck would have obtained sufficient stiffness to work as restraints. The strength of the deck was also checked according to this theory by calculating the forces acting on the screws. This show that the attachments between the top flanges and the metal decks lack the required resistance. This failure transformed the crossed cross section of the Y1504 Bridge into an open cross section which was exposed to lateral torsional buckling.

In addition to these analyses the Y1504 Bridge was modelled in ABAQUS and FE-analyses were performed with different parameters analyzed. These analyses show that the used deck should have had sufficient strength to sustain loading from the concreting, in opposite to the prescribed metal deck that would have failed. It shows that the number of fasteners has a great impact on increasing the safety against lateral torsional buckling, but was not decisive in this bridge case. A thickness of 0.65 mm would be sufficient for the used deck but due to the large forces acting on the screws a 5 mm thick metal deck is necessary to avoid failure of the edge of the screw holes. Perhaps if the connections between the metal decks and the top flange was performed differently, avoiding the concentrated forces i.e. with welds, the Y1504 Bridge might not have failed.

The chain is no stronger than its weakest link so for future designing of decks, only checking the shear flow resistance and the rotational stiffness of the system is not enough. It is important to have a metal deck with sufficient shear stiffness for the particular cross section for it to be restrained, in combination with secure attachments.
# Table of contents

1. **Introduction** ........................................................................................................................ 1
   1.1 **Background** .................................................................................................................. 1
   1.2 **Aim and scope** ............................................................................................................. 2

2. **Literature review and theory** .............................................................................................. 5
   2.1 **Lateral torsional buckling of beams** ............................................................................ 5
   2.2 **Global buckling of twin girder system** ........................................................................ 7
   2.3 **Horizontal truss bracing** ............................................................................................. 8
   2.4 **Metal deck as lateral bracing** ..................................................................................... 9
   2.5 **Stiffness and strength requirements of metal deck** .................................................... 10
       2.5.1 **Stiffness requirement** .................................................................................. 10
       2.5.2 **Required stiffness for Y1504** ............................................................................. 12
       2.5.3 **Strength requirement** .......................................................................................... 12
       2.5.4 **Required strength for Y1504** .............................................................................. 14
   2.6 **Calculating stiffness of metal deck on Y1504 according to SDI** .................................. 15
       2.6.1 **Theory** ................................................................................................................ 15
       2.6.2 **Result** .................................................................................................................. 17

3. **Description of the case study** ............................................................................................. 19
   3.1 **Background** ................................................................................................................ 19
   3.2 **The Y1504 Bridge** ..................................................................................................... 19
   3.3 **Failure mode** .............................................................................................................. 20
   3.4 **Investigation of failure** ............................................................................................... 21
   3.5 **Evaluation of calculations in failure reports** ............................................................... 24
       i. **Evaluation of failure report by Professor 1** ............................................................ 24
       ii. **Evaluation of failure report by Professor 2** .......................................................... 25
       iii. **Evaluation of failure report by Professor 1** ........................................................ 26
       iv. **Evaluation of failure report by Professor 3** ........................................................... 26
       v. **Summary of evaluation of failure reports** ............................................................... 28

4. **FEM model of Y1504** ........................................................................................................ 29
   4.1 **Dimensions** ................................................................................................................ 29
   4.2 **Material properties** .................................................................................................... 31
   4.3 **Meshing and element types** ........................................................................................ 31
   4.4 **Boundary conditions and loads** .................................................................................. 32
   4.5 **Course of action** ......................................................................................................... 34
   4.6 **Results from FEM** ...................................................................................................... 36
1 Introduction
In respect to the integrity of the firms and people involved in the analyze of the failure of the Y1504 Bridge, their names have been encoded in this thesis.

1.1 Background
In recent years, several bridge failures due to instability problems have occurred during concreting of the deck of composite bridges with trapezoidal cross sections, sometimes with unfortunate outcome. When the Marcy Bridge in New York, figure 1-1, collapsed in 2002 due to this cause, one person was killed and nine people were injured. (Corr et al., 2009).

When a composite bridge with steel girders and a concrete deck on top is designed, a critical phase often during casting of the concrete deck. When eventually the concrete deck has hardened the lateral torsional stiffness of the entire system is high and it is able to resist large forces. However, during the casting of the concrete, bare steel girders have to resist loading from its own self weight, the wet concrete, form work, concreting machines and workers, without any strength from the concrete. When the girders are subjected to this high bending moment, plus the fact that the bridge is not yet stabilized from the hardened concrete deck, the risk of lateral torsional buckling is very high and needs to be prevented with lateral bracing (Eglimez et al., 2007).

One solution, not very commonly used in the bridge industry yet, is to count for the lateral restraint that the permanent form work of metal decks provide. These metal sheets, spanning between the top flanges of the adjacent steel girders, have a sufficiently high shear resistance to act as lateral restraint and therefor prevent lateral torsional buckling of the bridge during casting of the deck.

The Y1504 Bridge, a composite bridge with a trapezoidal cross section, was designed and performed with this permanent metal deck formworks spanning between the flanges of the bridge. These were designated to also act as stabilizing elements for the box girder, and therefore the performance of these decks was of course a sensitive and important factor for maintaining the desired strength of the entire bridge system. Unfortunately, it was not clear to the contractors that the metal deck had such important influence to the stability of the entire system and the installation was not performed according to the design specifications. This is common in the construction industry where it is custom to perform changes if they result in a cheaper solution but still fulfil the same requirements as the prescribed. This miscommunication together with insufficient design calculations for the stabilizing function of the metal deck unfortunately led to collapse of the bridge during the construction phase.
After evaluating the reason of failure of the Y1504 Bridge, it mostly indicates that the shear forces became too high for the screws connecting the metal decks to the flanges to resist. This led to failure of the screws which in turn turned the former closed bridge section to an open section with a significantly lower torsional resistance and the bridge thereby failed due to lateral torsional buckling.

Due to both time and money efficiency in using the lateral restraint of the metal decks it is of great interest to find a good design approach for them to prevent these kind of failures. After the failure of the Y1504 Bridge, and after the failure reports were made, some new research was made. Helwig et al. performed research about the requirements for the stiffness and strength of metal decks acting as lateral torsional bracing, this to keep the cross section intact.
and not losing the torsional stiffness, which will in worst case lead to lateral torsional buckling of the bridge.

1.2 Aim and scope
The aim of this study was partly to find the resistance of the Y1504 Bridge, performed through a FE-analysis, with the metal decks working as lateral restraints, but with altering parameters prescribed below. This resistance was to then be compared to the bending moment acting on the bridge to find out in what conditions the bridge would have failed or not.

The aim was also to, by studying the failure of the bridge, get a greater understanding about what aspects are important to look at when designing a steel beam bridge with permanent metal decks as formwork, making the metal decks also to work as lateral stabilizers.

The study is divided in three different parts which are at the end of the thesis combined and analysed together.

i) In the first part, the author was investigating the failure reports of the Y1504 Bridge performed by several well renowned Swedish constructors. Then they were analysed by studying the different design methods to find what conclusions the constructors make and which content of the reports are of interest for future design of metal decks working as lateral torsional bracing of steel bridges.

ii) The second part is a theoretical study. Theory about stiffness and strength requirements of metal decks working as lateral torsional bracing performed by Eglimez et al. (2007) and by Helwig and Yura (2008) that were performed after the failure of the Y1504 Bridge, was studied. Were these new design methods applicable on the Y1504 Bridge and would the constructors received different results if they had had access to this theory at the time of failure?

iii) The third part of the study is a FE-analysis that partly investigated if the Y1504 Bridge would have failed if the metal decks where designed and performed to resist lateral torsional buckling. It also includes parametric analyses to find if a variation could have eliminated failure. The bridge as it was performed in reality was modelled in the FE-program Abaqus FEA and linear buckling analyses are performed to find the buckling modes and critical bending moment. The FE-model was then separated in to different models with one variable parameter and the impact of the varying parameters was analysed separately. The altered parameters were:

1. Number of screws attaching the sheets to the top flanges, altering between one and two screws per valley of the metal deck.
2. Attaching the metal decks with end fasteners through an extension of the flange according to figure 1-1.
3. The shape of the metal deck, altering between the prescribed sheet TRP 45 according to figure 1-2 and the constructed sheet PEVA45 according to figure 1-3.
4. Varying thicknesses of the metal decks.
Figure 1-1 End of beam with device for end fasteners

Figure 1-2 Prescribed sheet TRP 45. Reproduced from (Armat. 2008)

Figure 1-3 Constructed sheet PEVA 45. Reproduced from (Kumlaviken AB. 2010)
2 Literature review and theory

2.1 Lateral torsional buckling of beams

Beams subjected to a load bending the beam in the stiffer plane i.e. about the y-axis according to figure 2.1, are exposed to the risk of lateral torsional buckling. This buckling is a combination of lateral movement and a twist as shown in figure 2.2. (Gardener, 2011)

According to EC 1993-1-1, a laterally unrestrained beam subjected to bending moment must be checked against lateral torsional bending according to equation 2.1.

\[
\frac{M_{E,d}}{M_{b,Rd}} \leq 1,0
\]  

(2.1)

Where

- \( M_{E,d} \) is the design bending moment
- \( M_{b,Rd} \) is the bending moment resistant

In a similar way as the simple buckling design of a column, the design lateral torsional bending resistance moment \( M_{b,Rd} \) for an unrestrained beam is defined in equation 2.2.

\[
M_{b,Rd} = \frac{\chi_{LT}W_yf_y}{\gamma_{M1}}
\]  

(2.2)

Where

- \( \chi_{LT} \) is the reduction factor for lateral torsional buckling
- \( W_y \) is the major axis section modulus, according to:
  - \( W_y = W_{pl,y} \) for Class 1 and Class 2
  - \( W_y = W_{el,y} \) for Class 3
  - \( W_y = W_{eff,y} \) for Class 4
- \( \gamma_{M1} \) is the partial safety factor
The reduction factor for beams $\chi_{LT}$ is decided using different buckling curves according to EC 1993-1-1, section 6.3.2, depicted in figure 2-3.

![Buckling curves for reduction factor $\chi_{LT}$](image)

**Figure 2-3 Buckling curves for reduction factor $\chi_{LT}$. (EN 1993-1-1)**

A slenderness $\bar{\lambda}_{LT}$ need to be determined according to equation 2.3 which in its turn is depending on a critical bending moment.

$$\bar{\lambda}_{LT} = \sqrt[\frac{W_y f_y}{M_{cr}}}$$  \hspace{1cm} (2.3)

The course of action to determine this critical moment is similar to when designing a strut, using Euler’s critical force.

For the simple case of uniformly distributed load on a simply supported beam where the beam ends are prevented from lateral and vertical movement and twisting, this critical elastic moment is defined in equation 2.4. (Timoshenko and Gere, 1961).

$$M_{cr} = \frac{\pi^2 E I_y G J + \pi^2 E^2 I_y C_w}{L_b}$$  \hspace{1cm} (2.4)

Where

- $L_b$ is the length of the beam between points along beam prevented from twist
- $E$ is the modulus of elasticity
- $I_y$ is the week axis second moment of area of the section
- $G$ is the shear modulus
- $J$ is the torsion constant of the section
- $C_w$ is the warping constant of the section
This solution is only concerning a single girder and is transformed by Yura (2008) into a
criteria for the twin girder system where all section properties are for the single girder,
presented in equation 2.5.

\[ M_g = 2 \frac{\pi}{L_g} \sqrt{EI_y GJ + \frac{\pi^2 E I_y}{4L_g^2} (I_y h_0^2 + I_x S^2)} \]  

(2.5)

Where

- \( L_g \) is the entire length of the beam
- \( S \) is the spacing between the girders

The conservative approximation of the simplified global buckling moment for the twin girder
system, \( M_{gs} \), is shown in equation 2.6. It is depending on the presumption that the \( I_y S^2 \) term is
dominating and can be used to determine whether global buckling is at risk or not.

\[ M_{gs} = \frac{\pi^2 SE}{L_g^2} \sqrt{I_y T_x} \]  

(2.6)

Equations 2.4-6 are only regarding doubly symmetric sections and for a singly symmetric
system where the upper smaller flange is in compression Yura (2008) has defined a design
formula for global lateral torsional buckling that can be used both for singly and doubly
symmetric sections, presented in equation 2.7 with the conservative formula in equation 2.8.

\[ M_{gl} = 2C_b \frac{\pi E}{L_g^2} \sqrt{\frac{l_{yc}}{I_y} + \frac{\pi^2 I_y h_0^2}{L_g^2} + \frac{\pi^2 I_{eff} I_y S^2}{4L_g^2}} \]  

(2.7)

\[ M_{glis} = C_b \frac{\pi^2 SE}{L_g^2} \sqrt{I_{eff} T_x} \]  

(2.8)

Where

- \( C_b \) is a factor accounting for the moment gradient
- \( I_{eff} \) is the effective moment of inertia = \( I_{yc} + (b/c) l_{yt} \)
- \( l_{yc} \) is the moment of the compression flange about the y-axis
- \( b \) is the distance from the centroid axis to the tension flange
- \( c \) is the distance from the centroid axis to the compression flange

After comparing FEA solutions to this equation, Yura (2008) shows that a good assumption
for the critical global buckling moment for a singly symmetric section, loaded at the top
flange is \( 0.9M_{gl} \).

2.2 Global buckling of twin girder system

When considering lateral torsional buckling today, using the current prescribed design
approaches only the single beam is checked together with the buckling capacity between the
bracing of the girders (Yura, 2008). A common failure mode for beams only braced with
intermediate X- or K-bracing is a connected rotation for the pair of beams and global buckling
over the entire span length as shown in figure 2-4 (Hendy and Jones, 2009). Depending on the
dimensions of the girders, this global buckling mode can occur before buckle in between the
bracing of the girders occur. A well performed diagonal truss system as shown in figure 2-5
will result in a greater moment of inertia about the week axis than about the strong axis and
global buckling is prevented. The weaker connections with intermediate cross frames or diaphragms between the girders will still be exposed to the risk of global buckling (Yura, 2008). Bridge systems most exposed to the risk of global buckling mode are bridges with large span-to-width ratios and two girders, connected with these particular connections just mentioned.

Figure 2-4 Global buckling mode of twin girders. Reproduced from (Hendy and Jones, 2009).

2.3 Horizontal truss bracing
Global lateral buckling of beams can be prevented by either increasing the distance between the girders or by increasing the moment of inertia of the cross-section. These solutions though are often limited by geometrical regulations and not possible to construct (Yura, 2008). A better solution, also in an economical matter is to add plan bracing to the system. By connecting two parallel girders by either a triangular system in plan or vertically, as shown in figure 2-5, the risk of lateral buckling is decreasing as the lateral stability of the system is increasing. Full lateral restrain is achieved at the attachments for the plan bracing and lateral torsional buckling should still be checked between these (Gardener, 2011). If only the torsional bracing system is installed, that is the vertical bracing system, the entire system is still in risk of global buckling and the effective length is not to be reduced to the length in between the bracings. But when adopting the plan bracing, it is actually sufficient to add this between the top flanges just for a few fields at the supports to resist global lateral torsional buckling (Yura, 2008). Providing only a fifth of the entire length at each support with plan bracing provides warping restraints that can be assimilated with fixed ends (Yura, 2008).

The plan bracing can be performed by trapezoidal metal decks spanning between the girders. If it is conducted properly the equations for the critical bending moment presented in section 2.2 can be used, but then the length $L_g$ should be replaced with $0.6 L_g$, based on the FEA performed by Yura (2008). Also, if the girders are prevented from lateral movement by the supports, $C_p = 1.0$ (Yura, 2008).
2.4 Metal deck as lateral bracing

When the concrete deck of a composite bridge has hardened, the bridge is fully restrained and laterally stabilized by the concrete deck and its diaphragm action (Gardener, 2011). But when the concrete is still wet and the bridge has not yet achieved its final stiffness from the composite action, the lateral stability of the bridge is significantly lower. At this stage, the girders are carrying the load from the wet concrete by itself and their buckling capacity needs to be increased, lateral torsional bracing is required.

It is common in both bridge and building designs to use metal decks, spanning between the top flanges of two main girders, as temporary formwork during concreting of the deck. Their great in-plane shear stiffness are often enough for the sheets to also work as lateral stabilizer, that is if the attachments to the girders are performed in a good way. They have the capacity to resist lateral loading and also warping between the top flanges that they are attached to (Eglimez et al., 2007). If the corrugated sheets are placed perpendicular to the steel girders as figure 2-6 shows, the system is laterally restrained by the metal deck and stabilized due to the shear stiffness of the deck (Gardener, 2011).
The shear stiffness per unit length, S, of a trapezoidal sheet can be calculated according to equation 2.9. (EN 1993-1-3.)

\[ S = 1000 \sqrt{t^3 \left( 50 + 10 \cdot 3 \sqrt{b_r} \right)} \frac{s}{h_w} \]  \hspace{1cm} (2.9)

Where

- \( t \) is the thickness of the sheet in mm
- \( b_r \) is the length of the sheet, perpendicular to the girder directions in mm
- \( s \) is the spacing between the girders in mm
- \( h_w \) is the depth of the sheet in mm

To ensure that the sheets contributes with full restraint, the shear stiffness, according to EC 1993-1-1, BB.2.1, needs to fulfil the requirement in equation 2.10, assuming the sheet is connected to the girder at every rib. If the sheets instead only are connected at every second rib, it is prerequisite with a shear stiffness five times higher. (Gardener, 2011)

\[ S \geq \frac{700}{h^2} \left( EI_w \frac{\pi^2}{l^2} + GI_T + 0.25h^2 EI_z \frac{\pi^2}{l^2} \right) \]  \hspace{1cm} (2.10)

Where

- \( S \) is the shear stiffness of the sheet
- \( L \) is the length of the beam
- \( h \) is the height of the beam

Well conducted between the girders, the decks are working as diaphragms, restraining lateral movement of the top flanges and they are the most effective in the end of the girders where the shear deformations are the highest. A test made by Eglimez et al. (2009) shows that the stiffness of a system with two parallel steel girders will increase around 10 times when metal decks are applied to the system on top of the girders. They tested both to deck the entire length of the beams, and also only decking the end parts of the bridge, approximately a fourth of the entire length at both ends. The results shows a very small difference in lateral stiffness of the fully deck and the partially decked system. This in turn follows an intuition that the decking closest to the supports are most important to the stabilization, since this is where the largest shear forces will occur.

2.5 Stiffness and strength requirements of metal deck

When using metal decks as bracing for steel beams there are requirements both on the stiffness and the strength of the decks. This section contains requirements for these according to Helwig and Yura (2008).

2.5.1 Stiffness requirement

When studying the stiffness of the metal deck the shear rigidity Q, which is the product of the shear modulus \( G' \) and the width \( s_d \) of the deck providing bracing to the beam, is sufficient to investigate. The width \( s_d \) of the deck is determined through equation 2.11.

\[ s_d = \frac{n_g^{-1}}{n_g} \cdot S \]  \hspace{1cm} (2.11)

where
\( n_g \) is the number of girders
\( s \) is the spacing between the girders [m]

Research on the behavior and the requirements on metal decks working as bracing for bridges has been made in the 1960’s and 1970’s. Errera and Apparao in 1976 as well as Nethercot and Trahair in 1975 derived a simplified way of determining the critical bending moment, \( M_{cr} \), for a beam braced with metal decks at the top flanges, see equation 2.12.

\[
M_{cr} = M_g + 2Qe
\]

where
\( M_g \) is the buckling capacity for an unbraced beam [kN/m]
\( Q \) is the shear rigidity of the metal deck [kN/rad]
\( e \) is the distance between the center of gravity of the cross section to the plane of the metal deck [m]

This equation on the other hand was only valid for a double symmetric beam subjected to uniform moment and therefore needed to be modified so it could be used for different types of loading and also for single symmetric cross sections. Helwig et al. (1999) provides an equation shown below that is applicable both for singly and doubly symmetric cross sections, where a \( C^*_b \) factor is introduced that takes into account for gradient of moment so that the equation not only is valid for uniform moment. For top flange loading they recommend to use \( C^*_b = C_b/1.4 \) and the \( C_b \) factor for uniformly distributed load is often taken as 1.14. They also show that the distance, \( e \) is the distance between the mid-height of the beam to the plane of the metal deck, both regarding the doubly and singly symmetric cross section, which turns the factor 2\( e \) in the equation above into \( d \), the height of the cross section, in equation 2.13.

\[
M_{cr} = C^*_b \cdot M_g + mQd
\]

where
\( C^*_b \) is a factor for moment gradient including the effects of load height
\( d \) is the height of the cross section of the beam [m]
\( m \) is a factor depending on the location of the loading point
\( M_g \) and \( Q \) are defined above

By combining and rearranging equations 2.11-13 together with \( Q = G' \cdot s \cdot d \) the requirement for the shear modulus of the metal deck working as bracing for beam bridges is obtained as presented in equation 2.14.

\[
G' = \frac{(M_{cr} - C^*_b M_g)(n_g)}{m \cdot (n_g - 1) \cdot s \cdot d}
\]

This stiffness is the so-called ideal stiffness of the metal deck, and to be able to control the deformations and the bracing forces, a stiffness of four times the ideal stiffness must be obtained (Helwig and Yura, 2008).

In a research made by Helwig et al in 2008, they conclude that the parameter \( m \) is not only depending on the location of the loading, but also on the slenderness of the beam web and if the beam has discrete intermediate bracing or not. Depending on these parameters different \( m \)
as presented in table 2-1 were obtained where h and \( t_w \) is the height and the thickness of the web.

Table 2-1 Values of \( m \) for different conditions depending on beam slenderness and bracing presence

<table>
<thead>
<tr>
<th>Bracing condition</th>
<th>Centroid loading</th>
<th>Top flange loading</th>
<th>Centroid loading</th>
<th>Top flange loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>No intermediate discrete bracing</td>
<td>0.85</td>
<td>0.5</td>
<td>0.5</td>
<td>0.375</td>
</tr>
<tr>
<td>With intermediate discrete bracing</td>
<td>0.85</td>
<td>0.85</td>
<td>0.5</td>
<td>0.375</td>
</tr>
</tbody>
</table>

When designing the metal deck with respect to the stiffness, equation 2.15 together with the parameter \( m \) chosen from table 2-1 above can be used, where the critical moment \( M_{cr} \) is replaced with the maximum design moment acting on the beam, \( M_u \) (Helwig, 2008). All the ingoing parameters are defined above.

\[
G' = \frac{4(M_u - C_r M_g)}{m s d d} \quad (2.15)
\]

If the metal deck reaches this effective shear stiffness it is sufficient to work as bracing for a steel bride, preventing it from lateral torsional buckling.

### 2.5.2 Required stiffness for Y1504

Using this theory, the required stiffness of the metal deck for bracing the Y1504 Bridge is 4876 kN/m, which is four times the ideal stiffness. Detailed calculations are presented in Appendix II.

### 2.5.3 Strength requirement

It is also necessary to control the strength of the deck. When considering this, Helwig et al. (2008) recommends to design it by calculating the maximum brace moment per unit length of the beam \( M_{br}' \) according to equation 2.16.

\[
M_{br}' = 0.001 \frac{M_u L}{d^2} \quad (2.16)
\]

where

- \( M_u \) is the maximum moment acting on the beam
- \( L \) is the length between the bracing points restraining twist
- \( d \) is the height of the cross section

This expression is based on an initial deformation of \( \theta = L/500d \) and a shear rigidity of the metal deck, \( Q \) of four times the ideal stiffness. If the deformation differs from this the calculated force can be changed proportionally and if the stiffness of the deck is higher than four times the ideal value Helwig et al. (2008) describes a way of recalculating the bracing forces by simply reducing them with the factor \( C_r \) derived in equation 2.17.

\[
C_r = \frac{3}{4} + \frac{1}{4} \left( \frac{Q_{req}}{Q_{prov}} \right)^2 \quad (2.17)
\]

where
Q_{\text{req}} \text{ is the required stiffness = four times the ideal stiffness}
Q_{\text{prov}} \text{ is the provided stiffness}

The brace moment per unit length of the beam $M'_{\text{br}}$ is depicted in the figure below. This is also a picture showing a way of predicting the forces acting on the fasteners, which is often the critical part when determining the strength of the metal deck.

The effect from the moment $M_{\text{br}}$ and the shear force $V_{\text{br}}$ can be divided into force components $F_M$ and $F_V$ that are acting on the fasteners. Looking at figure 2.9 and saying that the shear force $V_{\text{br}}$ is taken equally by the fasteners, and if there are $n_e$ fasteners per sheet in the longitudinal direction of the beam, then the force component from the shear force $F_V$ on the fastener can be derived as in equation 2.18. Observe that $M'_{\text{br}}$ is prescribed above.

$$F_V = \frac{V_{\text{br}}}{n_e} = \frac{2M_{\text{br}}}{n_e L_d} = \frac{2M'_{\text{br}} w_d}{n_e L_d} = \frac{2 \cdot 0.001 M_{\text{br}} L_d}{d^2 n_e L_d}$$

(2.18)

According to Egliremez et al. (2005) the stress distribution in the metal decks is depicted in figure 2-7 and the forces from the brace moment will then approximately be resisted by the fasteners as simplified depicted in figure 2-8. From these assumptions the force acting on the fasteners as a result from the brace moment can be derived through a force equilibrium as presented below. Observe that this derivation is an approach of calculating the force and is accurate for a sheet with five fasteners. The same approach is applicable for any other number of fasteners, but must then be modified according to equation 2.19.

---

**Figure 2-7 Bracing moment and shear force on metal deck.** (Egliremez et al., 2008).

**Figure 2-8 Stress distribution on metal deck.** (Egliremez et al., 2008).
When both the force from the shear force and the brace moment is calculated, the total force acting on the fasteners $F_{\text{tot}}$ can easily be determined through equation 2.20 and used as a design value.

$$F_{\text{tot}} = \sqrt{F_V^2 + F_M^2}$$ (2.20)

### 2.5.4 Required strength for Y1504

The strength of the sheets are mostly depending on the fasteners between the girders and the sheets. According to the strength requirement of Helwig, the forces acting on the fasteners are presented in table 5-1 below with calculations in Appendix III.

**Table 2-2 Forces acting on fasteners due to brace moment, based on a stiffness of four times the ideal**

<table>
<thead>
<tr>
<th>Total force/fastener</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>PEVA45, 4 one fastener per valley</td>
<td>30.6 kN</td>
</tr>
<tr>
<td>PEVA45, 4 two fasteners per valley</td>
<td>22.5 kN</td>
</tr>
<tr>
<td>TRP45, fastener in every valley</td>
<td>27.25 kN</td>
</tr>
</tbody>
</table>

It is important to know that these results are based on that the stiffness of the metal deck is four times the ideal stiffness. If the stiffness is differing from this required stiffness the forces will be modified, increased forces with lower stiffness and vice versa.

Considering the TRP sheet where the stiffness calculations according to SDI are more reliable than for the PEVA45 sheet, due to the fact that the stiffness is higher than the required stiffness (see section 2.6) the forces should be reduced by a factor $C_r$ of 0.77 according to equations 2.17 calculated below.

$$C_r = \frac{3}{4} + \frac{1}{4} \left( \frac{Q_{\text{req}}}{Q_{\text{prov}}} \right)^2 = \frac{3}{4} + \frac{1}{4} \left( \frac{4876}{16561} \right)^2 = 0.77$$
This gives the total force of 21.0 kN acting on the screws connecting the TRP45 sheet to the top flanges.

2.6 Calculating stiffness of metal deck on Y1504 according to SDI

2.6.1 Theory

The effective shear stiffness of metal decks working as bracing on bridges can be calculated according to the Diaphragms Design Manual, Second edition, from the Steel Deck Institute (SDI). The approach according to SDI is presented in this section.

The shear stiffness of a metal deck used for bridge application is based on the geometry of the plate and the connector flexibilities and is determined through equation 2.21. All equations in the SDI manual is compatible with the American system of measuring i.e. inches and feet for length and kip for forces.

\[ G' = \frac{E \cdot t}{2.6 \cdot d^{2} \cdot \phi \cdot D_n + C} \]  

(2.21)

where

- \( E \) is the elasticity modulus
- \( t \) is the thickness of the metal deck
- \( s \) is the girth of the corrugated metal deck
- \( d \) is the pitch of the corrugation
- \( \phi \) is a reduction coefficient for \( D \) that depends on the number of similar spans, =1 for the twin girder system with one span
- \( D_n \) is the warping constant =D/12L
- \( L \) is the panel length
- \( C \) is the slip coefficient

The warping constant \( D_n \) contains a mix of several different other warping constants \( D \), that is depending on the arrangements of the fasteners at the ends of the panel. Below DW1 and DW2 is derived which represent fasteners in every respectively in every other valley. In figure 2-10 the indications are depicted. The same derivation is available for every third and fourth valley but they are not presented here.

\[ WT = 4f^{2}(f + w) \]
\[ WB = 16e^{2}(2e + w) \]
\[ PW = 1/t^{1.5} \]
\[ A = 2e/f \]
\[ V = 2(e + w) + f \]
\[ D1 = \frac{1}{3}h^{2}(2w + 3f) \]
\[ D2 = D1/2 \]
\[ D3 = \frac{1}{12}a^{2}[V(4e^{2} - 2ef + f^{2}) + d^{2}(3f + 2w)] \]
\[ C1 = 1/(D3 - D2/2) \]
\[ C2 = 1/[e(D2/f) + D3] \]
\[ C4 = A/[e(D1/f) + D2] \]

Figure 2-10 Description of equation indications
The equation for the slip coefficient \( C \) in SDI second edition is not suitable for bridge application rather than for building application, but a suitable equation can be found in the first edition of SDI, see equation 2.22. The slip coefficient \( C \) depends on the shear forces directly at the side laps which, in turn, depend on the number and location of fasteners in a panel.

\[
D4(1) = (24f / C1) \left( \frac{c1}{W_f} \right)^{0.25} \\
D4(2) = (24f / C2) \left( \frac{c2}{W_f} \right)^{0.25} \\
D4(4) = (48e / C4) \left( \frac{c4}{W_B} \right)^{0.25} \\
G4(1) = D4(1) \\
G4(2) = 2[D4(2)] + A[D4(4)] \\
DW1 = G4(1)(f / d)PW \\
DW2 = G4(2)(f / 2d)PW
\]

\[
G(2) = 2G(1)(f / d)PW
\]

\[
\frac{C}{a} = \frac{24E \cdot L \cdot S_f}{a} \left( \frac{n_{sh} - 1}{2\alpha_1 n_p x_2 + 2n_s S_f / S_s} + \frac{1}{2\alpha_1 n_p x_2 + n_e} \right)
\]

(2.22)

where

- \( S_f \) is the flexibility for the fasteners connecting the sheet and the beam
- \( S_s \) is the flexibility for the fasteners connecting the sheets
- \( a \) is the overall panel width
- \( n_{sh} \) is the number of individual deck sheets per panel
- \( \alpha_1 \) is the end distribution factor = \( \Sigma X_e / W_{sh} \)
- \( \alpha_2 \) is the purlin distribution factor, similar to \( \alpha_1 \)
- \( X_e \) is the distance from the center line to each fastener, see figure 2-13
- \( W_{sh} \) is the width of the individual deck
- \( n_p \) is the number of purlins = 0 for bridges
- \( n_s \) is the number of side lap fasteners per seam
- \( n_e \) is the number of edge connectors = 0 in this case

**Figure 2-11 Illustration of** \( X_e \)**

For No. 12 and No. 14 Buildex TEKS screws connecting sheets to heavier substrate material the screw flexibilities \( S_f \) and \( S_s \) is presented in SDI as

\[
S_f = 1.3 \cdot 10^{-3} / t^{0.5} \text{ in/kip}
\]

\[
S_s = 3.0 \cdot 10^{-3} / t^{0.5} \text{ in/kip}
\]
2.6.2 Result
According to the approach presented in section 2.6 the shear stiffness of both the used metal deck PEVA 45 and the prescribed TRP 45 is calculated, both with attachments in every and in every other valley. Detailed calculations are presented in Appendix IV and the result are shown in table 2-3.

*Table 2-3 Calculated stiffness of metal decks according to SDI*

<table>
<thead>
<tr>
<th>Place of attachments</th>
<th>PEVA 45</th>
<th>TRP 45</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Every</td>
<td>Every</td>
</tr>
<tr>
<td>Shear stiffness [kN/m]</td>
<td>11526</td>
<td>1727</td>
</tr>
<tr>
<td></td>
<td>Every</td>
<td>Every</td>
</tr>
<tr>
<td>Shear stiffness [kN/m]</td>
<td>19994</td>
<td>2687</td>
</tr>
</tbody>
</table>
3 Description of the case study

3.1 Background
In January 2002, construction company 1 and technical consultant firm 1 started off their cooperation for designing and constructing the Y1504 Bridge over Gide Älv in the northern part of Sweden. Y1504 was a composite bridge with a trapezoidal shaped steel girder connected to a concrete deck and technical consultant firm 1 was hired by construction company 1 as a calculation consultant to perform the construction documents. On the day of the construction start, on June the 12th, the bridge collapsed during concreting of the deck. Six people were located on the bridge during the collapse but everyone were luckily uninjured. Failure studies made after the collapse shows that the execution of the bridge was not performed as it was prescribed in the construction documents made by technical consultant firm 1. Through onsite visits and investigations the actual design of the bridge could be done in retrospect. It turns out that the metal decks intended to work as permanent form work was not performed by the on construction document prescribed make. Nor the design of the screws connecting the sheets to each other or the ones connecting the sheets to the girders was performed as prescribed. Neither the dimensions of the screws or the spacing in between them. Several failure reports where conducted after the collapse, both by construction company 1 and technical consultant firm 1, but also by external parts.

3.2 The Y1504 Bridge
The Y1504 Bridge was a simply supported composite bridge that covered a 65 meter long span and had a clear bridge width of 7 meters. The beam had a trapezoidal steel box cross section with a concrete deck on top according to figure 3-1. The composite action was supposed to act between shear studs placed on the top flanges, and the concrete deck. Along the bridge, nine steel diaphragm walls with a central hole were placed with different spacing together with two end diaphragms at each end. The exact dimension of bridge is presented in section 4.1.

![Figure 3-1 Principal cross section of the Y1504 Bridge](image)

Working both as permanent form work and as lateral stabilizer, metal decks were attached to the top flanges of the steel girder. Although, in the construction documents, the deck type TRP45 was prescribed, the permanent form work for the concrete deck was build up by another type of corrugated metal deck, i.e. PEVA 45. According to a measurement made by Lars Åström at the Luleå University of Technology, the computational thickness of the used metal deck was 0.72 mm. This is to be compared to the computational thickness of 0.755 mm for the prescribed deck. Another difference between the two metal decks are that the PEVA
45 is equipped with a reinforcing seam in the profile bottom according to figure 3-2, which made it impossible to attach the sheets to the girders centric in the profile bottoms. According to the makers Kumlaviken AB (2010), this reinforcing seam in the profile bottom is working as bottom reinforcement for a cheaper and more time efficient construction. This reinforcing seam along with the dimensions are shown in figure 3-2, compared to the prescribed TRP45 in figure 3-3 below.

![Figure 3-2 Cross section with measurements of PEVA 45. Reproduced from (Kumlaviken AB. 2010).](image)

![Figure 3-3 Cross section with measurements of TRP 45. Reproduced from (Armat. 2008).](image)

The metal decks were attached to the top flanges through screws with a diameter of 4.5 mm, one screw in every profile bottom. In the overlap joints the decks were attached to each other with nine screws per joint with a diameter of 4.8 mm, the spacing was approximately 250-300 mm. Even this screw diameter and spacing differed from the construction documents where the screw diameters where prescribed to 6.3 mm with a spacing of 150 mm. The metal deck was supported by a wooden joist in the mid line of the bridge and the joist in turn was upheld by a support placed inside the box cross section. Except for the last two sheets closest to support 1, all of the permanent form work was installed when the failure occurred. The metal decks were not attached neither to the end crossbeam nor the diaphragm walls.

Before the concreting started a small height difference of 15 mm between the two top flanges in the center of the bridge was discovered but this difference was within the limits of the tolerance and therefore not adjusted.

### 3.3 Failure mode

The performance on the concreting of the deck was divided in to different stages. The first stage of concreting was in the center of the bridge, was 33 meters long and involved approximately 66 m$^3$ concrete, corresponding to a line load of 48 kN/m. To eliminate eccentric loading the casting started in the center of the middle line of the bridge so that the concrete would spread evenly towards the two end supports and the end beams. When approximately 17 m$^3$ of the concrete was casted failure occurred when the beams suddenly
rotates 90°. When all of the concrete had drained of the form work together with some of the
reinforcement, the beam raised back against its original position, loosening the form work
from one of the top flanges in the center part of the bridge. In this new equilibrium position
the end of the beams had cambered considerably and the bridge seat at support 1 was pushed
back and failed.

3.4 Investigation of failure
The following section is a resume of failure reports, meetings minutes and other notes
involving the failure conducted by the construction company 1 and the design consultant
Technical consultant firm 1 Sverige AB, further mentioned construction company 1 and
technical consultant firm 1. This in order to get a better understanding about possible
parameters affecting the failure. The sequence of events are presented in chronological order
and the investigated calculations with its respective constructor are as follows

i. Professor 1, 02-11-05, Calculations with performance according to drawings and
   according to actual performance at failure
   in Kärrsjö”
iii. Professor 1, 2003-08-15, Failure of Y1504 Bridge over Gideälv in Kärrsjö: Comments
    to report made by Professor 2, technical consultant firm 2 AB
iv. Professor 3, 2004-08-10, Failure of the Y1504 Bridge over Gideälv in Kärrsjö

It is obvious that there are several different approaches to determine the ability for a metal
deck to work as bracing and therefor these failure reports, performed in different ways, are of
great interest. For more detailed calculations, see Appendix I.

The evaluation of the failure began immediately after the collapse. Without performing
calculations technical consultant firm 1 assumed that the failure had to be due to either
deficiencies in material specification, wrong material in sheets, the concreting execution,
mounting errors or error in standards (Failure meeting 1, 2002). At the request from technical
consultant firm 1, calculations on the torsional stiffness of the beam during concreting were
performed and preliminary results showed adequate safety. (Failure meeting 2, 2002)

After an on-site visit of the collapsed bridge, technical consultant firm 1 found out that the
metal deck that was used as permanent form work was not performed by the prescribed
TRP45, but by PEVA 45. The screws attaching the sheets to the girders were conical in shape
and only one screw per valley was applied, not on each side of the fold, which in turn were
deformed. The seam fasteners between the sheets were attached with 4.8 mm screws with a
spacing of 300 mm instead of the prescribed 6.3 mm with a spacing of 150 mm. It was also
discovered that the metal deck had detached from the center of the bridge and at one of the
supports.

After studying the calculations performed by technical consultant firm 1, construction
company 1 argued that the ones for the construction stage were deficient. There were no
calculations for either lateral torsional bracing nor global buckling for the entire system
during construction. Calculations due to buckling of sheets were also missing. The
calculations made by technical consultant firm 1 after failure, showing adequate safety,
included conditions as connection to end beams which was not accurate for Y1504 and would
therefore not be reliable (Failure meeting 3, 2002). Instead, construction company 1 believed
that, due to the low level of loading, the primary reason for failure presumably was that the
permanent form work was not able to transfer the resulting bracing forces. Failure if the construction was made according to instructions would not occur at this stage (Memo from construction company 1, 2002).

Technical consultant firm 1 claimed that the permanent form work was necessary for transmitting the horizontal force component from the shear force in the tilting beam webs. The sheets connect the girders and are then primary stabilizing as well. Technical consultant firm 1 also proclaimed that attachment in diaphragms or end beams were unnecessary due to bearing resistance. (Letter correspondence between construction company 1 and technical consultant firm 1, 2002-09-11-17)

The thickness of used PEVA45 sheet was determined through Luleå University of Technology 2002-11-04 to 0.720 mm.

According to technical consultant firm 1, the biggest deformations when a sheet is exposed to shear force is due to the fact that it is only attached in the profile bottom. In this case, the used PEVA 45 provided approximately 80% more shear deformations compared to a metal deck without a reinforcing seam in the profile bottom. But if two screws were attached, one screw on each side of the reinforcing seam, this deformation would decrease significantly. They proclaimed that despite this high shear deformations, if the metal deck would have worked as supposed to, the safety against lateral torsional buckling were too big to be the reason of failure and something else made the sheet not work as supposed to (Letter correspondence between construction company 1 and technical consultant firm 1, 2002-09-11-17). The eccentricity of the concreting together with a potential initial rotation overloaded the screws in the sheet joints which in turn led to failure of the edge of the screw holes and failure of the metal decks. When this happened the closed cross section became open and lateral torsional buckling occurred. So, they proclaimed that the main reason of failure was a decreased shear stiffness due to less screws with lower strength that decreased the shear strength of the sheet. But if the performance was made according to the construction document, failure would not have occurred. (2002-11-05, Professor 1)

According to a report from technical consultant firm 2 at the request from construction company 1, the initial rotation of the beam was 0.0053 rad. After concreting the first stage in two different sets, this rotation increased first, after approximately 8 m$^3$ concrete, with 0.0039 rad and after the second stage there was an additional increase by 0.013 rad. Due to this increase of rotation, the rotational moment also increased, and this in turn led to an increase of the shear flow in the sheet. The transversal joints between the sheets became gradually more loaded until finally failure of the edge of the screw holes occurred. technical consultant firm 2 claims that the construction would also have failed if it was performed as prescribed, but at a later time. (2003-04-10, Report from technical consultant firm 2, P.O. Professor 2. Questions from construction company 1)

According to calculations made by Professor 3 (2008), Chalmers, the failure would not have occurred during the first stage of concreting if the original designed bridge was intact. He proclaims that failure would still occurred if construction company 1 would have performed he construction as the construction documents prescribed, since the shear resistance of the TRP 45 is too low and that the situation became worse when the sheets are not attached to the end of the bridge. He stated that the failure occurred due to progressive shear failure in the
sheets at both ends of the bridge, which in turns led to lost torsional stiffness of the bridge and finally failure due to lateral torsional buckling. (Professor 3, 2004-08-10)
As a summary, the parameters that could have been affecting the failure of the bridge according to construction company 1 and technical consultant firm 1 are

- Using a different metal deck than prescribed
- Using a different kind of fasteners and the spacing between them
- No connection between the metal decks and the end beams or diaphragms
- Thickness of the sheeting
- One versus two fasteners per valley
- Ignoring the influence of initial twist of girders

Some of these parameters will be checked in the FE-analyses to find their impact on the resistance of the bridge.

3.5 Evaluation of calculations in failure reports

In the following section the calculations in the failure reports performed by well reputed constructors has been studied to get an understanding about what methods the investigators have used when evaluating if the metal deck, whether the prescribed one or the used one, was able to work as a stabilizing element of the bridge.

i. Evaluation of failure report by Professor 1

Professor 1 is making his calculations according to StBK-N5, Standard for thin-plate constructions (“norm för tunnplåtskonstruktioner”), “Handboken Bygg” and T. Höglund, 2000, “Stabilisering genom skivverkan”, SBI Publ 169, a supplemental handbooks to European standards.

He calculates the effective thickness of a plane plate that gives the same shear deformations as the prescribed sheet TRP 45. This gives a thickness of 0.045 mm. He uses the program CrossOpt to calculate the variables of the cross section which results in a moment of inertia of $I_y = 3.51 \cdot 10^{11}$ mm$^4$, a torsional stiffness factor of $K_V = 2.14 \cdot 10^9$ mm$^4$ and a warping stiffness factor of $K_w = 9.481 \cdot 10^{16}$ mm$^6$.

According to StBK-K2 he then controls lateral torsional buckling by calculating the bending moment capacity of the steel beam with a closed cross section in the quarter point of the bridge, to 37.2 MNm, comparing this with the acting bending moment of 22.2 MNm. This gives a security of 1.678 against lateral torsional buckling. The bending moment capacity is also calculated when considering a beam without any warping stiffness and this results in 35.7 MNm which also gives sufficient safety against lateral torsional buckling. The specific calculations for controlling lateral torsional buckling with an open cross section is not available in the calculations hold by the author, but professor 1 writes that the safety against it is below 1.0 which is considered to be reliable.

So professor 1 stated that lateral torsional buckling was not the only reason of failure since the safety against this is sufficient if the sheet would have secured a closed section. But if the metal deck on the other hand was not obtaining sufficient shear stiffness for some reason, preventing the cross section from being closed, the number of safety would then decrease below 1.0, which is the case here.
These are the only calculations hold by the author from this evaluation from professor 1. He proclaims that calculations of the forces in the screws and nails are made with consideration to rotational moment is made, both for the prescribed and the used performance, but these specific calculations are not available in this thesis. But in words he describes that a calculation has been made where both an eccentricity of the applied concrete of 0.2 meter and an initial rotation of the cross section is taken in account for. Both of these result in a rotational moment which gives add forces to the overlap joints and the screws attaching the sheet to the top flanges. For the used sheet PEVA45 a security way below 1.0 is calculated for the overlap screws which then will be subjected to failure in the edge of the holes. The calculations for the sheet TRP45 and attachments according to the drawings on the other hand shows that the forces acting on the overlap screws would be considerably lower since the rotational stiffness of this system would be higher, and the failure would not then have occurred. The calculation and values of the stiffness is unfortunately not found in the obtained evaluation.

ii. Evaluation of failure report by professor 2
Professor 2 is commissioned by construction company 1 to answer questions regarding the planned basis with drawings etc., the performance of the construction and the effect of the performed changes. His calculations are performed according to Bro 94 (Vägverket, 1994), the general technical description from the Swedish Road Administration for bridges. The separate appendix with the performed calculations are not in possession of the author, only the results of the calculations are therefore investigated.

The professor estimates an initial rotation of the bridge of 1/200 and uses a recommended crookedness with a maximum amplitude of 97.5 mm. He also uses the wind load recommended by BRO 94 since no investigation has been made, but the wind still seems to have a significant impact of the stability.

The calculations are divided in to two cases;

- case 1, the performed construction
- case 2, the prescribed construction

He calculates the effective thickness of the PEVA 45 to 0.036 mm and an effective thickness of 0.055 mm for TRP 45, this according to “European Recommendations for the Application of Metal Sheeting acting as a Diaphragm” Publ. No 88, European Convention for Constructional Steelwork, Brussels 1995. Although, he proclaims that the basis in these recommendations are not really sufficient to take in account for the extra stiffness due to the reinforcing seam on the PEVA 45, and this seam should result in a decrease of the shear stiffness. Despite this assumption professor 2 still uses the calculated effective thickness of 0.036 mm.

He also claims that the shear flow capacity of the cross section is critical in the overlap joints, where failure of the edge of the screw holes is the critical part. The estimated shear flow capacity calculated according to StBK-N5 for failure of the edges of the screw holes is 3.6 kN/m for case 1 and 8.3 kN/m for case 2. Looking at the shear flow capacity in the attachments between the sheets and the top flanges it amounts to 15.1 kN/m for case 2 and 17.1 kN/m for case 2, which due to the higher value are not of interest.
When calculating the shear forces acting on the bridge, the professor first controls the load combination without any applied concrete, only applying the self-weight, rotations, imperfections and the wind load. Already at this load combination it is shown that the shear flow capacity for both case 1 and case 2 is insufficient, the shear flow for the cases namely becomes 19.6 and 20.2 kN/m respectively. Knowing this, professor 2 calculates the amount of concrete possible to apply before the shear flow capacity of 3.6 kN/m•γn (γn=1.2) for case 1 is reached, which amounts to 15 m³ of concrete, compared to the reality where about 17 m³ of concrete was applied when the failure occurred. For case 2, the shear flow capacity of 8.3 kN/m•γn (γn=1.2) is reached after applying about 45 m³ concrete, which corresponds to approximately 68% of the first stage of concreting. In these last calculations no wind load is taken in account for.

### iii. Evaluation of failure report by professor 1

When reviewing the calculations of professor 2, professor 1 presents the difference in their calculated effective thickness of the sheets according to table 3-2 which he claims should be one of the reasons of their differing results. Especially for the used sheet PEVA 45.

**Table 3-1 Used effective thicknesses of metal decks according to Professor 1 and Professor 2**

<table>
<thead>
<tr>
<th>Prescribed (TRP 45)</th>
<th>Used (PEVA 45)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Professor 1</td>
<td>0.045 mm</td>
</tr>
<tr>
<td>Professor 2</td>
<td>0.055 mm</td>
</tr>
</tbody>
</table>

The difference depends on different approaches where professor 1 has used the Swedish Standard for thin-plate constructions, unlike professor 2 which uses the European recommendations that does not consider the dimension of the screws.

He also proclaims that the shear flow capacity for TRP 45 calculated by professor 2 is performed wrong and should instead of 8.3 kN/m be 11.3 kN/m which should be reached after applying approximately 70 m³ of concrete which is more than the first stage of concrete which consists of about 66 m³. In a further answer from professor 2 to this assumption (see Appendix I) he calculates the possible amount of concrete applied on the bridge with TRP 45 with the effective thickness of 0.045 mm to reach the shear flow capacity of 11.3 kN/m, all according to professor 1. The result is 54 m³, 82 % of the first stage of concreting. Applying the entire first stage would result in a shear flow of 15.2 kN/m.

### iv. Evaluation of failure report by professor 3

On basis on the already performed calculations by professor 2, professor 3 uses the effective thickness of the TRP 45 to 0.055 mm. Also when deciding the shear flow capacity of the metal deck, instead of performing a new investigation he observes the already calculated values from the manufacturer Plannja and from the former investigations by professor 2 and professor 1, and decides to use a design value of 12 kN/m.

Professor 3 analyses the failure by using structural mechanics. The first two questions he asks himself are if the metal deck, studying the Plannja TRP 45 deck, has sufficient strength to resist the design shear flow, and if it together with its nail and screw joint has sufficient shear stiffness to stabilize the bridge in full during the first stage of concreting. The metal decks braced at the ends of the bridge are most exposed and needs to resist the tendency of warping...
of the cross section. They need to carry the maximum shear flow and also have sufficient shear stiffness to give security against lateral torsional buckling during concreting.

At first he determines the properties for the cross section and compares the shear center for both an open and a closed cross section. It turns out that due to the thin plate (0.055 mm), the shear center only moves 14 mm closer to the geometric center when closing the cross section. The difference in the torsional moment will therefore not be noticeable between the open and the closed section.

He estimates the initial deformation/rotation of the bridge to 0.34 degrees which will increase with the increasing load level. The rotation gives an additional rotational moment at the supports of the bridge that amounts to 170 kNm. (36 kN/m from the self-weight of the steel construction, 95 kNm from the concrete from the first stage of concreting and 39 kNm due to a tilt of the formwork providing uneven spread of the concrete). This rotational moment subjects a shear flow of 15 kN/m around the cross section that the sheets (both the permanent form work, the bottom plate and the webs) at the ends of the bridge needs to resist, calculated according to the equation below.

\[ D = \frac{T_{\text{max}}}{2A_{\text{in}}} = \frac{170 \text{ kNm}}{2 \cdot 5.6 \text{ m}^2} = 15 \text{ kN/m} \]

where

- \( T_{\text{max}} \) is the rotational moment
- \( A_{\text{in}} \) is the enclosing area of the cross section.

But since the sheets are not attached to the diaphragms at the ends of the bridge, this gives a shear flow at the ends of 0 kN/m and a maximum value close to the ends of a value higher than 15 kN/m, about \( 1.4 \cdot 15 = 21 \text{ kN/m} \).

This estimation of the shear flow is compared to the shear flow capacity of the metal deck that professor 2 presents in his calculations (\( D = 13 \text{ kN/m} \) alternatively \( 9 \text{ kN/m} \)) and the value professor 1 calculated for the same deck (\( D = 11.3 \text{ kN/m} \)) which according to professor 3 obviously is too low. He proclaims that this value instead should be in the range 15-25 kN/m. When calculating the capacity of a sheet with the shear flow resistance of 12 kN/m professor 3 finds that this sheet will fail after applying about 29 m\(^3\) concrete, which corresponds to approximately 44 % of the first stage of concreting. So the conclusion of this is that the shear resistance of the sheet is too low to resist the first stage of concreting, and the corrugated deck will fail at the longitudinal joints at the ends of the beam.

The St Venant rotational stiffness, \( G K_v \), of the closed cross section is calculated to 161 MNm\(^2\) which is extremely low. The thin plate with the equivalent thickness of 0.055 mm (based on the calculations by professor 2) is decreasing the \( K_v \) factor of the torsional stiffness a lot, to 2016 \( \cdot 10^6 \text{ mm}^4 \), making the bridge beam as rotationally stiff as for example four square pipes (V KR) 450x250x16.0. If the sheet instead was as thick as the bottom plate of 22 mm the \( K_v \) factor of the rotational stiffness would have been 128 times bigger. But on the other hand, an open cross section without the sheet closing the cross section, would have a 72 times lower value of the \( K_v \) factor of the St Venant rotational stiffness.

Professor 3 considers an intact cross section of the bridge with the initial rotation, applying the load from the self-weight and a part \( \lambda \) of the concrete from the first stage of concreting as
a point load, \( Q = Q_{self} + \lambda \cdot Q_{conc} \). He uses this to calculate the proportion \( \lambda \) of the concrete from the first stage of concreting that can be applied before failure occurs. Using the calculated St Venant rotational stiffness of 161 MNm² results in a torsional stiffness \( S \) [kNm/rad] at the middle of the bridge according to the equation below.

\[
S = 2 \frac{GK_e}{L/2} = 2 \frac{161}{65/2} = 9910 \text{ kNm/rad}
\]

The condition where the required and the provided outer rotational moment to cause the rotation \( \varphi_m \) is equal then gives

\[
S \cdot \varphi_m = (Q_{self} + \lambda \cdot Q_{conc}) \cdot \varphi_m \rightarrow \lambda = 1.8
\]

which shows that the cross section could be loaded with the entire load from the first stage of concreting and even more, without failing through lateral torsional buckling.

So, according to professor 3, if the metal decks were performed in a way keeping the cross section intact and closed during the entire concreting the bridge would not have been subjected to lateral torsional buckling. In this case the shear strength of the metal deck is too low to resist the loading from the concreting.

v. Summary of evaluation of failure reports

The constructors agree on the fact that the bridge did fail due to the initial failure of the metal deck, if the bridge was intact the failure would not have occurred. Professor 1 says that the torsional stiffness of the built system was too low and that this was why the metal deck failed. If the prescribed metal deck was used, the entire system would obtained a higher rotational stiffness which would give lower forces on screws and nails and the failure had been avoided.

Professor 2 is making his failure assumptions on the fact that the design shear flow was too high for both of the metal decks to resist, which did not have sufficient shear strength even to resist a, from concrete, unloaded beam.

Professor 3 also claims that the design shear flow is higher than the shear strength of the deck, which is the reason of failure. By calculating the rotational stiffness of the entire beam he also shows that the intact beam would not have been subjected to lateral torsional buckling.
4 FEM model of Y1504

The three-dimensional finite element program ABAQUS was used in this study to perform buckling analyses on the Y1504 Bridge. The FE-analyses were performed in a way that changes for certain parameters can be compared to each other.

4.1 Dimensions

When modelling the bridge, the girder were divided into three parts with different dimensions where both the flanges and the webs are modelled as shell elements. The different parts were named according to figure/drawing 4-1 and the dimensions used when modelling the parts are presented both in this figure and in table 4-1.

<table>
<thead>
<tr>
<th>Part</th>
<th>Length [mm]</th>
<th>Top flange [mm]</th>
<th>Web [mm]</th>
<th>Bottom flange [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>11000</td>
<td>29x600</td>
<td>18x2088</td>
<td>22x2400</td>
</tr>
<tr>
<td>2</td>
<td>11500</td>
<td>40x600-750</td>
<td>18x2088</td>
<td>39x2400</td>
</tr>
<tr>
<td>3</td>
<td>10000</td>
<td>40x750</td>
<td>18x2088</td>
<td>43x2400</td>
</tr>
</tbody>
</table>

The total steel height from the lower edge of the bottom flange to the upper edge of the top flange was 2000 mm along the entire beam. Since the thickness of the top and bottom flanges differ between the different parts the height of the web in reality also differs between the different parts. This was not considered when modelling in ABAQUS since the top flanges are without thickness when the parts are to be assembled, and the web needs to have the same height along the entire beam. This was considered not to have any significant impact on the result. The webs had an inclination of 16.7 degrees according to figure 4-2.
Nine diaphragms, D₁-D₉, depicted as (i) plus two end diaphragms, ED depicted as (ii), in figure 4-3 were divided over the entire beam with distances according to figure/drawing 4-1. The thickness of all the diaphragms were 10 mm.

The formworks PEVA 45 and TRP45 were modelled with shell elements. Both of them were conducted with closed ends according to figure 4-4 and the length of the sheets were 2900 mm, spanning between the top girders, overlapping them by 50 mm at both flanges. The dimensions of the sheets are shown in figure 3-2 and 3-3 and the tested thicknesses are presented in table 4-4.

---

**Figure 4-2** Principal cross section of the Y1504 Bridge, dimensions in millimeter

**Figure 4-3** Dimensions in millimeter of intermediate (i) and end diaphragms (ii)

**Figure 4-4** PEVA45 with closed ends
4.2 Material properties

The material of the girder was varying between the different parts and the properties are presented in table 4-2. The steel was modelled with elastic-plastic properties in Abaqus with the shear-strain relation according to table 4-3.

Table 4-2 Material properties for parts in Bridge Y1504

<table>
<thead>
<tr>
<th>Steel quality</th>
<th>Flange</th>
<th>Web</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic yield strength [MPa]</td>
<td>460</td>
<td>420</td>
<td>355</td>
</tr>
<tr>
<td>Young’s modulus [GPa]</td>
<td>210</td>
<td>210</td>
<td>210</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Density [kN/m³]</td>
<td>77</td>
<td>77</td>
<td>77</td>
</tr>
</tbody>
</table>

Table 4-3 Shear strain relations for used steel

<table>
<thead>
<tr>
<th>Steel quality</th>
<th>Stress [MPa]</th>
<th>Strain [%]</th>
<th>Stress [MPa]</th>
<th>Strain [%]</th>
<th>Stress [MPa]</th>
<th>Strain [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>S355</td>
<td>355</td>
<td>0</td>
<td>420</td>
<td>0</td>
<td>460</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>370</td>
<td>0.05</td>
<td>440</td>
<td>0.05</td>
<td>470</td>
<td>0.05</td>
</tr>
<tr>
<td></td>
<td>380</td>
<td>0.1</td>
<td>450</td>
<td>0.1</td>
<td>475</td>
<td>0.1</td>
</tr>
<tr>
<td></td>
<td>385</td>
<td>0.15</td>
<td>460</td>
<td>0.15</td>
<td>479</td>
<td>0.15</td>
</tr>
<tr>
<td></td>
<td>390</td>
<td>0.2</td>
<td>-</td>
<td>-</td>
<td>480</td>
<td>0.2</td>
</tr>
</tbody>
</table>

4.3 Meshing and element types

The elements used for the entire model was S4R reduced integration shell elements, based on a thick shell theory. As depicted in figure 4-5 the element consists of four nodes with 6 active degrees of freedom per node, three translations and three rotations. The S4R element allows transverse shear deformation, and the transverse shear becomes very small as the shell thickness decreases.

Figure 4-5 Shell element S4R

The girder was meshed with Quad, Structured elements with the global approximate seed size of 50 mm. The diaphragms were meshed with a seed side size of 100 mm and the sheets with the approximate seed size of 20x60 mm, shown in figure 4-6.
4.4 Boundary conditions and loads
Since the FE-model only served the purpose to give the critical bending moment through the buckling analyses the bridge was subjected to a unit load of 1 N at the positions of the diaphragm that is shown in figure 4-1. In total nine unit loads at each girder.

The eigenvalue obtained from the buckling analyses then represents the critical load $P_{cr}$ that gave the critical bending moment $M_{cr}$ in the middle of the bridge. The derivation of the critical bending moment is shown in figure 4-7.
This gave a critical bending moment of

\[ M = 9P \cdot 32.5 - 2P(27.5 + 21.3 + 14.5 + 7.0 + 0.2) = 151.5 \cdot P \]

This critical bending moment obtained from the FE-analyses was compared to how the Y1504 Bridge was supposed to be loaded in reality. The first stage of concreting contained as mentioned before 66 m$^3$ of concrete spread over the 33 meters in the middle of the span of the bridge. This corresponds to a line load of 48 kN/m and resulted in a moment as depicted in figure 4-8.

![Figure 4-8 Acting bending moment from first stage of concreting](image)

This gave a bending moment from the first stage of concreting of

\[ M = 792 \cdot 32.5 - 48 \cdot \frac{16.5^2}{2} = 19.2 \text{ MNm} \]

When the first stage of concreting had hardened the stiffness of the bridge would increase significantly and the problem with lateral torsional buckling is not considered a problem anymore, therefore this was the bending moment that was considered the critical one.

The bridge was simply supported with two bearings at each support with a transversal spacing of 2200 mm. At support one the bearings were fixed and at support two the bearings were allowing movement in the longitudinal direction. These boundary conditions were modelled in two points at each ends of the bridge. At support one the points were pinned i.e. translation in both the x-, y- and z-direction is prevented. At support two the translation in the y- and z-direction was prevented. In addition to these conditions the ends of the top flanges were prevented from movement in the y- and z-direction at the middle of the flanges. The boundary conditions are all depicted in figure 4-9 below.
4.5 Course of action

In the FE-analysis linear eigenvalue buckling analyses were performed with different modifications on the original setup of the bridge. These analyses were performed on perfectly straight beams without any imperfections.

First of all, the girder system with end diaphragms but without metal decks or intermediate diaphragms were modelled and analysed through a buckling analysis and the critical bending moment was sought. This model was called the “base model” and after the meshing of this model together with all the boundary conditions and the internal connections was performed it was copied to the other analysed models to make sure that possible errors do not occur due to different meshing or a similar simple mistake.

After the analysis of the base model was performed, both the diaphragms and then the used sheet PEVA45 were connected to the girder through tie connections. The panels of PEVA45 sheets had been merged together to one big sheet since previously performed analyses showed that the connection between the adjacent sheets were not critical for the analysis of the entire system. The model of the bridge now had similar appearance as the Y1504 Bridge had in reality. Two buckling analyses were performed, one where the sheet was connected to the girder at one point per valley, i.e. at one side of the reinforcing seam, and one where the sheet was connected at two points, on each side of the seam. New critical bending moments were found together with the shape of the first buckling mode.

The next modification of the bridge was to attach end plates to the bridge as depicted in figure 4-10. These plates work as attachment plates for the sheets to be connected to the end of the bridge in complement to the longitudinal fasteners. The plates were created to be of the same material and thickness as the outermost parts of top flange and they had the dimensions 50x2800 mm. They were placed in between the adjacent top flanges with a tie connection. The connection between the sheet and the end plate was, just as the connection between the sheets and the girder, also performed by a tie. The final critical bending moment for the girder was found and they could all be compared to see the impact of both the sheets, diaphragms and the end fasteners.
To see if the alteration from the prescribed metal deck TRP45 to the used PEVA45 had any impact on the system a new model was made where again the base model was used, but now with the TRP45 sheet instead of the PEVA45. A buckling analysis was performed to also here find the critical bending moment. Just as for the PEVA45 sheet, the alternation between the number of fasteners were performed, but since the TRP45 sheet don’t have the bending in the middle of the valley, the alternation for this sheet was one fastener per valley or one fastener in every other valley.

In the model with the constructed deck, PEVA45, an alternation of the thickness of the deck was performed to see the impact on the critical bending moment. The thickness of the prescribed sheet TRP45 differed from the thickness of the used sheet, therefor this thickness was analysed along with a few other thicknesses presented in table 4-4.

Table 4-4 Different analysed thicknesses of PEVA45

<table>
<thead>
<tr>
<th>Analysed thicknesses</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.35 mm</td>
</tr>
<tr>
<td>0.50 mm</td>
</tr>
<tr>
<td>0.65 mm</td>
</tr>
<tr>
<td>0.72 mm (constructed thickness)</td>
</tr>
<tr>
<td>0.755 mm</td>
</tr>
<tr>
<td>0.85 mm</td>
</tr>
<tr>
<td>1.05 mm</td>
</tr>
</tbody>
</table>
4.6 Results from FEM

In table 4-5 below the critical bending moments, $M_{cr}$ determined through the FE-analyses for the six different models are presented. Associated mode shapes are depicted subsequent from figure 4-12 to 4-15. The obtained critical bending moment for the models subjected to lateral torsional buckling are reduced with the factor $\chi_{LT}$ according to section 2.1.

**Table 4-5 Critical bending moments for different performances of Y1504 Bridge**

<table>
<thead>
<tr>
<th>Analyzed model</th>
<th>$M_{cr}$</th>
<th>$\chi_{LT}$</th>
<th>$M_{cr}$</th>
<th>Mode shape acc. to figure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bare girders</td>
<td>13.2</td>
<td>2.4</td>
<td>4-12</td>
<td></td>
</tr>
<tr>
<td>Bare girders with intermediate diaphragms</td>
<td>16.7</td>
<td>3.5</td>
<td>4-13</td>
<td></td>
</tr>
<tr>
<td>Bridge with PEVA 45, one fastener per valley</td>
<td>58.2</td>
<td>27.4</td>
<td>4-14</td>
<td></td>
</tr>
<tr>
<td>Bridge with PEVA 45, two fasteners per valley</td>
<td>60.0</td>
<td>-</td>
<td>4-15</td>
<td></td>
</tr>
<tr>
<td>Bridge with PEVA 45, one fast. per valley, end attach.</td>
<td>58.2</td>
<td>27.4</td>
<td>4-14</td>
<td></td>
</tr>
<tr>
<td>Bridge with TRP 45, one fastener per valley</td>
<td>43.1</td>
<td>19.4</td>
<td>4-14</td>
<td></td>
</tr>
</tbody>
</table>

In figure 4-11 below, the critical bending moments for the six different models are displayed with the bending moments at failure, after the first stage of concreting and after all concrete is applied. As calculated in section 4.3, the bending moment from the first stage of concreting, $M_{cl}$ was 19.2 MNm. At the time of failure approximately a fourth of the concrete from the first stage was applied, corresponding to a bending moment, $M_{fail}$ of 4.9 MNm. If all of the concrete was applied at once, this would result in a bending moment of 25.4 MNm. This assumption though is on the safe side since the second stage of concreting would have been applied on the bridge where the first stage of concrete already would have hardened somehow and the composite effect could be credited for.

![Critical bending moments](image)

**Figure 4-11** Critical bending moments compared to acting bending moments at failure, from first stage of concreting and for total concrete loading
Figure 4-12 First buckling mode of base model, mode shape corresponding to lateral torsional buckling

Figure 4-13 First buckling mode of girders with diaphragms, mode shape corresponding to lateral torsional buckling
Figure 4-14 First buckling mode, girder with PEVA45, one fastener per valley, mode shape corresponding to lateral torsional buckling.

Figure 4-15 First buckling mode, girder with PEVA45, two fasteners per valley, mode shape corresponding to local buckling in webs between diaphragms.
Altering the thickness of the PEVA45 sheet gave critical bending moments as presented in table 4-7 and 4-8 together with the mode shapes where the buckling of end of plates are depicted in figure 4-17.

**Table 4-6 Critical bending moment of Y1504 Bridge with PEVA45 with altered thicknesses and one attachment per valley**

<table>
<thead>
<tr>
<th>Thickness</th>
<th>( M_{cr} )</th>
<th>( \chi_{LT} \cdot M_{cr} )</th>
<th>Mode shape</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.35 mm</td>
<td>24.1 MNm</td>
<td>5.4</td>
<td>Lateral torsional buckling</td>
</tr>
<tr>
<td>0.50 mm</td>
<td>33.6 MNm</td>
<td>11.8</td>
<td>Lateral torsional buckling</td>
</tr>
<tr>
<td>0.65 mm</td>
<td>46.9 MNm</td>
<td>21.1</td>
<td>Lateral torsional buckling</td>
</tr>
<tr>
<td>0.72 mm</td>
<td>58.2 MNm</td>
<td>27.4</td>
<td>Lateral torsional buckling</td>
</tr>
<tr>
<td>0.755 mm (as TRP45)</td>
<td>58.3 MNm</td>
<td>27.4</td>
<td>Lateral torsional buckling</td>
</tr>
<tr>
<td>0.85 mm</td>
<td>60.0 MNm</td>
<td>-</td>
<td>Local buckling between diaphragms</td>
</tr>
<tr>
<td>1.05 mm</td>
<td>60.0 MNm</td>
<td>-</td>
<td>Local buckling between diaphragms</td>
</tr>
</tbody>
</table>

**Table 4-7 Critical bending moment of Y1504 Bridge with PEVA45 with altered thicknesses and two attachments per valley**

<table>
<thead>
<tr>
<th>Thickness</th>
<th>( M_{cr} )</th>
<th>Mode shape</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25 mm</td>
<td>15.8 MNm</td>
<td>Buckling end of plates</td>
</tr>
<tr>
<td>0.35 mm</td>
<td>28.6/38.8 MNm</td>
<td>Buckling end of plates/ LTB</td>
</tr>
<tr>
<td>0.50 mm</td>
<td>55.8 MNm</td>
<td>Buckling end of plates</td>
</tr>
<tr>
<td>0.65 mm</td>
<td>60.0 MNm</td>
<td>Local buckling between diaphragms</td>
</tr>
<tr>
<td>0.72 mm</td>
<td>60.0 MNm</td>
<td>Local buckling between diaphragms</td>
</tr>
<tr>
<td>0.755 mm (as TRP45)</td>
<td>60.0 MNm</td>
<td>Local buckling between diaphragms</td>
</tr>
</tbody>
</table>

These critical bending moments were compared to the bending moment acting on the bridge after the first stage of concreting, \( M_{c1} \) and also the loading from the entire concrete weight, \( M_{tot} \) and presented in figure 4-16 below.
Figure 4-16 Critical bending moment for PEVA45 for different thicknesses and one respectively two attachments per valley

Figure 4-17 Buckling of metal deck at end of the bridge
5 Discussion

In this chapter the three different parts of the thesis, the failure reports, the theoretical analysis and the FE-analysis, are discussed, first separately and lastly combined together.

5.1 Failure reports

Studying the failure reports it is obvious that there is no standard way of dealing with the design of this kind of construction. Several well-known constructors tackles the problem with different approaches and what they agree on is that the bridge failed because the metal decks did not work as they were supposed to, because with an intact bridge, lateral torsional buckling would not have occurred, which was the failure mode in this case.

The calculation approaches differ between the constructors. Professor 1 calculates the shear stiffness of the deck that is then re-calculated to an equivalent thickness that can be used for checking the total rotational stiffness of the entire cross section. With this stiffness he can then calculate the forces acting on the screws connecting the deck and check if they are strong enough. There is never any check for neither the strength nor the stiffness of the metal deck itself, if it is sufficient enough. And in the controls made by Professor 2 and Professor 3, which are similar to each other only with somewhat different results, the metal deck controls consists of calculating the acting shear flow, checking the strength of the deck.

All of these controls that are performed is of course necessary when designing this kind of construction and it is impossible for the author to make any conclusions of her own about these, more than it is necessary to look at the theory produced after the failure regarding the stiffness requirements of the metal deck alone to work as lateral bracing, and see if there is anything missing in the constructors calculations. After all, professor 1 comes to the conclusion that the prescribed metal deck would, due to its higher stiffness, not have failed. And this assumption is made on the calculation of the stiffness of the entire cross section which in total then would have had sufficient torsional stiffness. This assumption is the only one that possibly could be incorrect when adding new theory to the failure reports.

5.2 Theoretical analysis

5.2.1 Stiffness of metal deck

According to calculations performed by the approach of Helwig the required stiffness for bracing the Y1504 Bridge is 4876 kN/m. The calculated stiffnesses for both the prescribed deck and the used deck according to SDI is presented in section 2 and repeated in table 5-1 below.

<table>
<thead>
<tr>
<th>Place of attachments</th>
<th>PEVA 45 (constructed)</th>
<th>TRP 45 (prescribed)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear stiffness [kN/m]</td>
<td>11526</td>
<td>1727</td>
</tr>
</tbody>
</table>

The calculations show that the prescribed sheet TRP45 has a remarkably higher stiffness than the used deck PEVA45. The TRP45 sheet have a stiffness of about four times from where I can see the required stiffness when attached in every valley. Attaching it in every other valley on the other hand decreases the stiffness a lot, down to about half of the required stiffness. This shows the importance of attaching the sheets in every valley where the ratio between the
stiffnesses for attachment in every versus every other valley is 5.5 respectively 6.2 for PEVA45 respectively TRP45.

It seems like even the used metal deck PEVA45 have sufficient stiffness to work as lateral bracing for the Y1504 Bridge, at least when attaching the sheet in every valley, as performed. The question is how reliable the results are for the PEVA45. The dimensions of the TRP45 is considered applicable for the SDI calculations, but the result for the PEVA45 sheet could be misleading due to the reinforcing seam in the profile bottom which is not taken in account for in the calculations, and this seam will obviously affect the stiffness of the sheet. It is probably not correct to say that the sheet is attached in each valley since it is only attached on one side of the center seam, and the attachment therefor become eccentric. Due to this insecurity it is with this basis hard to make a statement whether the stiffness of the used sheet was sufficient enough. Though if the sheet would have been attached twice, on each side of the seam, one might assume that at least the calculated stiffness of the sheet with attachments in every valley would be reliable and the prerequisite stiffness would have been obtained.

In the calculations according to SDI, the dimensions of the sheet shown in figure 5-1 together with the number of screws, partly attaching the sheets to each other and partly attaching the sheets to the top flanges of the girder, and the spacing of the screws is affecting the outcome.

![Figure 5-1 Indications of sheet dimensions](image)

To obtain a stiffer sheet the thickness and the number of screws attaching the sheets to each other should obviously be bigger. It is also advantageous to decrease the length of the tilted side, w, and the width of the top and bottom horizontal surface, f and e, and the height of the sheet, h. The bottom line here is that a sheet with “tighter” dimensions will give higher stiffness. Comparing the different dimensions between PEVA 45 and TRP 45 shows that in all of the varying parameters, the TRP 45 sheet has the more beneficial value for obtaining as high stiffness as possible.

Even though one might assume that if the PEVA45 sheet was attached on each side of the reinforcing seam it would have obtained sufficient stiffness, the prescribed TRP45 sheet, with more suitable dimension for this approach, would be a safer choice and the author is entitled to say that with attachments in every valley this sheet would obtain sufficient stiffness to work as bracing for the Y1504 Bridge.

If this control was made in the failure reports, they would have found out that the used metal deck is doubtful to obtain sufficient shear stiffness to work as lateral torsional bracing, at least for the performed attachments.

### 5.2.2 Forces on screws

When calculating the strength of the metal deck by checking the forces acting on the screws the stiffness of the deck is an important parameter and when using the approach according to Helwig et al. (2008), the stiffness is predicted to be four times the ideal stiffness. Since it is
hard to predict the stiffness of the used sheet, it is impossible to say that the results of the calculated forces on the screws are corresponding to reality. If the metal deck is less stiff than four times the ideal stiffness the forces on the critical screw will be even higher than calculated. Therefore the results should be considered with cautiousness.

The calculated forces acting on the screws connecting the sheet to the top flanges, see table 5-1, are significantly higher than what any normal sized nail or screw could be able to resist. Assuming screws with the diameter of 6.3 mm, as prescribed in the case of Y1504 Bridge, according to Eurocode SS-EN 1993-1-8 the strength of the screw then have to be 1444 MPa to be able to resist the force of 21.0 kN acting on the TRP45 sheet. See equation 5.1.

\[ F_{v,Rd} = \frac{\alpha f_{ub} A}{\gamma M_2} \]  

(5.1)

where

- \( \alpha_p \) is a reducer due to strength class, 0.6 for class 4.6 and 8.8, 0.5 for class 10.9
- \( f_{ub} \) is the ultimate limit strength of the screw [MPa]
- \( A \) is the area of the screw [mm\(^2\)]
- \( \gamma M_2 \) is the partial coefficient = 1.2 for fasteners according to SS-EN 1993-1-8, 2.2

\[ f_{ub} = \frac{F_{v,Rd} \gamma M_2}{\alpha_p A} = \frac{21.0}{0.6} \frac{1.2}{31.17} = 1347 \text{ MPa} \]

This required strength can be compared to the ultimate limit strength of different screw classes according to SS-EN 1993-1-8, 3.1.1 which are 400 MPa for class 4.6, 800 MPa for class 8.8 and 1000 MPa for class 10.9. Forces of this size acting on a screw with the dimension as prescribed is too large to resist even for the strongest screw.

To get an idea of the size and strength of a screws required to resist the forces, increasing the diameter to 7.5 mm, using screws with a ultimate limit strength of 1000 MPa, each screw can resist a force of 22.1 kN which would be sufficient for the TRP45 sheet with one fastener in every valley.

According to these calculations it is quite obvious that even when using the prescribed sheet TRP45, with sufficient stiffness for working as lateral bracing, the forces acting on the screws are too large to resist. The assumption of professor 1 that the bridge would not have failed if the performance was done as prescribed is not correct according to the author, who’s making the statement that the screws attaching the metal deck to the top flanges would have failed, on the basis on the approach by Helwig.

It is also of interest to find the magnitude of the strength of the edge of the screw holes in the metal deck, according to equation 5.2.

\[ F_{b,Rd} = \frac{k_1 \alpha_b f_u d t}{\gamma M_2} \]  

(5.2)

where

- \( k_1 \) = 2.5 for screws at edge
- \( \alpha_b \) = approximately 0.8
- \( f_u \) is the ultimate limit strength of the metal deck [MPa]
\[ F_{h,Rd} = \frac{2.5 \cdot 0.8 \cdot 360 \cdot 7.5 \cdot 0.755}{1.2} = 3.4 \text{ kN} \]

It is obvious that the thin plate in this case is the critical part of the strength of the deck. To achieve a strength of 21 kN, which is necessary for the TRP45 sheet, together with the conditions of a screw described above, the thickness of the deck needs to be approximately 5 mm, i.e. more than six times as thick as the prescribed sheet.

5.3 FE-analysis

The result from the first analysis of only the base model with the diaphragms gives an indication that the modelling is performed close to reality. Both due to the mode shape that shows lateral torsional buckling, which would be the predicted mode, and also due to the reasonable value of the critical bending moment of 3.5 MNm with the diaphragms. If assuming that the Y1504 Bridge in reality did not obtained any rotational restraint from the sheets, as the investigators proclaimed after their failure evaluations, the bridge would have acted as an open section which failed when subjected to the bending moment of 4.9 MNm.

When the used metal deck PEVA45 is applied the beam is still failing due to global lateral torsional buckling, but now at a significantly higher value of the critical bending moment. The critical bending moment of 27.4 MNm is almost ten times higher than for the girders without the metal decks which shows their great strengthening effect. The model is now modelled as it was performed in reality and the critical moment is greater than both the resulting bending moment from the first stage of concreting, and also higher than the bending moment from the entire concreting load. This shows that if the used metal decks was performed with intact attachments, one attachment per valley would be sufficient, the bridge would not have failed due to concreting the deck.

To analyse the impact of the number of screws attaching the PEVA45 deck to the top flanges, one fastener per profile bottom is added, on the other side of the reinforcing seam. The big difference between the results lays within the shape of the buckling mode. The first mode shape for model with only one fastener per profile bottom is lateral torsional buckling and the model with two fasteners seems to be subjected to local buckling between the diaphragms. So it seems like the difference of adding the extra fastener to the sheet actually makes the lateral restraints stronger and thereby eliminating the risk of global lateral torsional buckling.

Looking at the results of the shear stiffness calculations according to SDI the stiffness is increasing a lot when adding the extra fastener, passing the limit of working as bracing against lateral torsional buckling according to requirements by Eglimez et al. So the impact of the extra fastener is big and although in this case it is not decisive for avoiding lateral torsional buckling since one attachment per valley is sufficient.

The attempt of attaching the metal decks to the end of the bridge through end plates shows that this attachment simply does not have any effect on the system, neither when analysing the PEVA45 sheet nor the TRP45 sheet. The critical bending moment does not change when the metal decks are attached. To troubleshoot the modelling of the end plates, the thickness of the sheet is decreased to the extreme low value 0.002 mm and two jobs are performed, both with and without the end plate. This results in a higher critical bending moment for the model with end plates, which gives the indication that the modelling of the end plates are performed.
correctly. The fact that the end plates does not affect the critical bending moment in this case could be because the stiffness of the system with the sheets alone attached to the adjacent girders at the ends of the beam are sufficient and the end attachments are not necessary for preventing failure.

The 30% decrease of the critical bending moment when changing the metal deck to the prescribed TRP45 sheet, together with same mode shape i.e. global lateral torsional buckling is an unexpected result. It says that the TRP45 sheet with attachment in every profile bottom for some reason is subjected to lateral torsional buckling at a lower critical bending moment than for the PEVA45 sheet with only one attachment per valley. Since the attachments are performed identically in the different models as ties it seems like the TRP45 metal deck is providing a less stiff system, compared to the PEVA45 sheet. Despite this decrease of resistance the system would still be able to resist the load from the first stage of concreting, albeit just on the verge since the loading is just 0.2 MNm above the critical bending moment. It should be highlighted though that in these calculations made by the author, no safety factors according to Eurocode are included and since the margin is so small, the metal deck would not have passed the design criteria even for the first stage of concreting.

When studying the impact of the thickness of the metal deck, the PEVA45 is analysed with both one and two attachments per valley. With one attachment, a thickness of 0.65 mm would have been sufficient to resist the loading from the first stage of concreting, and for the system with two attachments per valley, corresponding thickness would have been 0.35 mm.

With one attachment per profile bottom the critical bending moment is increasing somewhat linear until the mode shape changes from global buckling to local buckling in between diaphragms, i.e. when the thickness is 0.85 mm resulting in the critical bending moment of 60.0 MNm. The thickness obviously has a great impact of the resistance. Looking at the thicknesses of 0.5 mm and 0.65 mm, a difference of 0.15 mm, the critical bending moment almost doubles and increases above the bending moment from the first stage of concreting. Although it is not until the thickness of the deck is 0.85 mm that the metal deck reaches a stiffness where the lateral torsional buckling is no longer critical, but the webs become to slender to resist a moment of that size and the failure mode changes to local buckling instead. Increasing the thickness above 0.85 mm is therefore not necessary for receiving a stronger system, provided that the attachments are working as wanted.

For the same metal deck attached with two attachments instead, the critical bending moment is obviously higher and the failure mode for the thinner decks are buckling of the end of the metal decks. At a thickness of 0.65 mm the failure mode changes and instead becomes local buckling between the diaphragms. The greatest shear forces appear at the ends of the bridge, this is why end fasteners might have been appropriate for a cross section with a thinner metal decks, to prevent local buckling here. But still, the second mode shape for the 0.35 mm metal deck with two fasteners is lateral torsional buckling at the critical bending moment of 38.8 MNm, which is reduced to $\chi_{LT} \cdot M_{cr} = 15.6$ MNm. So even if the buckling of the ends are prevented, global lateral torsional buckling will still be an issue for this thin metal deck and failure would have occurred during the first stage of concreting.

5.4 Further research

Further research on this subject could be performed by developing the FE-analysis, introducing initial imperfections in the model for more accurate values of the critical bending
moments. These FE-results could then be compared to results from ongoing studies where the Y1504 Bridge is modelled and the metal decks are tested in reality.
6 Conclusions
Finally, looking at the performed Y1504 Bridge, the used metal deck PEVA45 should have been able to work as lateral restraint for the first stage of concreting as it was performed with one attachment per valley. Even though calculations according to SDI shows that the prescribed metal deck TRP 45 had sufficient stiffness to also restrain this loading, the FE-results indicates the opposite, that the bridge would have failed with the prescribed metal deck. Since the dimensions of the cross sections of the bridge are almost identical for the two different metal decks, the stiffness of these have to be the decisive parameter. And since the two different models in the FE-analyses are made in a way so that they are exactly the same in every aspect except for the decks the author simply does not know how to trouble shoot this result in the FE-analysis.

In future designing of metal decks working as lateral restraint against buckling, in addition to checking the shear flow resistance and the rotational stiffness of the system, it is important to have a metal deck with sufficient shear stiffness for the particular cross section for it to be restrained. But this requirement of a stiffness of four times the ideal stiffness alone does not seem to be the critical part. Even though great stiffness is obtained in the decks, the attachments between the metal decks and the top flanges is of course critical, both adapting the theory performed by Helwig and looking at the forces acting on the screw, but even more important, using these forces and comparing it to the resistance of the edge of the screw holes for the thin deck.

Using the metal deck PEVA45 the thickness of only 0.65 mm would according to the FE-analysis be sufficient to resist the first stage of concreting, but using a thin plate as this, attaching it with screws, shear forces will be concentrated by the attachment points and both she screws and the plate will be exposed to excessive forces, which would have led to failure. Looking at the simple calculations in section 5.2.2 it is shown that the metal deck in this case needs to have a thickness of about 5 mm to resist failure of the edge of the screw holes. To avoid these local concentrations of force, welding the metal decks to the top flange could have been an alternative. Thereby the thin metal deck of PEVA 45 could still have been utilized without failure due to lateral torsional buckling.
7 References


Appendix I
Failure reports, translated to English

This appendix contains of the following nine documents, translated from Swedish to English:

1. Regarding failure of Bridge Y1504 over Gide älv, 2002-09-06 technical consultant 1
2. Calculations with performance according to drawings and according to actual performance at failure, 02-11-05 Professor 1
3. The report “Evaluation of reason of failure of Y1504 Bridge in Kärrsjö”, 2003-04-10, Professor 2
4. Failure of Y1504 Bridge over Gideälv in Kärrsjö: Comments to report made by Professor 2, technical consultant firm 2 AB, 2003-08-15, Professor 1
5. Damage investigation for Y1504 Bridge over Gideälv in Kärrsjön, 2003-12-16, technical consultant 1
6. Comments to the opinions of Professor 1 about the report “Evaluation of reason of failure of Y1504 Bridge in Kärrsjö” by Professor 2, 2004-04-28
7. Failure of the Y1504 Bridge over Gideälv in Kärrsjö, 2004-08-10, Professor 3
8. Technical consultant firm 1 have through technical consultant 2 got the mission for undersigned to give an announcement about the failure of the Y1504 Bridge over Gide älv. 2004-10-23, technical consultant 3
9. Would the bridge have failed if it was performed according to the drawings? 2005-0202 Professor 1
1. Regarding failure of Bridge Y1504 over Gide älv
2002-09-06 technical consultant 1

Failure of Bridge Y1504 over Gide älv

After the failure of bridge Y1504, involved contractor, planner and steel supplier interacted to identify the preconditions, the failure and its reasons. At the request from construction company 1, the undersigned has been hired to be a part of this failure group mentioned above and thereby evaluate facts, actions and results. This rapport concludes the work and conclusions as undersigned has been commissioned to perform.

The work is limited to on-site visits, interviews, meetings with the failure group and the examining of papers from planning to authorized constructions documents.

The conclusions and assumptions presented in this rapport are based on above mentioned sources where nothing else is mentioned and is only related to what is presented and what has been served to undersigned when establishing his report.

Generally

The bridge is a simply supported beam bridge with a span of 65 meters and a free bridge width of 7 meter. The beam is a conventional composite bridge. The load taking construction is a box section where the bottom part is a steel through where a concrete deck is casted on. The construction works in a way that the steel should have the ability to carry its own self-weight, concreting loads and loads from launching the beam. Other loads such as traffic load is carried by the steel and the concrete together. The conduction of the bridge is performed in three different stages, assembling the steel, launching and casting the concrete deck.

Description of the failure

On Wednesday, June 12, the casting of the deck is commenced. Since the casting was performed at summertime during warm weather the casting was to be performed in the evening. This to avoid exposing the concrete for a lot of heat and great sun irradiation that augment the risk of plastic shrinking cracks plus that is makes the concrete harden to fast. The day of casting was however cloudy and the sun did not emerge until later that evening, approximately at the same time the casting started. The first sequence of concreting of about 66 m$^3$ was 33 meters long and covers approximately half of the lengths of the bridge. Concrete was casted in the middle of the bridge with a pump, creating two concrete fronts, one against each support. To avoid unnecessary large eccentric loading the concrete was casted in a center line of the bridge and was performed evenly against the end beams. The casting front at this kind of performance was shaped as a V. According to measurements there was already a height difference between the both top flanges of approximately 15 mm in the middle of the bridge span when the concreting started. Even the bottom of the box was tilting to the side, shown by water runoff. The supervisor of the bridge work was controlling a center point of the bridge during concreting. This measure point was located next to the railing on the cantilever downstream the bridge and the measure station was located at support 2. After casting the first loading of concrete, 8 m$^3$, the cantilever has lowered 17 mm. After casting the second loading, the deformation had almost completely reverted. The supervisor then decided to see if the measure point had been moved, but it was intact when controlled. Out on the
bridge the supervisor noted that the elevation difference of 8 mm between the mould for the cantilever and the top flange was gone and that the close casting front was rather lower. The supervisor was to follow up this during the continued casting. During the time people are working on the bridge by vibrating the concrete and at the same time there is two men inside the box to flush away sludge water penetrated the box during concreting. At this point, with approximately 17 m³ of concrete applied (about 25%) the bridge suddenly tilt. Reportedly the middle part of the bridge turns 90° immediately without any warning. Five of the six people on the bridge fell down in the river and swam ashore. None of the involved was significantly hurt. The applied concrete ran off the bridge together with some of the reinforcement. When the concrete had ran off the bridge is turning back somewhat and the mould is loosening from one of the top flanges in the center section of the bridge. People inside the box where able to escape. They notice that all of the struts supporting the mould, from the middle of the bridge to support 1, are intact since they need to step over them to get out of the box. The entire laps from the time when the box tilt until the bridge reaches a new equilibrium takes place in about 5-10 seconds. The bridge now has vaulted greatly at the ends and stock pallets at support 1 has been pushed backwards and failed.

Work conducted onsite

Steel assembling and work ashore

The steel and the steel mounting was delivered by Dem Verk Mekaniska AB in Umeå. The steel box was built in a workshop in three parts, 22.5-20.0-22.5 meters long, and was transported to the work site already painted. At the location of the bridge a plane place was prepared behind support 2 where the bridge parts were propped temporary before assembling. Dem Verk Mekaniska AB was responsible for drawings, assembly welding and painting. After assembling onshore the straightness and the elevation of the bridge was within given tolerance. While the steel box still was onshore, the permanent form made by trapezoidal sheets was applied. According to drawings the permanent form was performed by Plannja trp 45 \( t_{\text{nom}}=0.85 \), computational thickness 0.755 mm, attached with screws \( \varnothing 4.5 \) mm in each profile bottom and attached to each other in the splice with screws \( \varnothing 6.3 \) mm c/c 150 mm. In the procurement of the plate material, construction company 1 sent a request to the plate supplier with the construction documents as basis. Construction company 1 procured sheets of the kind PEVA 45 from “svensk Bygglåt AB” to use as permanent form work according to the suppliers recommendations suitable for requested purpose.

The chosen sheet profile differs from Plannja trp 45 as follows. PEVA 45 has an extra bending in the profile bottom, it’s a bit thinner, \( t_{\text{nom}}=0.8 \) mm which gives a computational thickness of 0.72 mm. When buying the form work, the buyer ordered the, from the supplier recommended, joint screw instead of the screw prescribed on the drawing. The opinion of undersigned is of that the designer should have been consulted before deviation from the construction documents were made. Even in cases where changes may have a small effect, details could still have functions that are not obviously primary.

The profile was assembled in the joint with screws \( \varnothing 4.8 \), c/c 250-300 mm instead of \( \varnothing 6.3 \), c/c 150. To avoid concrete leakage between the sheets, adapted sealing tapes were placed in the joints between them. Otherwise the form work was assembled as prescribed with a support
in the middle line of the bridge consisting of a wooden stud 45x145 which in turn was supporting on struts. The struts were secured at the tops to not fall down.

The permanent form work was assembled along the entire bridge except for the last two sheets, adjacent support 1, and they were attached with screws along the bridge. No connections were made to the end beams or end diaphragms. Remaining form for the cantilevers was assembled on both of the sides of the bridge according to instructions from the form supplier Hünnebeck Sverige AB.

Before the launching approximately 20 m of the reinforcement was assembled adjacent support 2.

Launching the bridge beam.

Dem Verk Mekaniska AB were responsible for the launching of the assembled box. The launching was made along with the bridge, starting at support 2. Before the launching a launching nose, constituting of a 33 m long lattice, weighing 13 tons, was attached at the protruding part of the bridge. Part of the reinforcement was also placed at the end of the bridge, adjacent support 2 that was supposed to work as counterweight when the bridge was launched. The launching was carried out as planned without any problems. When all of the bridge was launched, the launching nose was removed and the bearings were adjusted and cast in place.

Remaining work for the casting of the deck.

The remaining reinforcement was assembled in the deck. Usual preparations was made. The casting of the deck is presented above under the headline “Description of the failure”.

Calculations and theories about the behavior of the steel beam during construction

Theories about rotation, diaphragm action and instability/lateral torsional buckling

To be able to familiarize with the function of a beam and its mode of action some general structural mechanic relations and definitions are presented below. These are primary to understand the underlying mechanisms causing the failure.

The rotation of the cross section of the beam is occurring along with the beam axis. Depending on the shape of the cross section different deformations of the beam occur. For a massive cross section or a closed, thin-walled, cross section a clean rotation of the cross section occurs. The shear flow due to the rotation retains a closed shape. If the cross section instead is a thin-walled open cross section, for example a U-profile, a significant camber at the ends of the beam occurs in addition to the rotation. A camber means that the cross section does not remain planar but deforms along the length of the beam. All cross sections, except for circular symmetrical, cambers more or less. For a closed, thin-walled cross section the tendency to camber is increasing since the thickness of one of the sides is decreasing relatively to the other sides.

If you are able to prevent cambering of a beam when it is loaded with rotation, the rotation capacity is increasing significantly. This prevention can be done in between supports of
continuous beams in several fields. For a simply supported beam this is hard to achieve without great contribution by attaching the ends of the beam along with the beam.

By diaphragm action it means the ability for a planar load taking structure to transfer loads in its own plane. The plate then comprises of the surface load bearing construction, the studs and the field boundary beams. For diaphragm action to occur it is important that the sheet is attached about the edges so that shear equilibrium can be obtained in plane.

The instability phenomena of lateral torsional buckling is characterized by that the flange in pressure, when loaded with moment, is buckling sideways whereby the cross section will rotate since the bottom flange is in tension. Lateral torsional buckling can be controlled according to StBK-K2 or Handboken Bygg part K. These calculations are regarding the rotation and the camber stiffness of the beam. Parameters affecting the instability are for instance the position of the loading point, the camber of the beam and external rotational moment. A rough way of studying lateral torsional buckling is to consider the top flange as a strut in pressure in the weak direction and then check it for buckling. This is on the safe side since the rotational capacity of the beam then hasn’t been utilized.

Review of design calculations and drawing

The design of the bridge was performed by technical consultant firm 1 Luleå, which has designed several bridges of the same construction type. Many of these has during the stage of construction had permanent form works made out of trapezoidal sheets, working as stabilizing elements during the construction stage. The trapezoidal sheet is meant to transfer loads through diaphragm action. The calculations made for the bridge is performed in the calculation program BALKBRO (cross section calculations) and CONTRAM (stress calculations). The calculations seems to be satisfying and has been reviewed briefly by undersigned since these especially concerns the permanent stage of the bridge.

Documented check in the stage of construction is performed by the planner, presented in chapter 4.9, containing 2 pages. Two moments are missing in this check. The first moment includes the design of the overlap joint. This has been designed in a way that the plate should be able to resist a force from one of the top flanges of the steel box, equivalent to 1.5 % of the pressure in the flange from the same. This force is according to the planner transferred through shearing in the sheet to the diaphragm of the steel box. Further on, it seems like,
according to the planners’ way of looking at it, as if the joint is under dimensioned. The number of screws the planner said was necessary for every meter beam has been spread over the entire length of the joint, which is 2.45 m, i.e. not even half of the capacity is achieved. Undersigned has not been able to see how this force is transferred to the end diaphragms through this documentation.

It does not show in the documentation presented by the planner how the stability in total of the bridge has been assessed, how eccentric loading and external rotational moments is taken in the construction stage, how the shear force equilibrium is reached in the sheets and how the attachments (nails and screws in joints) are affected from the above mentioned things.

The working instructions mentioned on drawing -20 is indicated by the planner to comprise of the text in section D20 on the same drawing. Neither the drawing nor other presented documents tells the purpose of the permanent form work for the different construction stages. The opinion of the undersigned is that it should be presented in the construction documents when temporary installations has a primary impact of the total stability of the construction. It is also appropriate to prescribe additional checks to secure the important function that ingoing parts of the stabilizing construction has.

The planner has presented a check calculation of the suggested performance to KTH, dated and sent 2002-06-26. This calculation consist of a lateral torsional buckling calculation where the permanent form work is a part of the beam cross section.

Calculations has been performed with the base from Handboken Bygg part K chapter 24 and StBK-K2 “Kommentarer till stålbyggnadsnorm 70, Knäckning Vippning Buckling”.

Calculations are performed for form works similar to the one the planner specified on the drawing. With this trapezoidal sheet recalculated to an equivalent planar sheet, an increased rotational stiffness is obtained in this calculation of about a tenfold compared to an open section. In this calculation the trapezoidal sheet has been recalculated to a planar sheet with equivalent thickness with respect to deformations properties along with the length of the bridge. The cross section constants has then been calculated for a closed cross section, both rotational and camber stiffness has been calculated. The calculated trapezoidal sheet has a height of 45 mm $t_{nom}=0.85$ (computational thickness 0.788 mm) and a profile width of 150 mm which is somewhat shorter than for Plannja 45. Calculated sheet gets according to calculations an equivalent thickness of 0.04 m when it is modelled as a planar sheet and during the circumstances that it is attached all the way around. The attachment to the steel beam is performed by screws I every profile bottom and along the end beams. The beam is predicted to be straight in the vertical plane when it is subjected to lateral torsional buckling.

The conditions mentioned in the above calculations is deviating from what actually was the conditions for the Y1504 Bridge. The calculations is predicting that the sheet was attached all the way around, i.e. also at the end beams which was not the case in reality. The analysis is made for a sheet with a smaller width and a thicker plate core than the one prescribed on the drawing which gives a computational stiffer sheet. The impact of external rotational moments and the impact from the camber has not been treated. The analysis is performed for a concreting sequence bigger than what was intended to cast. Due to this, it is the opinion of the undersigned that the above presented results, safety against lateral torsional buckling 1.2,
which approximately correspond to the requirements for safety class 3 according to the partial coefficient method, is afflicted with a non-quantified insecurity.

Possible reasons of failure

A possible passage of events during the lateral torsional buckling is that the form work has not been able to transfer she shear forces along with the beam through the screws. A rotation has occurred and along with this, failure or deformations about the screw attachments along the top flange of the beam. This contributes to the decrease of rotational stiffness and the total failure crystallizes as lateral torsional buckling for an open, thin-walled section. The displacement of the sheet and the damage with failure in the edge of the holes of the screws plus the great camber in the ends of the beam speaks for this scenario. Since the beam is subjected to lateral torsional buckling for a relatively low level of loading (approximately half of the total moment loading from concreting) it seems like the primary reason of failure is that the permanent form work has not been able to work as a stabilizing element.

A number of factors could have worked together in a way that the permanent form work not was able to work as a stabilizing element. A number of factors that according to undersigned has reduced the possibility for the form work to act as a stabilizing element is presented below:

- The working instructions has not mediated the vital function of the permanent form work as a stabilizing element during concreting. Information is missing about the fact that the sheet is primarily stabilizing, how much of the sheeting that should be assembled before launching and casting of concrete for the different sequences and how the sheet should be attached to cross beams and end diaphragms.
- A different kind of sheet with somewhat different properties and attachments has been used.
- The sealing tapes that was mounted along the ends of the sheet could have had a negative impact on the screws by introducing a small bending moment to this for horizontal loading.

Finally, with given basis from the planner, undersigned cannot determine whether the failure would have occurred or not if the prescribed sheet had been used since it is not verified in this basis how and if the prescribe form work should work. The used sheet, the weaker over-lap joints, the sealing tapes along the edges of the sheet are possible factors that probably affected the construction negative.

The ongoing completion of calculations from the constructor should show if he prescribed performance is computationally safe.
To perform a calculation taking all deviations in account is rarely possible. For the calculations of lateral torsional buckling below, used plate, screws and overlap screws lengthways are taken into account. Later the missing of attachments at the ends of the beam is also taken into account.

The biggest shear deformation of a sheet made by trapezoidal profiled plate, are the deformations due to the fact that the sheet is only attached in the profile valleys. The special groove in the used PEVA 45 contributes with extra deformations caused by the height of the groove. Comparing FEM-calculations shows that the groove contributes with an augment of approximately 80% for the shear deformations. See figure below. If screws where attached in each plane part of the valley these deformations would have been a lot smaller.

Even though the mentioned deviations are taken in account for, the calculated safety against lateral torsional buckling for concrete loading at the time of failure is still that big that only lateral torsional buckling cannot explain the failure alone. If you on the other hand anticipate that the sheet lacks shear stiffness, the safety number will be less than 1.0. So why did the sheet not function as it should? A possible explanation is that the attachments became overloaded due to rotation due to the fact that the uneven concrete over the width and that the bridge had an initial rotation before the concreting started. A calculation is made for an estimated eccentricity for the applied concrete at 0.2 m.

Measurements made before the concreting showed that the level of the top flanges differed 30 mm in the middle of the bridge. Since the shear center is below the bridge this contributed with a side bending of the top flanges that in turn gives a rotational moment added to the moment from the eccentric concrete load.

The rotation of the bridge leads to a safety lower than 1.0 against the phenomena where the steel fails at the border of the screw holes, by the screws in the side over laps. The sheets are not attached to the end diaphragms of the bridge. The outer screws therefore become overloaded then they are taking the shear flow from the sheet, but are not able to transfer it to the diaphragms. The top flanges can locally transfer big shear forces. The outer screws gets additional loading and deform since the border of the screw holes fail. The screw attachments can deform significantly without decreasing load. The extra loading on the outer screws can be relocated to the closest screws without any extra deformation. The screws carry load through pressure on the border of the screw hole and is therefore not dependent of friction or any sealing tapes.

So there is only a few screws that is overloaded at the end of the bridge. All of the side overlap screws on the other hand are overloaded and when the steel fails at the boarder of the holes, the entire sheet loses most of its stiffness and the box cross section becomes and works as an open section, and that's when lateral torsional buckling occurs.

Of course it is hard to determine after the failure how big the eccentricity was for the concrete. But you can see that some parts of the mould does not have any traces of concrete,
which shows that the concrete was applied with an eccentricity. Would the same failure have occurred if the sheets and attachments were performed according to the drawings? Calculations show that the force on the screws and nails becomes smaller when the rotational stiffness of the bridge becomes bigger. When their capacity also becomes bigger the safety against failure is absolutely bigger than 1.0 for the first sequence of concreting if the bridge was performed according to prescribed construction documents.

As a summary, the failure primarily was due to using less and weaker screws than prescribed. This led to loss in shear stiffness for the sheet. The changing of sheet also contributes to this due to the high groove and an eccentric attachment in the profile valley which leads to high deformations at the ends of the sheet. See figure below. Applying the concrete eccentric and also some deviation from straightness in the bridge led to a rotational moment which in turn led to overloading of the side overlap screws. When the borders of the screw holes fail, the stiffness of the sheet decreases and the sheet stops working as a plate and lateral torsional buckling occurs as for an open box section. For an open box cross section the shear center is located a few meters below the bottom of the box and a box like that becomes very unstable. If the screw were attached according to drawings, safety against the failure would have been enough.

Calculations

Calculations are presented in detail below. The construction is very complicated if all the factors should be considered. Here the calculations are divided in a way that they hopefully can be checked after each step.

- Shear stiffness for the sheet. An effective stiffness that gives the same shear deformations as the plate attached to the profile valleys at the ends and with the actual side overlap screws is calculated. Notice that the deformations at the ends of the sheet are dominating deformation components.
- Cross section measurements for the bridge with a top flange corresponding to a sheet with the effective sheet thickness. This gives a thickness much thinner than in reality.
- Lateral torsional buckling is calculated with these measurements. To comparison safety for lateral torsional buckling is also calculated for an open cross section.
- Forces in screws when rotation due to eccentric loading is calculated.

Calculations are partly made for performance according to the drawings and partly for performance according to reality.
Performance according to drawings

Shear stiffness for a plate

<table>
<thead>
<tr>
<th>Study a square plate</th>
<th>a=3400 mm</th>
<th>b=a</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plate thickness and profile split</td>
<td>t=0.778 mm</td>
<td>b_d=150 mm</td>
</tr>
<tr>
<td>Profile top, bottom and height</td>
<td>b_o=30 mm</td>
<td>b_u=60 mm</td>
</tr>
<tr>
<td>Flange slope</td>
<td>b_w=0.5(b_d-b_o-b_u)</td>
<td>b_w=30 mm</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>E=210000 MPa</td>
<td></td>
</tr>
<tr>
<td>Plannja Kombi deck 45 nominal thickness</td>
<td>0.85 mm</td>
<td></td>
</tr>
</tbody>
</table>

Calculations according to chapter K24 in Handboken Bygg, Band K. Liber Förlag Stockholm 1985.

Formulas according to example in Professor 1 T., Stabilisering genom skivverkan. SBI Publ 169, 2000

Plate directly on beams

Magnitude for formula in $c_{1,1}$

$$\alpha_1 = 1 \quad \alpha_4 = 1 \quad pr = \left(\frac{b}{a}\right)^2$$

$$a=3.4 \text{ m} \quad b=3.4 \text{ m} \quad pr=1 \quad \frac{b_o}{b_d} = 0.2 \quad \frac{h_w}{b_d} = 0.3 \quad \theta = \arctan\left(\frac{h_w}{b_w}\right) \cdot \frac{180}{\pi} \quad \theta = 33.69$$
Deformation at ends of sheet

\[ K_1 = 0.157 \]
\[ c_{1.1} = \frac{a \cdot b \cdot \alpha_1 \cdot \alpha_4 \cdot K_1}{E \cdot t \cdot b^2} \cdot pr \cdot 2 = 0.227 \text{ mm/kN} \]

Magnitude for formula in \( c_{1.2} \)

\[ \alpha_2 = 1 \quad v = 0.3 \]

Shear deformations in sheet

\[ c_{1.2} = \frac{2 \cdot a \cdot \alpha_2 \cdot (1 + v) \cdot \left( \frac{1}{b} \cdot \frac{h_w}{b_d} \right)}{E \cdot t \cdot b} \cdot pr = 0.025 \text{ mm/kN} \]

Magnitude for formula in \( c_{2.1} \)

One screw in each profile valley

\[ d_{pin} = 4.5 \text{ mm} \quad k_2 = 5 \quad \alpha_3 = 1 \quad p = b_d \]

Displacement in attachment between sheet and end beams

\[ s_p = \frac{1 \text{ mm}^2}{k_2 \cdot d_{pin} \cdot \frac{1000}{\text{N}}} = 0.050 \text{ mm/kN} \]

Magnitude for formula in \( c_{2.1} \)

\[ c_{2.1} = \frac{2 \cdot a \cdot s_p \cdot p \cdot \alpha_3}{b^2} \cdot pr = 0.0044 \text{ mm/kN} \]

StBK-N5 33:152

Distance screws in side overlap

\[ c_n = 150 \text{ mm} \quad n_s = \frac{b_d}{c_n} \quad n_z = 23 \quad \beta_1 = 1 \quad n_p = 2 \]

Displacement in attachment at overlap

\[ c_{2.2} = \frac{2 \cdot s_p \cdot s_p \cdot (n_{sh} - 1)}{2 \cdot n_s \cdot s_p + \beta_1 \cdot n_p \cdot s_s} \cdot pr = 0.02112 \text{ mm/kN} \]

Magnitude for formula in \( c_{2.3} \)

\[ n = 1 \quad s_{sc} = 0 \text{ mm kN}^{-1} \quad n_{sc} = 100 \]

Displacement in attachment between sheet and shear transfer element

\[ c_{2.3} = \frac{4 \cdot (n+1) \cdot s_{sc}}{n^2 \cdot n_{sc}} \cdot pr = 0 \text{ mm/kN} \]

Magnitude for formula in \( c_1 \)

Axial strain in end beams

\[ \alpha_3 = 1 \quad A = 3000 \text{ mm}^2 \]

\[ c_3 = \frac{0 \cdot c_2^3 \cdot \alpha_3}{4.8 \cdot E \cdot A \cdot b^2} = 0 \text{ mm/kN} \]

Shear flexibility, sum

\[ c = c_{1.1} + c_{1.2} + c_{2.1} + c_{2.2} + c_{2.3} + c_3 = 0.278 \text{ mm/kN} \]

Shear deformations in sheet if plane

\[ c_{plan} = \frac{2 \cdot a \cdot (1 + v)}{E \cdot t \cdot b} = 0.016 \text{ mm/kN} \]

Thickness for plane sheet giving same deformations as the profiled sheet

\[ t_{eff} = \frac{c_{plan}}{c} \cdot t = 0.045 \text{ mm} \]

Lateral torsional buckling calculations. Performance according to drawings.

| Effective thickness for trapezoidal plate | \( t = 0.0045 \text{ mm} \) |
| Cross section measurement at quarter point | \( b_0 = 668 \text{ mm} \quad t_0 = 40 \text{ mm} \quad b_u = 2400 \text{ mm} \quad t_u = 39 \text{ mm} \quad h_w = 2000 \text{ mm} \quad t_w = 18 \text{ mm} \) |
| Cross section magnitudes at quarter point according to calculations with CrossOpt | \( I_y = 3.510 \cdot 10^{11} \text{ mm}^4 \quad K_y = 2.140 \cdot 10^9 \text{ mm}^4 \quad K_w = 9.481 \cdot 10^{16} \text{ mm}^6 \quad B_y = E \cdot I_y \quad C = G \cdot K_y \quad C_w = E \cdot \) |
Table

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_w$</td>
<td>$-4931$ mm</td>
</tr>
<tr>
<td>$t_y$</td>
<td>$1.353 \cdot 10^8$ mm$^3$</td>
</tr>
<tr>
<td>Bridge length, yield point</td>
<td>$L = 65000$ mm</td>
</tr>
<tr>
<td>$f_{yk}$</td>
<td>$440$ MPa</td>
</tr>
<tr>
<td>$C_{ritn}$</td>
<td>$C$</td>
</tr>
<tr>
<td>Distance from shear center to bottom flange</td>
<td>$a_u = (400 + 499)$ mm</td>
</tr>
<tr>
<td>Distance from concrete plate to shear center</td>
<td>$a_b = a_u + h_w + 150$ mm = 3049 mm</td>
</tr>
<tr>
<td>Distance from steel beam to shear center</td>
<td>$a_b = a_u + 1190$ mm = 2089 mm</td>
</tr>
<tr>
<td>Loading from concrete (incl. mould and reinforcement) and steel, first sequence of concreting</td>
<td>$q_b = 38$ kN/m $q_s = 18$ kN/m $\sum q = q_b + q_s$</td>
</tr>
<tr>
<td>Loading point, weighted mean value</td>
<td>$a = \frac{q_b \cdot a_b + q_s \cdot a_s}{q_b + q_s} = 2740$ mm</td>
</tr>
<tr>
<td>Calculations according to StBK-K2 fig 4:312 (beam with camber stiffness)</td>
<td>$\beta = \frac{t_y}{2} \frac{B_y}{C_w} = -4.744$ $\gamma = \beta \left(\frac{a}{t_y} - 0.572\right) = 5.35$ $\frac{a}{t_y} = -0.556$ $k = \sqrt{\frac{C}{C_w}} = 0.093$ m$^{-1}$ $k \cdot L = 0.056$</td>
</tr>
<tr>
<td>From diagram (careful estimate)</td>
<td>$m_{kr} = 12$</td>
</tr>
<tr>
<td>Critical load according to elasticity theory</td>
<td>$q_{kr} = m_{kr} \cdot \sqrt{\frac{B_y \cdot C}{L^3}} \cdot \left(1 + \frac{\pi^2}{(kL)^2}\right) = 86$ kN/m</td>
</tr>
<tr>
<td>Critical moment in quarter point</td>
<td>$M_{kr4} = q_{kr} \cdot \left[\frac{L}{2} \cdot \frac{L}{4} - \frac{1}{2} \left(\frac{L}{4}\right)^2\right] = 6.959 \cdot 10^4$ kNm</td>
</tr>
<tr>
<td>Comparing with center of bridge</td>
<td>$M_{kr} = \frac{q_{kr} \cdot L^2}{8} = 9.279 \cdot 10^4$ kNm</td>
</tr>
<tr>
<td>Number of slenderness and reduction factor</td>
<td>$\lambda_b = \sqrt{\frac{W_0 \cdot f_{yk}}{M_{kr4}}} = 0.925$ $w_b = \frac{1.16}{1 + \lambda_b^2} = 0.625$</td>
</tr>
<tr>
<td>Moment capacity</td>
<td>$M_R = w_b \cdot W_0 \cdot f_{yk} = 3.722 \cdot 10^4$ kNm</td>
</tr>
<tr>
<td>Moment in quarter point</td>
<td>$M_E = \frac{3 \sum qL^2}{48} = 3.722 \cdot 10^4$ kNm</td>
</tr>
<tr>
<td>Safety</td>
<td>$M_{kr,r_{ritn}} = M_{kr4}$ $M_{ritn} = M_E$</td>
</tr>
<tr>
<td>Calculations according to StBK-K2 Fig 4:312 c (beam without camber stiffness)</td>
<td>$\gamma = \frac{a}{L} \sqrt{\frac{B_y}{C}} = 0.871$</td>
</tr>
<tr>
<td>From diagram</td>
<td>$m_{kr} = 11$</td>
</tr>
<tr>
<td>Critical load according to elasticity theory</td>
<td>$q_{kr} = m_{kr} \cdot \sqrt{\frac{B_y \cdot C}{L^3}} = 142.97$ kN/m</td>
</tr>
</tbody>
</table>

With respect to the fact that the camber contribution is small and that the deformation figure for lateral torsional buckling with and without camber stiffness is the same, the critical moment is corrected with a factor according to StBK-K2 4:312, Approximate course of action

$$q_{kr} = q_{kr} \cdot \sqrt{1 + \frac{C_{w} \cdot \pi^2}{C \cdot L^2}} = 161.061$$ kN/m
$$M_{kr4} = q_{kr} \cdot \left[\frac{L}{2} \cdot \frac{L}{4} - \frac{1}{2} \left(\frac{L}{4}\right)^2\right] = 6.38 \cdot 10^4$$ kNm

Number of slenderness and reduction factor

$$\lambda_b = \sqrt{W_0 \cdot f_{yk}} = 0.966$$ $w_b = \frac{1.16}{1 + \lambda_b^2} = 0.6$
<table>
<thead>
<tr>
<th>Moment capacity</th>
<th>$M_R = w_b \cdot W_0 \cdot f_{yk} = 3.572 \cdot 10^4 \text{kNm}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Safety</td>
<td>$\frac{M_R}{M_E} = 1.61 \text{ OK!}$</td>
</tr>
</tbody>
</table>

The concreting is performed in sequences. The safety above applies to the first sequence of concreting. For the later sequences this part of concrete has hardened and for the last sequence the stiffness is a lot greater than according to the calculations above.
3. The report “Evaluation of reason of failure of Y1504 Bridge in Kärrsjö”

1 Introduction

At the request of construction company 1 undersigned got the mission to evaluate the reason of the failure of the Y1504 Bridge.

The starting point of the statement are the documents available at the point of failure.

According to the assignment specification of construction company 1 the evaluations should answer following questions:

1. Is the planned basis, such as instructions and work descriptions, correct, established with the environmental responsibility from the planner and presented with sufficient information so secure correct execution?
2. Is the design of the construction performed correctly, construction stages analysed and are the conditions practically feasible?
3. Are any production of similar constructions performed anywhere else or are they altered or strengthened?
4. The changes made by construction company 1, (regarding sheet and attachments) do they have an important significance for the failure?

2 The construction of the bridge

The construction of the bridge is shown in the construction drawings [1] made by Technical consultant firm 1, dated 2002-03-15 and approved by the Swedish Road Administration, the department for bridge and tunnel according to a letter dated 2002-04-12 with reference number Y1504 2002:370. The bridge has a theoretical span of 65 meters and is made of a steel box and a concrete deck, working as a composite bridge. The steel box is supplied with diaphragms with a distance of about 7 meters. The steel box was made in a work shop in three different parts, transported to the building site where it was welded together and assembled on-site through launching. Before launching the parts of the permanent form work for casting of the concrete deck was applied on the steel box. This casting was to be done when the bridge was in its final position.

The part of the deck located between the top flanges of the box was to be casted on trapezoidal sheets, working as permanent form work. This sheet should at the same time, according to page 4.15 and 4.16 in the design calculations [2] give stability to the rotationally weak construction element the open steel box is, this through diaphragm action. The cantilevers of the deck was supposed to be casted on wooden form work carried by system from Hünnebeck.

The casting was planned to be made in two stages where the first stage included a part of about 33 meters centrally located. The second stage included the two side parts of about 16 meters each.
The failure

A description of the failure is presented in [3] made by technical consultant 1, construction company 1 Teknik in Gothenburg. From this and a protocol from a levelling of the top flanges of the steel box it is shown that there is a height difference between these, about 18 mm in the middle of the bridge, where the flange “upstream” had the lower level. This is an imperfection that could have been caused by an initial rotation of the cross section. The height difference could also have been due to different heights of the webs. However it was shown that the bottom of the box also was tilting at the same direction which was discovered through water runoff. Supposing the entire height difference was due to a rotation, this would have been 18/3400 i.e. 0.0053 rad or 0.3°.

According to technical consultant 1 [3] a levelling of the cantilever downstream showed that after casting the first part of concrete about 8 m³ this had lowered 17 mm. The calculated deflection for this load was 31 mm. The measuring point was placed in the middle of the edge beam i.e. about 3600 mm from the center line of the bridge. From this the additional rotation is estimated to be (31-17)/3600=0.0039 rad.

After casting the second part the deformation had almost totally been reverted. The calculated additional deflection for the second part was 30 mm. Supposing that the deflection of the cantilever has reverted totally the additional rotation from the two parts of concrete casting is 61/3600=0.0169 rad from which 0.013 rad has raised due to the second part, i.e. the additional rotation of the cross section during the second loading is about 3.3 times bigger than from the first loading. Applying Southwell’s plot [4] on the presented values gives results showing that you are at this stage very close to instability, which also occurred immediately after the checking of the measuring point.

Due to the increased rotation, the rotation moment due to outer loading also increases which in turn leads to an increased shear flow in the trapezoidal sheet. The overlap joints are loaded even more and eventually the edges of the screw holes fail. This reduces the shear stiffness of the sheet quickly and the continued rotations acts as if the box cross section is open with a global instability - lateral torsional buckling - as a result to this, which is characterized by a great rotation. Great cambering occurs during this rotation since the ability of taking the St Venant shear forces for the trapezoidal sheet gradually ends. It is without a doubt the ability of the trapezoidal sheet to take shear forces that is critical for the stability of the box.

4 Instability for transversely loaded beams with U-shaped cross section

The vulnerability for tilting is primarily controlled by the rotational stiffness and the side bending stiffness of the beam. In the present case the side bending stiffness is rather big and the rotational stiffness for the open box cross section rather low and therefore gets a dominating impact of the tilting phenomenon. For open, thin-walled cross sections the Vlasovks part of the rotational moment becomes significant whereas for closed cross sections the St Venants part dominates. Closed cross sections generally has a higher rotational stiffness and are therefore preferred when subjected to rotation. For the actual construction, the concrete deck together with the steel plate creates a closed cross section with a high rotational stiffness, suitable for taking up eccentric traffic load e.g.
As long as the U-shaped is open the construction is vulnerable to instability, which makes the stage of concreting critical. Eccentric loading, initial rotation, initial deflection in the horizontal plane, deviations of measurements for the cross section, wind load etc. are all effects that affects the rotational phenomena.

5 The sheet working as a stabilizing element

By connecting the top flanges of the steel box with a plate, a closed cross section is created with a significant increase of the St Venant rotational stiffness. It is however important that the plate and the attachments can be able to take the occurring shear flow in consequence of above mentioned loads and imperfections.

Due to the geometry of the trapezoidal sheet a distortion of the end cross section occurs that decreases the shear stiffness substantially. Together with the resilience in the connections this turns the shear stiffness, recomputed to an equivalent thickness for a plane sheet of the size 0.03-0.06 mm for in this case usual profiles. Even if the St Venant rotation stiffness hereby is augmented with a tenfold, the equivalent thickness is so small that the position of the rotational center is not noticeably affected.

The ability for the plate to resist shear forces in the actual construction type is mainly limited to following factors:

- The capacity of the attachments to the flanges
- The capacity of the connections in the side over-lap
- Bending of profile corners
- Web indentation
- Local buckling of web and flanges
- Global buckling

Thus there are several phenomena linked to the stabilizing function of the sheet needed to be considered to determine if the stability in the construction stage can be secured with the help from the permanent form work.

To do this you have to decide occurring forces in the plate, which generally means that you have to observe second order effects on the rotational phenomena, by the fact that the rotational stiffness still is low even though the additional plate.

By giving the plate such a decisive function in the collateral arrangements of the stability, high requirements on execution and checks also need to be done. The plate is through its thin walls of 0.7 mm sensitive to outer affects causing more or less serious reduction of the resistance.

6 Other possibilities to ensure stability for the concreting phase

6.1 Framework

Significantly bigger addition to the St Venant rotational stiffness could be achieved using framework built up by e.g. hollow section in the plane of the top flanges. This method has to be used for in plane curved boxes to not obtain big rotations from the concrete loading.

6.2 Trapezoidal medium plate
In Norway trapezoidal sheets made by medium plate (thickness 3-5 mm) has been used as permanent form work and stability increasing construction elements. The plate is attached to the flanges of the steel box through welding joints. Through this a significantly higher rotational stiffness than the one with trapezoidal thin plate is obtained.

6.3 Rotation preventing actions

By first casting parts of the deck at the ends of the bridge a required increase of the tilting load can be achieved.

As for the designing of the plate, it’s also here necessary to take in account for emerging forces in the elements increasing the stiffness.

7 Comments to the specific questions from construction company 1

7.1 Question 1

The documents controlling the final design and function of the steel construction are impeccable. I’m here referring to the drawings K2020-K2040.

On the other hand the documents describing the construction phase are insufficient. Specially noted:

- A separate description for the permanent form work in between beam webs is missing.
- It is not clear through the drawings how the plate should be attached to the end diaphragms or the other diaphragms. This is of great importance for the function and the resistance of the construction. With only two-sided attachment great axial forces occur in the plate close to the free boundary, forces that also affect the attachments.

To sum up, my opinion is that sufficient information is missing in the construction document for the bridge to be built in a safe way.

7.2 Question 2

Concerning the construction phase I believe that this is completely insufficiently analysed since the risk of global instability is not examined.

Indeed, the nominal strut force of 1.5 % of the actual pressure in the flange has been considered in a not clear way, where the constructor presents that the TRP 45 is able to resist a shear flow of 19 kN/m. The design value for the shear capacity in the TRP 45 t=0.85 mm is for example with respect to bending of profile corner about 13 kN/m in safety class 3 according to the Plannja catalogue [9], and with respect to the shear flow capacity in the overlap joints 9.0 kN/m (screw $\phi$ 6.3 mm, c/c 150 mm) calculated according to Tunnplåtsnormen StBK N5 [10] and therefore insufficient with the base from this approach. No chain is stronger than its weakest link, which in this case is the permanent form work.

Further mentioned in the construction documents is that various forces should be transferred to the diaphragms, but without any further presentation of how this is done.

The applied method could be compared to when for an axial pressure loaded strut of a framework, designing diagonals connection to verticals, but omit the buckling control of the connected strut in total.
7.3 Question 3

I’m not able to give a general answer to the question.

TYRÈNS AB has recently established construction documents for a similar bridge but with a smaller span of 57 meters. This bridge is provided with an end wall so you can create a torsion box with a rotational preventing function in the beam ends. The plate can be used here as permanent form work, but does not have computational been used for stabilization. Of course it is contribution in a positive way to the rotational stiffness of the bridge, but it is not considered sufficient to alone give enough stability.

It seems as if the method with a trapezoidal thin plate has been commonly used on the current bridge type. But, to my knowledge, this has never been used on a simply supported bridge with a span as large as this. The critical tilting load is strongly linked to the span of the beam, inversely proportional to the span in third potency. This could be an explanation to why you haven’t encountered this failure before.

7.4 Question 4

The answer to this question is given in the summary of the calculation results based on e.g. following conditions regarding the two current types of plate:

TRP 45 with \( t=0.85 \) mm and PEVA 45 with \( t=0.8 \) mm has according to catalogue data following capacity values

<table>
<thead>
<tr>
<th>Profile</th>
<th>( M_{\text{dim}} ), SFT kNm/m</th>
<th>( M_{\text{dim}} ), BFT kNm/m</th>
<th>( R_{\text{dim},c}=45 ) mm kN/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>TRP 45, ( t=0.85 ) mm [8]</td>
<td>3.44</td>
<td>3.34</td>
<td>27.14</td>
</tr>
<tr>
<td>PEVA 45, ( t=0.8 ) mm</td>
<td>3.61</td>
<td>4.63</td>
<td>23.62</td>
</tr>
</tbody>
</table>

Plannja states \( t_{\text{ber}}=0.755 \) mm. For PEVA 45 \( t_{\text{ber}}=0.72 \) mm.

The delivered plate PEVA 45 at a later control made by Testlab at Luleå Technical University turned out to have a steel core with the thickness 0.676 mm, i.e. 6.5 % less. The current tolerance for \( t_{\text{ber}} \) is 5 %.

For calculating the equivalent thickness for TRP 45 one can use the instructions in the publication nr 88 of ECCS “European Recommendations for the Application of Metal Sheeting as a Diaphragm” [5]. For PEVA 45 that has an extra bending in the wide flange, the instructions don’t give enough basis to take in consideration of the extra bending. Qualitatively this means a decrease of the shear stiffness. The occurrence of the bending affects the part of the shear flexibility that comes from profile distortion. A two dimensional analysis of a frame with the geometry of the profile contour indicates of a 10 % increase of the flexibility.

8 Calculations

The calculations that are presented in a separate appendix refers to check in ultimate limit state in the construction stage, i.e. load combination II according to BRO 94 [6] chapter 22.22.
8.1 Loads

Following loads shall be taken in account for in the calculations:

- Self-weight steel box
- Self-weight form work
- Reinforcement (refers to parts where there is no concrete)
- Concreting load
- Wind load

In addition to this the effect of different imperfection shall be taken in account for

- Imperfections such as initial crookedness, initial rotation, deviation in measurement of the cross section and eccentricity of concreting load.

The impact of the imperfections are not specifically mentioned in BRO 94 as something one should take in account for. In the co-valid document BSK 99 [7], chapter 2:3 it is explicitly mentioned that “Measurement and form deviations shall be considered when designing, if they are of significance for the verification of the requirements in the ultimate and serviceability limit state to be satisfied”.

Investigation of measurement or form deviations has not been made for the current bridge. An initial rotation has been located, as earlier mentioned by measuring the top flanges. Measurement deviations within given tolerances for the contour of the cross section could add a displacement of the position of the rotational center sideways relatively the center of gravity for the cross section of the magnitude 10-15 mm. For simplicity the effect of this is assumed to be added to an initial rotation amounting to 1/200.

Crookedness otherwise is reportedly “only visually checked” and can therefore not be quantified for the actual case. In lack of other information you could for example work on BSK 8:62 figure 8:62 “Normally predicted allowed incline and deviation from straightness and planeness” where the maximum amplitude for a curved beam is 0.0015L=97.5 mm.

The vertical camber – maximum 546 mm – has a non-stabilizing effect in conjunction with wind load. At the beginning of the casting the self-weight is applied and the maximum amplitude is therefore reduced to about 470 mm according to result from measurement.

BRO 94 indicates the value of wind load of 1.8 kN/m² with \( \psi_\gamma = 0.4/1.0 \) in load combination II. The load intensity 1.8 kN/m² appears a bit high to within reasonable probability occur at the same time as the casting of the deck. BRO 94 chapter 21.271 says that “If the wind load is assessed to have harmful impact during the construction phase or at the final stage of the construction or parts of, this should be examined in each single case.

In the present case the wind load definitely has a harmful impact especially on the plate and its attachment. Any investigation of this kind has not been presented by the constructor. The constructor has moreover in the assembly of loads, not even taken in account for the wind load.

I interpret the writing in chapter 21.271 in a way that if no particular investigation is made, than the wind intensity from bridge standards should be applied.
8.2 Measurements and dimensions

The calculations are based in measurements presented on the drawings. Two different cases are studied.

1. Case 1 refers to the performance of the failed bridge. Steel box according to drawing with PEVA 45 with \( t_{\text{ber}} = t_{\text{upp,mått}} = 0.676 \text{ mm} \), screws \( \varnothing 4.5 \text{ mm} \) in each profile bottom, and screws \( \varnothing 4.8 \text{ mm c/c 275 mm} \) in side over-lap joints. With these conditions you get \( t_{\text{ekv}} = 0.036 \text{ mm} \), calculated according to instructions in [5].

2. Case 2 refers to performance according to drawings. Steel box according to drawings with Plannja TRP 45 with \( 0.95 \times t_{\text{ber}} = 0.717 \text{ mm} \), (minus tolerance is included with respect to that in the calculations for Case 1 is based on measured value), screws \( \varnothing 4.5 \text{ mm} \) in each profile bottom, and screws \( \varnothing 6.3 \text{ mm c/c 150 mm} \) in the side over-lap joints gives \( t_{\text{ekv}} = 0.055 \text{ mm} \), calculated according to instructions in [5].

Decisive for the shear flow capacity is in both cases the screw connections in the over-lap joint. For case 1 this amounts to 3.6 kN/m and in case 2 to 8.3 kN/m, calculated according to StBK-N5 [9] 33:222 with respect to failure of the borders of the screw holes. The shear flow capacity is of sufficient meaning to be able to resist rotational moment caused by unintentional load eccentricity and geometrical imperfections next to wind load. In both cases it has been assumed that the yield point of the plate material is 350 N/mm\(^2\). The shear flow capacity in the case of failure is therefore only 43 % of the case according to the drawings.

When it comes to shear flow capacity in connections between plate and flange the differences are much smaller – 15.1 kN/m for case 1 and 17.1 for case 2. This circumstances could seem uninteresting in comparison with the much lower values decided for the connections in the over-lap joint. Since the plate is not connected to the end diaphragms, the local discontinuity can create great forces on the screw connections. Case 1 and 2 are at this fairly equivalent.

The size of these local forces is not further studied in this rapport, but is at the moment studied in a master thesis in the subject structural mechanics at the institution for mechanics at KTH.

9 Presentation of calculation results

In the following, results from two load cases are presented in order to enlightening the effect of different impacts.

Load combination 1 is meant to present results obtained with wind load according to BRO 94 and a sinusoidal shaped initial side way deflection with a maximum amplitude amounting to maximum tolerance of 1.5xL/1000, but without loading from concrete applied.

Load combination 2 is meant to simulate the conditions at the point of concreting.

Results shown with graphs are rotation, side deflection and shear flow in plate.

9.1 Load combination 1

Loads/imperfection

- Self-weight steel box + form work + reinforcement
- Wind load according to BRO 94, 1.8 kN/m², $\psi_\gamma = 1$
- Sinusoidal initial rotation, maximum 1/200
- Sinusoidal initial side-way deflection, maximum 0.0975 m
- Sinusoidal camber, maximum 0.47 m

**Figure 1- Rotation, case 1**

**Figure 2- Side deflection, case 1**

**Figure 3- Shear flow, case 1**
As presented in the diagrams, the shear flow capacity is exceeded in both case 1 and 2. Maximum shear flow is somewhat bigger in case 2 than case 1 – 29.2 kN/m against 28.2 kN/m which depends on the fact that the St Venant rotational stiffness in case 2 is bigger than in case 1. The remaining part of the rotational moment is taken by the Vlasovsk rotation.

Further you can see that the rotation is important – maximum 0.06 resp. 0.045 rad.
Regarding the investigated load case, following comment can be made

- The wind intensity of 1.8 kN/m$^2$ is extreme. If you start from given values in BSV 97 [10], you get the characteristic value of the wind intensity of 0.55 kN/m$^2$ that, with the form factor 2.0 and $\varphi_g = 1.3$ gives the designing wind intensity of 1.43 kN/m$^2$.
- The initial deflection side-ways – maximum 0.0975 – even that appears extreme even though the given value is given as normal tolerance in [7]. A reasonable attainable tolerance for a welded box should be around L/2000.

If the input is adjusted so they are adopted to these values, the maximum shear flows instead become 19.6 and 20.2 kN/m, still a lot higher than actual capacities 8.3 and 3.6 kN/m. The result of this excess is that failure of the edges of the screw hole occur and the St Venant rotational stiffness is significantly reduced and the global stability is jeopardized and tilting occurs.

Thus this is the situation for a realistic load combination – observe that no load from concreting is applied on the construction for this load combination. The investigation could be interrupted here with a finding that the from the constructor suggested performance is so under designed that global instability can be expected when applying concrete.

9.2 Load combination 2

Here, the load combination that’s assumed to prevail during failure is investigated. It is shown in the report by technical consultant 1 that the weather was warm and clear, why you can assume that the impact from wind is negligible.

Loads/imperfections

- Self-weight steel box + form work + reinforcement + concrete loading
- Sinusoidal shaped initial rotation, maximum 1/200
- Sinusoidal initial deflection side-ways, maximum L/2000 m
- Sinusoidal camber, maximum 0.47 m
- Eccentricity for concrete loading 0.025 m, corresponds to that the casting height deviates 10 mm at one of the edge beams and is varying linearly down to 0 at the other edge beam

With basis in these loadings, how big part of the first sequence of concreting that can be applied to reach the shear flow capacity $3.6 \cdot \gamma_n$ kN/m and $8.3 \cdot \gamma_n$ kN/m where $\gamma_n = 1.2$ is calculated.
The shear flow \(3.6 \cdot \gamma_n\) kN/m is obtained when approximately 15 m\(^3\) of concrete is applied. At the time of collapse about 17 m\(^3\) of concrete was applied. The shear flow is then increasing computationally to 4.8 kN/m and the rotation becomes 0.016 rad including the assumed initial
rotation of 0.005 rad. The total rotation immediately before the failure could be estimated to about 0.022 rad.

In case 2 the shear flow $8.3 \cdot y_t \, \text{kN/m}$ is obtained when about $45 \, \text{m}^3$ of the concrete is applied i.e. about 68% of the concrete in the first sequence of concreting. Associated rotation, side deflection and shear flow distribution is presented graphically in following diagrams.

**Figure 10** - Rotation, case 2

**Figure 11** - Side deflection, case 2
From the diagrams it is shown that for instance maximum total rotation amounts to 0.021 rad, which is corresponding to a height difference of about 160 mm in between the top surfaces of the edge beams, which is unacceptable.

It is assumed above that no wind load is affecting the construction since the primary purpose was to simulate the conditions at the time of failure. If you include loading from wind according to BSV 97 [10] with $\psi \gamma = 0.3$ this gives a wind intensity of about 12 m/s. For case 2 this wind contribution leads to an augmented maximum rotation to 0.031 rad and the shear flow increases to maximum 16.5 kN/m i.e. an overrun of the shear capacity of almost 100%.

10 Summary

Undersigned got the mission from construction company 1 to on the basis of various documents investigate the reason of failure of the Y1504 Bridge.

The base for this statement are the documents available at the time of failure.

According to the specification of the assignment from construction company 1, the investigation shall answer following questions:

1. Is the planned basis, such as instructions and work descriptions, correct, established with the environmental responsibility from the planner and presented with sufficient information so secure correct execution?
2. Is the design of the construction performed correctly, construction stages analysed and are the conditions practically feasible?
3. Are any production of similar constructions performed anywhere else or are they altered or strengthened?
4. The changes made by construction company 1, (regarding sheet and attachments) do they have an important significance for the failure?

The investigation shows that the primary reason of failure is that the performed construction with trapezoidal sheets as permanent form work next to the attachments of these has been completely insufficient in its function to create necessary rotational stiffness and strength for the bridge beam to avoid global instability as tilting.
Concerning the specific questions, these has been answered in the text above, why the most essential conclusions only are mentioned in the following.

1. The answer is without a doubt no.
2. The answer is no. The fact that the constructor hasn’t presented any check of global instability such as tilting is according to me extremely serious.
3. The question is hard to answer generally. Different possibilities to increase the rotational stiffness is discussed above.
4. The answer to this question is, according to my assessment by made calculations, without a doubt no. The failure would although have occurred later compared to the case of failure.
4. Failure of Y1504 Bridge over Gideålv in Kärrsjö: Comments to report made by Professor 2, technical consultant firm 2 AB

In the report “Evaluation of reason of failure of Y1504 Bridge in Kärrsjö”, Professor 2 got the mission from construction company 1, region Norrland to investigate the reasons of the failure of the bridge on June 12, 2002. According to the specification of the mission, the evaluation should answer the following questions:

5. Is the planned basis, such as instructions and work descriptions, correct, established with the environmental responsibility from the planner and presented with sufficient information so secure correct execution?

6. Is the design of the construction performed correctly, construction stages analysed and are the conditions practically feasible?

7. Are any production of similar constructions performed anywhere else or are they altered or strengthened?

8. The changes made by construction company 1, (regarding sheet and attachments) do they have an important significance for the failure?

Very brief, the answers from Professor 2 is “no” on question 1, 2 and 4 and “hard to answer generally” on question 3.

My comments on some of the headers are:

3 The failure

Third part: Even though the calculation of the rotation of the bridge is quite unsecure (measured level differences for the top flanges before concreting is mixed up with the difference between calculated and measured deflection of the cantilever downstream for the first and second loading) the result, that instability is very close, is correct.

Last part: Here we also agree. “It is without a doubt the ability of the trapezoidal sheet to take shear forces that is critical for the stability of the box.” Moreover we agree on most thing when it comes to the function of the bridge and the trapezoidal plate. The only differences in our ways of seeing it and our results will therefore be commented in the following.

5 The sheet working as a stabilizing element

Second part: The reduction of the shear stiffness of the plate has a lot to do with the distortion of the end cross section of the plate. The shear stiffness becomes, recalculated as an equivalent thickness for a plane plate according to European recommendations for diaphragm action in roofs, of the order 0.03-0.06 mm. In section 8. Measurements and dimensions the equivalent thickness is specified as

0.036 mm for performance of failed bridge (PEVA 45/0.676, Ø 4.8 c/c 275)
0.055 mm for performance according to drawing (TRP 45/0.717, Ø 6.3 c/c 150)

Here the results differ from my calculations in a way that PEVA 45 with its bending in the bottom of the profile and with less, sparser and smaller screws becomes much weaker:

0.015 mm for performance of failed bridge (PEVA 45/0.676, Ø 4.8 c/c 300)
0.045 mm for performance according to drawing (TRP 45/0.717, Ø 6.3 c/c 150)
For performance according to drawing the results are about the same, but for performance according to failed construction the difference between equivalent plate thicknesses is considerable. The difference between performance of failed construction and performance according to drawing according to the investigation 0.055/0.036-1=53% and the differences according to my calculations 0.045/0.015-1=200%.

The reason that we got such big differences between our results when it comes to the PEVA plate is partly because of the difference in assessing the size of the distortion at the end and party because of the fact that the shear deformations in the screw attachments in the side overlaps according to the European recommendations are independent from the dimensions of the screws when I have used the formula in 33:152 in the Swedish standard of thin plates.

\[ v = \frac{F_g}{K_d \sqrt{t^3}} \]

The shear deformation \( v \) is as shown inversely proportional to the diameter of the screw, \( d \) and the core of the plate thickness, \( t \) which together makes weaker and sparser screws and thinner plate matter a lot.

The moderate increase (10%) of the shear deformations due to distortion at the end plates for the PEVA plate at the attachment on one of the sides of the bending given in 7.4, last part, is a lot smaller than the 60% I concluded when comparing FEM calculations. More carefully performed calculations that is ongoing in a master thesis at the department of Steel constructing at KTH will give a difference of about 50% to 70% depending on what loading case you study.

8.2 Measurements and dimensions

For case 2, a 5% reduction of the calculation value of the steel thickness of the plate is taken in account of since the calculations for case 1 is based on a measured value. This is not correct since the calculation value already is considering a normal tolerance of 7%. The shear flow capacity will then become 11.3 kN/m instead of 8.3 kN/m and the shear flow capacity in the case of failure is only 32% of the case according to the drawings.

9.1 Load combination 1

Wind intensity

That the wind load according to BRO 94 (1.8 kN/m\(^2\) \( \psi \gamma = 1 \)) is extreme and unrealistic is clear from the investigation. But neither the reduced force of 1.43 kN/m\(^2\) is realistic. A more reasonable value is obtained of common wind load is used in combination with casting of the concrete, i.e. \( \frac{0.25}{1.3} \cdot 1.8 = 0.35 \) kN/m\(^2\).

The maximum shear flow is then decreasing drastically at the same time as the capacity is a lot bigger according to above. The statement that the performance suggested by the constructor should be a lot under designed is there for exaggerated.

9.2 Load combination 2

If the greater shear flow capacity of 11.3 kN/m is used instead of 8.3 kN/m the possible amount of concrete applied is increasing to about 70 m\(^3\) i.e. more than 66 m\(^3\) for the first sequence of concreting, when the other conditions remain the same. (The concrete then
represents a bit more than half of the loading. If proportionality would apply between impacts and loading an even bigger amount of concrete could have been applied, but that is not the case.)

10 Summary

With the calculation corrections made above, the answer on question 1 and 2 is “yes”. But you cannot comment with that much security as in the conclusion. What is absolutely sure is that the reason of failure is that the performed construction with trapezoidal sheets and with the performed attachments were completely insufficient to contribute with required rotation stiffness and strength for the bridge beam not to tilt. The changes made considering the plate and the attachments of this had a decisive meaning for the failure since they contributed to a weaker construction than the one intended on the drawings. The answer to question 4 is therefore definitely “yes”.
5. Damage investigation for Y1504 Bridge over Gideälven in Kärrsjön

Have received a letter from Bengt Wictorsson from Svenric technical consultant 2, Technical consultant firm 1, where he contests responsibility for the failure of the headlined object. Attached in this letter is also comments from Professor 1 to an earlier failure report made by Professor 2, technical consultant firm 2 AB.

I have been very involved in the investigation work that construction company 1 have made after the failure and would like to comment a few things in the above mentioned documents and partly direct the attention to what I feel is such errors that can make Technical consultant firm 1 responsible for the failure.

Comments to writings from technical consultant 2 and Professor 1

A formal detail in the letter from technical consultant 2 is that according to ABK96, which is the basis for the procurement of Technical consultant firm 1, it is incumbent to the consultant to prove his innocence. Therefore it is not construction company 1 that shall show an accountability to Technical consultant firm 1.

When it comes to the comment to the calculations of Professor 2 from Professor 1, it should be mentioned that it should be Technical consultant firm 1 that shows that the solution presented on the drawings is safe. Calculations on the construction phase with the trapezoidal sheet as a stabilizing element is a very difficult calculation and accepted calculation methods for the actual case does not exist. Only presenting what numbers that are used in the equations without having any discussion about the validity of the used equation is not enriching for the investigation. The technical discussion can also become tedious when industry practice does not exist for such constructions, but instead it becomes discussions about what people think and about principles.

Deficiencies in the planning of Technical consultant firm 1

It is especially two great deficiencies in the planning as I can see it. Both of them great enough to demand responsibility according to ABK96 alone.

First of all the design of the stabilizing impact from the trapezoidal sheet is substandard. The entire calculation contains of two pages which in turn contains errors. Looking at the advanced calculations performed after the failure, it seems as though the constructor did not realized the difficulty in the calculations of the above mentioned problem. Peter Collin, Technical consultant firm 1, have also at meetings after the failure expressed how complicated these calculations of rotation are. I think it is obvious that the constructor is neglecting the investigation of this stability problem. Later it seems as if a solution from another project is used without checking is the requirements for this solution was obtained also for this project. I will leave technical details about this at a later time.

Second of all it is not clear in the drawings that, what is called permanent form work, is a primarily stabilizing element. In for instance the book “Stabilization through diaphragm action” SBI publ. 169, by Professor 1 it says: If a sheet is used for diaphragm action it shall be design as a carcass structure in general. On the drawing it should be shown that the sheet has a stabilizing function. The strength of the sheet is first of all depending on the attachments to the beams and the joints”.

81
It is not obvious from the drawings how big part of the sheets that should be assembled before the concreting starts, neither how much eccentric loading that is allowed. Since the constructor hasn’t realized the importance of the sheet, and has not been able to convey its important function, it doesn’t feel like it can be required from a non-expert as the foreman on-site is, to realize this. The industrial practice is that you always seek the cheapest, equivalent product. That’s why it is likely that you, without any experience from the Y1504 Bridge, would ask for alternative form-work. I.e. another contractor or another site manager on construction company 1 will act in the same way for another project in the future. The delivered sheet is considered as equal or better.

The lack in presentation on the drawings shows a careless disregard of the responsibility the consultant has for the work environment and little care to try to transfer important information to the working site. This has, except for harm on the object, also put the staff in danger.

I claim that it is obvious that the consultant has a big responsibility for the occurred situation, party through not showing good workmanship when designing of the permanent form work, and partly the lack in transferring the necessary information to the working site, to create a safe working environment and to end the project in a safe way.
6. Comments to the opinions of Professor 1 about the report “Evaluation of reason of failure of Y1504 Bridge in Kärrsjö” by Professor 2

1 Background

At the request of construction company 1 undersigned got the mission to evaluate the reason of the failure of the Y1504 Bridge.

The starting point of the statement are the documents available at the point of failure.

According to the assignment specification of construction company 1 the evaluations should answer following questions:

9. Is the planned basis, such as instructions and work descriptions, correct, established with the environmental responsibility from the planner and presented with sufficient information so secure correct execution?
10. Is the design of the construction performed correctly, construction stages analysed and are the conditions practically feasible?
11. Are any production of similar constructions performed anywhere else or are they altered or strengthened?
12. The changes made by construction company 1, (regarding sheet and attachments) do they have an important significance for the failure?

In the summary to my report “Evaluation of reason of failure of Y1504 Bridge in Kärrsjö” dated 2003-04-10 I answered “no” to the questions 1, 2 and 4. Question 3 cannot be answered with a clear yes or no.

Professor 1 gave, on a mission from the constructor of the bridge, Technical consultant firm 1 an opinion called “Failure of Y1504 Bridge over Gideälv in Kärrsjö: Comments to report made by Professor 2, technical consultant firm 2 AB” dated 2003-08-15.

Contrary to the conclusions in my investigation, Professor 1 gives the answers “yes” to question 1, 2 and 4.

I have been given the opportunity from the lawyer of construction company 1, “White and Case Advokat AB” to give my comments on the comments from Professor 1, and here they are.

2 Additional observations

Starting with the comments from Professor 1 to chapter “10 Summary” of my investigation where he with reference to his “corrections of the calculations” gives the answer “yes” to questions 1, 2 and 4. I choose to first make comments to the answer on questions 1 and 2.

2.1 Question 1 and 2

Referring to “corrections to the calculations given above the answer to question 1 and 2 is Yes” according to Professor 1. I mean that these “corrections”, mainly referring to a calculation of an equivalent plate thickness, are not enough basis to answer question 1 and 2 with a “yes”.

83
Even though the answers in section 10 in my report can be perceived as somewhat categorical – according to Professor 1 “very short” – I still stand by this with reference to following motivations and clarifications.

2.1.1 Question 1

Question 1 is asked with respect to a lot of different criteria – my no-answer refers to how “the planned basis, as instructions and work instructions are correctly performed and presented with sufficient information to secure correctly performance”. The parts referring to “current engineer practice” respectively “the working environmental responsibility of the planner” I refrain to answer specifically since requirements on these should be met of the planned basis is meeting the rest of the criteria.

- On the construction drawings the plate is only named as permanent form work. There are no information in the documents that the sheet should have a stabilizing function in the construction phase. I mean that it is of high importance that the work management is informed about these kinds of things either through drawings or through working instructions.
- The separate working instruction that according to drawing K 20 20 is for “the permanent form work of trapezoidal sheet between beam webs” does not exist.
- For the plate to work as functioned it needs to be attached all the way around i.e. both along the top flanges of the longitudinal and the end beams. For the current case, the top flanges of the longitudinal beam is not at the same level as the top flanges of the end beams. The drawings does not tell how to attach the plate to the end beams, which it should since the attachment all around is of such great importance for the stabilizing function. If only two-sided attachment were done, as for the current case, an interruption in the shear flow is obtained at the free border of the sheet, which results in high forces in the plate, directed parallel to the free end respectively a high cutting force in the screw attachment about the area of the free border. That this is the case is shown in figure 1.
Figure 13- Failure at the border of the screw holes

Referring to the above mentioned facts my answer to question 1 is no.

2.1.2 Question 2

Question 2 include three parts

- “Is the design of the bridge performed correctly?” If the question is limited to refer to the operational stage the answer is yes. But if the construction phase is included then my answer is no, since I mean that the construction according to the drawings don’t give sufficient security in the construction stage. Also see my answer to question 4.

- “Are the construction stages analysed?” In the construction documents available at the time of failure there were no analyse of the construction phase. In the documents there were neither any presentation of the impact from the wind. BRO 94 is giving a wind load of 1.8 kN/m$^2$ if no particular investigation is made. Professor 1 agrees with me that this value is unrealistically high. If the wind load instead is determined according to “Boverkets handbok” for snow and wind load a characteristic value of 0.55 kN/m$^2$ can be determined with the form factor $\mu = 2$. According to BRO 94 chapter 22.22 the most unfavorable variable load be given the highest value of the load coefficient $\psi \gamma$. Other variable loads is given the lower value. For wind load, BRO 94 provides $\psi \gamma = 0.4/1.0$. With the rotational centre about 0.9 meters below the bottom of the box, the higher value of $\psi \gamma$ gives a shear flow of 15.1 kN/m only from the wind load., which exceeds the, from Professor 1, given value of capacity of 11.3 kN/m with about 33%. A check of the wind case would therefore give the constructor an indication to further investigate the construction phase.

- The question whether “the requirements are practically doable” is imprecise and will not be commented.
2.1.3 Question 3

The question is not commented by Professor 1.

2.1.4 Question 4

The calculated equivalent plate thicknesses, made by Professor 1, deviates significantly from my calculation results. Professor 1 has in his comments discussed several different reasons to these differences. It is not my intention to further discuss these. I note that Professor 1 reach the result than the equivalent thickness for the, according to drawings presents performance, is 0.045 mm.

With basis in these results and loads and imperfections as follows

• Self-weight steel box + form work + reinforcement
• Sinusoidal initial rotation, maximum 1/200
• Sinusoidal initial side-ways deflection, maximum L/2000 m
• Sinusoidal camber, maximum 0.47 m
• Eccentricity in concreting 0.025 m, corresponding to that the concreting height deviates 10 mm at one of the edge beams and varies linear down to 0 at the other edge beam

the shear flow capacity (according to Professor 1) 11.3 kN/m at an applied amount of concrete of 54 m$^3$. Calculated shear flow for full casting load (66 m$^3$) is 15.2 kN/m. In these calculations, wind load is not taken in account.

If the wind load is 0.35 kN/m$^2$ (reasonable value according to Professor 1) a shear flow capacity of 11.3 kN/m is obtained at applied concrete of 29 m$^3$. Calculated shear flow for full concreting load (66 m$^3$) plus wind is 23.5 kN/m.

It is also important to take in account for the so called serviceability limit state i.e. for instance performing deformations. When reaching the shear flow capacity of 11.3 kN/m the calculated rotation of the beam is about 2.4%, which results in a height difference between the edge beams of up to 170 mm, which is due to the low rotational stiffness prevailing before the concrete deck is completed. At full concreting load – 66 m$^3$ – the height difference would have increased to 310 mm if the connections had enough strength.

After the bridge failed a new bridge has been completed, where the beam was provided with a framework built by hollow profiles connecting the top flanges of the beam. Through this the rotational stiffness is increasing in the construction phase by a tenfold relative to the original drawing performance, which leads to that the addition of rotation for full concreting load (66 m$^3$) and wind, computationally becomes of the size order of 0.15%.

3 Summary

Professor 1 made comments about my report “Evaluation of reason of failure of Y1504 Bridge in Kärrsjö”. Professor 1 leads to completely opposite results in comparison to my investigation.

Professor 1 is making the assessment that with the performance according to the drawings, the bridge would be able to resist an applied amount of concrete of 70 m$^3$ i.e. a calculated safety
of $70 \cdot 24 + \frac{1150}{66 \cdot 24 + 1150} = 1.035$. A very low security with respect to current insecurities in the used calculation methods. Professor 1 also considers that my calculation of the equivalent thickness of the plate is not correct.

I have therefore in a new global analyse of the construction phase assumed the equivalent plate thickness of 0.045 mm, calculated by Professor 1, and also the shear flow capacity of 11.3 kN/m, where I reached the possible concreting load of 54 m$^3$ of no wind is present, and 29 m$^3$ if there is a wind intensity of 0.35 kN/m$^2$. Corresponding security becomes $54 \cdot 24 + \frac{1150}{66 \cdot 24 + 1150} = 0.894$ and $29 \cdot 24 + \frac{1150}{66 \cdot 24 + 1150} = 0.675$.

It is clear that the plate and the connections chosen by the contractor offers worse properties when it comes to stiffness and strength compared to the prescribed plate – the resistance was condemned already at 25% of the intended concrete loading. My calculations, based on the determined values from Professor 1 of the equivalent thickness (0.045 mm) and the shear flow capacity (11.3 kN/m), shows that even the plate prescribed on the drawing is insufficient – and its resistance is condemned at 82% of the intended concrete loading if there is no wind. At same time affect from wind, corresponding value is 44%. My conclusion is that the bridge would have failed whether the contractor would have made the changes or not.
7. Failure of the Y1504 Bridge over Gideälvd in Kärrejö
Calculations for the construction stage of a bridge performed according to drawing 040K2020 by Technical consultant firm 1, dated 2002-03-15.

Summary of results
The Y1504 Bridge with its thin form work is a jeopardizing construction inviting to failure during the construction stage.

The prescribed corrugated sheet Plannja \( t = 0.85 \text{ mm} \) is without any problem taking the concrete load from the first concreting stage between its supports. This sheet on the other hand, with its screw and nail attachments has a low shear resistance. A calculation, on the basis of a reasonable design case of initial deformation and load eccentricity, indicates that the bridge would had failed after about 40\% of the first concreting stage was performed. The resistance of the form work with its attachments between the sheets and between the sheets and the top flanges of the bridge is exceeded with a start at the end of the bridge beam where the greatest shear forces occur caused by eccentric loading. You get a progressive failure where the bridge in total fast loses its rotational stiffness and tilts/is subjected to lateral torsional buckling. The failure in 2002-06-12 would have occurred whether construction company 1 made the changes for the sheets and its attachments or not.

I start both the machine written and the hand written part of my document with some general reflections and function describing parts. This is a habit of mine remaining from my teaching in building mechanics and solid mechanics on Chalmers and from my occupation in external courses for civil engineering in the industry.

My mission
Lawyer Anders Reldén from White & Case in Stockholm called on 2004-05-18 and sent me thereafter a letter dated the day after. In the letter the failure of the Y1504 Bridge, that had been under construction by construction company 1 and designed by Technical consultant firm 1, is shortly described. Later it was orally decided that I would take on the four questions asked in the statements from Professor 2 and Professor 1. The fourth central question should according to Anders Reldén correctly be: “Would the failure still have occurred whether construction company 1 made the changes regarding the plate and attachments of it?”

My answers to the four questions
1. Planned material is according to me insufficient since a total analysis of the resistance and strength of the bridge during the construction phase is missing. No information is given on the drawings or any other place about the double function of the corrugated sheet, partly as permanent form work, partly as rotational resistance increasing, rotational stiffness increasing and stability increasing component (especially at the end of the beam) during the construction phase.
2. The construction phases are not analysed with respect to the above mentioned double requirements on the form work. Only the function as form work that carries the concrete during the concreting is secured plus some bracing of flange in pressure.
Nothing is mentioned about the importance of anchoring the form work to the end diaphragms and this kind of anchoring is not prescribed.

3. I don’t have any first-hand information but refer to the given information of Professor 2 where it says that Norwegian steel building contractors according to task in corresponding situation uses form work of the thickness 3-5 mm (instead of this prescribed thickness of 0.85 mm) which is welded to both of the top flanges of the bridge (instead of screws and nails).

4. The failure in 2002-06-12 would have occurred whether construction company 1 made the changes regarding the sheets and its attachments or not.

Some overall reflection

The calculations made by Technical consultant firm 1 during the spring 2002 contains in section “4.9 Check in construction phase” nothing about the requirements at both ends of the beam of resistance and strength in shear for the form work, regarding its ability to resist the tendency of the end cross section to tilt/arch/camber. The word tilt/arch/camber only occurs once in the documents by Technical consultant firm 1 (in section 4.1) but there it is wrongly used for the change of shape in the cross section plane that usually, for rectangular box beams is mentioned as parallelogramisation which is prevented by diaphragms.

Overall, technical consultant firm 1 is not discussing the rotational stiffness, rotational resistance or the stability of the beam in total during the first sequence of concreting at all in their documents from spring 2002. No computer program doing stress and deformation calculations is automatically alarming about threatening instability. The constructor is referred to his own (as I usually say) “sick imagination” to predict possible types of eccentricities and instabilities which thereafter can be investigated and lead to workarounds (in current case to introduction of an increased shear stiffness at the ends of the bridge for the horizontal connection between the top flanges of the beam).

Accepting authorities as SRA has obviously, just as technical consultant firm 1, not realized the risk of tension and instability failure during the construction phase of the bridge. As I can see, construction company 1 has believed that the essential function of the form work was to resist the concrete loading in bending during concreting and in tension connect the webs of the steel box. The subcontractor of the actual used form work PEVA 45, Svenska Bygglåt AB in Örnsköldsvik, has the least of all involved realized that the corrugated sheet had an important role on the side of its function as permanent form work.

Why does neither technical consultant firm 1 during the planning nor SRA during the approval of the documents observe the essential mechanical phenomena regarding resistance and stability for a construction as the current bridge beam? As a constructor it is easy to believe that if all the requirements from the standards are fulfilled then everything is alright, and you forget your responsibility for functions and behavior not specified in the sections of the standards you study.

Calculations

I have not myself tried to estimate the equivalent thickness \( t_b \) for a plane sheet that in my calculations replace the corrugated sheet Plannja TRP 45 \( t=0.85 \) mm. Instead I have adopted the value \( t_b=0.055 \) from Per Olof Professor 2, who used it on basis of the Europe
recommendation by ECCS. At the same time, I observed that Professor 1 on basis of SBI publication 169 found the smaller and less favorable values $t_b=0.038 \, \text{mm}$ 2002-06-26 and $t_b=0.045 \, \text{mm}$ 2002-11-05.

Neither when it comes to the capacity for the sheet TRP 45 $t=0.85 \, \text{mm}$, with its prescribed nail and screw attachments, to transfer the calculated shear flow $D \, [\text{kN/m}]$ to the top flanges of the bridge and in between the individual sheets have I conducted my own investigation. My calculated value $D=21 \, \text{kN/m}$ during the first stage of concreting, with reasonable assumptions of initial crookedness for the bridge beam and for eccentricity of the concrete load, can be compared to allowed value from Plannja, Professor 2 and Professor 1. The later indicates allowed values between 9 and 13 kN/m. I have chosen the design value of 12 kN/m.

I have chosen to, in my hand calculations, methodically and controllably move forward when deciding the positions of the geometry center of the cross section of the bridge, GK, the same for the shear center, SC and when calculating the surface moment of area $I_y$ and $I_z$ about the $y$- and $x$-axis through GC. I have calculated the cross section factor $K_v$ for the Saint Venant rotation stiffness on the same way.

When choosing design crookedness and eccentricity when estimating reasonable unbalance loading causing rotation of the bridge beam I have used my own judgement. I have almost never explicitly used data from building standards even though they have of course always been in the background.

I have calculated the critical value of the load (self-weight + applied concrete during first sequence of concreting) that leads to failure of the beam due to progressive shear failure in the sheet at both ends of the bridge, which leads to lost rotational stiffness for the beam and finally a total failure due to lateral torsional buckling. In a rough estimation I have showed that an intact bridge (without shear failure at the ends) probably would have been able to take the entire loading from the first sequence of concreting without lateral torsional buckling occurring.

**Calculations of critical load made by Professor 1**

In the part “Lateral torsional buckling calculations. Performance according to drawings” of his document 2002-11-05, Professor 1 finds his calculations according to StBK-K2 that safety against failure due to lateral torsional buckling during the first sequence of concreting should be satisfied ($M_R/M_E=1.61$). The equivalent thickness 0.0045 mm of the corrugated sheet Plannja TRP 45 $t=0.85$ was used there. In the above mentioned ratio, $M_R$ is the bending moment capacity in the quarter point of the bridge beam, with respect to lateral torsional buckling and $M_E$ is the actual bending moment in the same positions caused by self-weight and concrete loading. The value of 1.61 on the ratio means that you would have a safety margin of 61% against lateral torsional buckling.

In his calculation 2002-06.26, Professor 1 finds the equivalent sheet thickness of 0.0038 mm for Plannja TRP 45 $t=0.85 \, \text{mm}$. He gets the rounded equivalent thickness of 0.04 mm with a lower safety ($M_R/M_E=1.18$) against lateral torsional buckling during the first sequence of concreting, full loading.

Here it is important to observe that Professor 1 in his calculations assumes that the bridge beam, up to full loading from the concrete in the first sequence of concreting, is intact, i.e.
without shear failure in the sheets at the ends of the bridge and thereby a lower rotational stiffness than the beam in total. As mentioned above, I find in my design case that shear failure initiates and proceeds when only 40% of the concrete loading during the first sequence of concreting is applied. Lateral torsional buckling of the entire bridge beam therefore follows as a result of reduced rotational stiffness of the beam.

Strength and stability

"Buildings mechanics" and "mechanics" treats three main type of problems, namely:

- Stress and deformation problems
- Stability problems
- Vibrations problems

Here, the first two types are current and center on the properties for the prescribed corrugated sheet Plannja TRP45, t=0.85 mm including the attachment in the top flange of the steel construction and the overlap of the sheeting (covering width of one sheet is 0.88 m).

- Does the sheet have, with its own nail and screw attachment, sufficient strength (including local stability) to resist designing shear flow D [kN/m] (in addition to the bending moment of the loading from concreting)?
- Does the sheet have, with its own nail and screw attachment, sufficient stiffness (in shear) to, during the first sequence of concreting, be able to stabilize the bridge in large so that it is not at risk of failure through lateral torsional buckling?

Designing shear flow and proportion of concrete load during the first sequence of concreting at shear failure and lateral torsional buckling shall be calculated.

Highest requirements at the end of the bridge

During the first sequence of concreting the highest requirements are made on the corrugated sheet (Plannja RP 45, t=0.85 mm) by the end of the bridge when it comes to the strength and the stiffness in shear of the sheet. In the middle part of the bridge (33 m during the first sequence of concreting) there are only requirements on the strength of the sheet in bending during concrete load.
The corrugated sheet fields by the end of the bridge shall resist the tendency of the cross section of the bridge to welter (become “non-planar”, with axial displacement $u(x, y) = -y'(x)w(s)$). The sheet fields shall be able to carry calculated maximum shear flow $D$. They shall have sufficient shear stiffness $D/\gamma = G \cdot t_{\text{equivalent}}$ ($G=$shear modulus) to give security against overturning of the bridge during concreting.

Shear center/center of rotation role. Indications.

In the following chapter the position of the shear center SC (= center of rotation VC) will be calculated for the current cross section for the steel construction before and during the first sequence of concreting.
The sheets have the thickness $t_a$, $t_b$ and $t_h$ where $t_b$ is the equivalent thickness planar sheet replacing TRP45, $t=0.85\ mm$.

The geometry center (center of gravity) of the cross section is designated as GC.

Bending stiffness $EI_y$ och $EI_z$ about y and z axis through GC.

Saint Venant torsional stiffness $GK_v$ and Vlasovsk arch stiffness $EK_w$.

Outer transversal loading $P_z=P_z(x)$ and $P_y=P_y(x)$ on the beam through its axis of shear center gives planar bending of the beam in the vertical $zx$-plane respectively the horizontal $xy$-plane. A single acting outer torsional moment $q=q(x)$ causes rotation of the cross section for the beam about the shear center axis. When analysing, the outer loading is replaced by statically equivalent $p_y$, $p_z$ and $q$ (for each position $x$).

Cross section of the bridge
The width and height measurement will in calculations be used as measurements in between the center lines of the sheets (even though they deviate somewhat from that).

The smaller thickness of the sheeting in the top and bottom flanges refers to the end of the bridge.

The cross section can be idealized to the following when calculating the positions for geometric center GC and shear center SC mm.

<table>
<thead>
<tr>
<th></th>
<th>Middle part of the bridge</th>
<th>End part of the bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_c$</td>
<td>$0.750 \cdot 0.040 = 0.0300 \text{ m}^2$</td>
<td>0.0218</td>
</tr>
<tr>
<td>$t_a [\text{mm}]$</td>
<td>0.039</td>
<td>0.022</td>
</tr>
</tbody>
</table>
Equivalent thickness $t_b$ for planar sheet according to ECCS. Professor 1 calculates according his SBI publ. 169 $t_b=0.045$ mm.

**Centre of gravity/geometric center GC**

Total cross section area

$$A = 2 \left( A_c + a t_a + \sqrt{h^2 + c^2 t_h} + b t_b \right), \quad \sqrt{h^2 + c^2} = 2.09 \text{ m}$$

Static moment of area about the top flange gives

$$f = \frac{\left( 2a t_a h + 2\sqrt{h^2 + c^2 t_h \frac{h}{2}} \right)}{A}$$

Middle part:

$$A = 2(0.0300 + 1.10 \cdot 0.039 + 2.09 \cdot 0.018 + 1.7 \cdot 0.055 \cdot 10^{-3}) = 0.2212 \text{ m}^2$$

$$f = 1.12 \text{ m}$$

End parts:

$$A = 0.1672 \text{ m}^2$$

$$f = 1.03 \text{ m}$$

As an average when calculating the center of gravity for the load application level can be used

$$f = 1.10 \text{ m}$$

**Surface moment of inertia $I_y$**

Middle part:

$$I_y = 0.1680 \text{ m}^4$$

End parts:

$$I_y = 0.1188 \text{ m}^4$$

Deflection from self-weight of 115 tons steel and 160 tons concrete from the first sequence of concreting is approximately

$$\frac{5PL^3}{384EI_y} = 293 \text{ mm}$$

Reasonable result. Initial elevation is 546 mm according to drawing 040 K2020 by technical consultant firm 1 2002-03-15.

**Surface moment of inertia $I_z$**

Middle part:
The resultant to the horizontal shear flow on the length 2a is

\[ D_h = 2a D_z + \frac{\frac{2}{3} a t a^2}{I_z} \]

\[ I_z D_h = 2a (A_c b + l t_n \frac{a+b}{2} + \frac{2}{3} at a^2) \]

Moment about the intersection point x gives the shear center distance \( e_0 \).

\[ e_0 = \frac{a}{c} h (1 - D_h) \]

Middle part:

\[ D_h = 0.7358 \text{ N/N} \]

\[ e_0 = 3.667 (1 - 0.7358) = 0.969 \text{ m} \]

End parts:
\( D_n = 0.7366 \text{ N/N} \)
\[ e_0 = 3.667(1 - 0.7366) = 0.966 \text{ m} \]

Insignificant difference, take
\[ e_0 = 0.97 \text{ m} \]

**Shear center SC closed cross section**

A constant shear flow around the cross section is added to the shear flow for the cut up cross section. The shear flow of the open cross section becomes approximately as before.

For the total shear flow \( D_{(s)} \)

\[ \phi \frac{D_{(s)}}{\tau_{(s)}} ds = 0 \text{ where } \frac{D_{(s)}}{\tau_{(s)}} \text{ is the shear stress } \tau_{(s)} \]

The Simpson rule shall later be used for the inclined web. The middle ordinate \( D_{1.5} \) for the parable is needed.

\[ D_{1.5} = D_1 + \frac{lt}{2h} \frac{(a+b)}{l_x} = D_1 + lt \frac{a^3 + b^3}{8} \]

The contribution to \( \phi \) from the open cross section is

\[ \Delta \phi_0 = 2 \left[ \frac{1}{3} \frac{D_0}{t_b} + \frac{l}{6} \frac{D_1 + 4D_{1.5} + D_2}{t_h} + a \frac{D_2}{t_a} + \frac{2}{3} a \frac{D_3 - D_2}{t_a} \right] \]

\[ = \frac{2}{l_x} \left[ \frac{b^3}{6} + l \left( \frac{A_c b}{t_h} + \frac{0 + \frac{a^3}{6} + \frac{a^3}{2}}{l} \right) + a \left( A_c b + \frac{a+b}{2} \right) \right] + \frac{2}{3} a \frac{a^2}{2} \]

\[ = \frac{2}{l_x} \left[ \frac{b^3}{6} + l \left( \frac{A_c b}{t_h} + \frac{a^3}{6} \right) + a \left( A_c b + \frac{a+b}{2} \right) + \frac{a^3}{3} \right] \]
The constant shear flow $D_r$ counter clockwise ($ds<0$) gives the contribution

$$\Delta_r \phi = -D_r \left( \frac{2b}{t_b} + \frac{2l}{t_h} + \frac{2a}{t_a} \right) = -2D_r \left\{ \frac{a}{t_a} + \frac{b}{t_b} + \frac{l}{t_h} \right\}$$

$\Delta_0 \phi + \Delta_r \phi = 0$ gives

$$D_r = \frac{1}{l_z} \left[ \text{see above} \right] \left\{ \text{see above} \right\} \quad [1/m]$$

Moment about the intersection point $x$ (see open cross section):

$$V_y \left( \frac{a}{c} h - e_s \right) = D_h \frac{a}{c} h + D_r 2A_{in} , \quad V_y = 1$$

where the contained area in the cross section is

$$A_{in} = 2 \frac{a+b}{2} h, \quad e_0 = \frac{a}{c} h (1-d_h)$$

$$e_s = e_0 - 2D_r A_{in} = e_0 - 2(a+b)hD_r$$

Shear center is approaching geometric center.

Middle part:

$$D_r = 0.00128$$

$$e_s = 0.955 \text{ m} \approx e_0 = 0.969 \text{ m}$$

End parts:

$$e_s \approx e_0$$

The extremely thin upper sheet (equivalent thickness $t_b=0.000055 \text{ m}$) moves the shear center a short distance (14 mm) closer to the geometric center when passing over from open to closed cross section, i.e. $e_s$ is practically equal to $e_0$.

**Torsion and shear flow by eccentric loading**

Closed cross section (box cross section) transverse the beam is considered. The ends has finger bearings. ($y=0$).
Completed first sequence of concrete with $c=33$ m and $66$ m$^3$ concrete is studied whereof the vertical loading intensity

$$p_{btg} = \frac{66\, m^3 \cdot 2400\, \frac{kg}{m^3} \cdot 10\, N/kg}{33\, m} = 48\, kN/m$$

Totally

$$p_{btg} \cdot c = 48 \cdot 33 = 1584\, kN \text{ (approximately 160 tons)}$$

A straight SC-axis shall be considered between the ends of the beam.

The entire steel construction weighs approximately 115 tons and has the weight of 1150 kN (which gives $p_{steel} = \frac{1150}{65} = 17.7\, kN/m$)

$f=1100$ mm
The eccentricity $e_{GC}$ can be in initially equally warm condition plus addition to the sun irradiation against one side of the bridge.

The vertical forces from the self-weight of the steel construction and the concrete could have eccentricity considering the axis of shear center $SC$ of the steel beam. Reasonable designing assumption is that the center of gravity-line (GC-axis) has an initial crookedness with the middle ordinate.

$$e_{GC} = 0.0015L = 0.0015 \cdot 65000 = 98 \text{ mm}$$

An initial rotation deformation (distortion) of the cross section of the beam could in a reasonable designing assumption amount to, in the middle of the bridge

$$\varphi_0 = \frac{10 \text{ mm}}{1700 \text{ mm}} = 5.9 \text{ mrad} = 0.34 \text{ degrees}$$

When the concrete is loaded the rotation of the cross section will augment in addition to the initial value.

Total outer torsional moment loading from the self-weight of the steel construction becomes (sinusoidal shaped crookedness $e_{GC} \sin \frac{\pi x}{L}$)

$$Q_{steel} = 71.8 \text{ kNm}$$

From this, half is taken by each support

$$\tau_{max}^{egen} = 36 \text{ kNm}$$
Concerning the concrete load, its initial eccentricity is amplified by the rotation of the cross section rotation to, in the middle of the bridge,

\[ e_{GC} + f y_0 = 98 + 1100 \cdot 0.0059 = 105 \text{ mm} \]

More important is that the rotation of the cross section with augmenting loading will amplify above \( \varphi_0 \). A reasonable designing assumption could be \( e_{btg} = 120 \text{ mm} \) (mean value along loaded length) whereof the total outer rotational moment loading

\[ Q_{btg} = p_{btg}c_{btg} = 48 \text{ kN/m} \cdot 33 \text{ m} \cdot 0.120 \text{ m} = 190 \text{ kNm} \]

and

\[ \tau_{max}^{btg} = \frac{1}{2} 190 = 95 \text{ kNm} \]

The position of shear center SC has no significance in these calculations, i.e. the horizontal displacement in the bridge field of this mode. Maximum rotational moment occurs at the end of the beam and no cross section rotation occurs here, neither any displacement of SC. Intact shape of cross section is assumed.

You should consider the rotation of the beam about a straight shear center axis between both of the ends of the bridge beam.

Incidental inclination transverse the concrete mould could give, let’s say, 10 mm thicker concrete on one of the sides and 10 mm thinner concrete on the other side of the center line of the bridge with a variation according to below

The depicted unbalanced forces R becomes for the entire sequence of concreting

\[ R = \frac{11}{2} 7.65 \cdot 0.010 \text{ m} \cdot 33 \text{ m} \cdot 2400 \frac{\text{kg}}{\text{m}^2} \cdot 10 \frac{\text{N}}{\text{kg}} = 15.1 \text{ kN} \]

whereof outer rotation moment loads

\[ Q_{obal} = R \cdot 2 \cdot 2.55 \text{ m} = 77 \text{ kNm}, \quad \frac{77}{33} = 2.33 \text{ kNm/m} \]

And

\[ \tau_{max}^{obal} = \frac{1}{2} 77 = 39 \text{ kNm} \]
Additional unfavorable effect will come from the elevation of the bridge etc. In total the constructor should design the horizontal shear attachment (the corrugated sheet) at the ends of the bridge for a rotational average moment

\[ T_{\text{max}} = T_{\text{max}}^{\text{gen}} + T_{\text{max}}^{\text{tg}} + T_{\text{max}}^{\text{ab}} = 36 + 95 + 39 = 170 \text{ kNm} \]

From this a shear flow around the cross section is obtained

\[ D = \frac{T_{\text{max}}}{2A_{\text{in}}} \]

with the enclosed area of the cross section of

\[ A_{\text{in}} = 2 \frac{a+b}{2} h = 2 \frac{1.10+1.70}{2} \times 2.00 = 5.60 \text{ m}^2 \]

The shear flow

\[ D = \frac{170}{2 \times 5.60} = 15 \text{ kN/m} \]

should be resisted by the bottom sheet and the end sheets and also by the corrugated sheet to then by these four sheets be transferred to the end diaphragms of the bridge and further on to the supports.

Due to the unbalance of the wall thickness of the box cross section, by the ends of the bridge, arch moment stresses will be induced, that the corrugated sheet will be subjected to by the corners.

None of the above mentioned reasoning about the role of the corrugated sheet to be resisting rotational moment by the end of the beam is mentioned in the design calculations made by technical consultant firm 1. Attachments by the end of the beams are missing. On drawing 040 K2008 with change 2002-03-19 it is specified that “at concreting the rotational moment is limited by eccentric loading to 100 kNm”. From this one would get \( T_{\text{max}}=50 \text{ kNm} \), compared to \( 95+39=134 \text{ kNm} \) on previous page. On the original drawing dated 2002-04-25 this passage of restriction does not exist.

Observe that the above calculated shear flow 15 kN/m caused by eccentric loading (that causes torsion to the box beam) is in addition to the 19 kN/m that technical consultant firm 1 is calculating based 1.5% from the compression in the unbraced web.

Also observe that some simultaneously affect in the trapezoidal plate from shear flow due to torsion and due to bending stresses due to concrete loading occurs. The combination is impairing the safety against buckling of the trapezoidal plate.

The lack of a screwed or nailed connection between the trapezoidal plate and the end diaphragms of the bridge (an attachment like this is not specified by technical consultant firm 1 in their calculations neither at the drawings) leads to a shear flow=0 by the ends of the bridge and to a maximum value of D bigger than 15 kN/m a small distance from the ends of the bridge.
Per Olof Professor 2 2003-04-10 page 8 indicates for the sheeting TRP45 t=0.85 mm shear flow capacity \( D=13 \text{ kN/m} \) alternatively 9 kN/m. Professor 1 2003-08-15 indicates for the same sheeting a shear flow capacity of 11.3 kN/m. Corresponding value of failure should be in the range of 15-25 kN/m.

Take the design value \( D_{\text{dim}}=12 \text{ kN/m} \). How much proportion \( \lambda \) of the total concrete loading 66 m\(^3\) during the first sequence of concreting could, with the presumptions as above, be applied before a progressive failure is at risk of being initiated by both of the ends of the beam in the corrugated sheet TRP 45 t=0.85mm adjacent to the connections (four) to both of the top flanges 0.750 m ∙ 0.029 m?

\[
\tau^{\text{max}} = \tau_{\text{max}}^{\text{gen}} + \lambda (\tau_{\text{max}}^{\text{tbg}} + \tau_{\text{max}}^{\text{obal}}) = 36 + \lambda (95 + 39)
\]

where

\[
D = \frac{\tau_{\text{max}}^{\text{gen}}}{2A_{\text{in}}} = \frac{36 + 134\lambda}{2.56} = 3.2 + 12\lambda \text{ kN/m}
\]

and, with an estimated concentration factor 1.4,

\[
D_{\text{max}} = 1.4 \cdot (3.2 + 12\lambda) = 4.5 + 17\lambda \text{ kN/m}
\]

The equation \( D_{\text{max}} = D_{\text{dim}} \) gives

\[
4.5 + 17\lambda = 12 \rightarrow \lambda = \frac{13 - 4.5}{17} = 0.44
\]

The answer is that 44% of 66 m\(^3\), i.e. 29 m\(^3\) concrete, could be applied before the corrugated sheet starts giving in.

**Torsion stiffness \( GK_u \)**
Under the assumption of traditional Saint Venant torsion/rotation of a beam with a closed and undistorted cross section (box cross section) the cross section factor from the torsion stiffness $GK_v$ is given as (Bredts formula)

$$K_v = 4A_{in}^2 / \left( \frac{2b}{t_b} + \frac{2l}{t_h} + \frac{2a}{t_a} \right)$$

where the enclosing area is

$$A_{in} = 2 \frac{a+b}{2} h = 2 \frac{1.10+1.70}{2} 2.00 = 5.6 \text{ m}^2$$

From the end parts of the steel beam you get

$$K_v = 4 \cdot 2.00 \cdot \frac{5.6^2}{\frac{1.10+1.70}{2} 2.00} = 2016 \cdot 10^{-6} \text{ m}^4 = 2016 \cdot 10^6 \text{ mm}^4$$

This is a very low value. The observed beam (with an average width of 2.8 m and height 2.00 m) is according to the calculation as weak in rotation as four square pipes (VKR) 450x250x16.0 that according to catalogue has $K_v = 505 \cdot 10^6 \text{ mm}^4$. The extremely thin (equivalent thickness of 0.055 mm) top sheet is impairing $GK_v$ drastically.

The Saint Venant torsion stiffness of the beam during the first sequence of concreting is thus (G=shear modulus)

$$GK_v = 80 \cdot 10^9 \frac{N}{\text{m}^2} \cdot 2016 \cdot 10^{-6} \text{ m}^4 = 161 \text{ MNm}^2$$

Here you can compare with the Euler bending stiffness about the vertical z-axis for the same beam,

$$EI_z = 210 \cdot 10^9 \frac{N}{\text{m}^2} \cdot 0.2945 \text{ m}^4 = 61800 \text{ MNm}^2$$

The deviation in magnitude is striking and remarkable. In the calculations for the critical value of the vertical load causing instability through lateral torsional buckling of the beam, the geometrical mean value is included

$$\sqrt{EI_z GK_v} = \sqrt{61800 \cdot 161} = 3150 \text{ MNm}^2$$

that consequently is impaired with a divisor of approximately 20, since $GK_v$ is equal to 161 MNm$^2$ instead of let's say 60000 MNm$^2$. 

The Saint Venant torsion stiffness of the beam during the first sequence of concreting is thus (G=shear modulus)
The extreme difference

\[ GK_v \ll EI_z \]

could admit a simple estimation of the vertical load causing instability of the bridge beam through lateral torsional buckling.

For the box cross section, the calculated value \( K_v = 2016 \cdot 10^{-6} \text{ m}^4 \) is extremely low. If the upper wall had been a sheet with the same thickness of 22 mm as the lower sheet, one would have got

\[ K_v = 4 \cdot \frac{5.6^2}{0.022 + 0.018 + 0.022} = 258 \cdot 10^{-3} \text{ m}^4 \]

i.e. \( 258 \cdot 10^{-3} / 2016 \cdot 10^{-6} = 128 \) times bigger.

On the other hand the open cross section (without the upper sheet) has

\[ K_v = \frac{1}{3} (2 \cdot 0.750 \cdot 0.029^3 + 2 \cdot 1.10 \cdot 0.022^3 + 2 \cdot 2.09 \cdot 0.018^3) = 28.1 \cdot 10^{-6} \text{ m}^4 \]

i.e. a value \( 2016 \cdot 10^{-6} / 28.1 \cdot 10^{-6} = 72 \) times lower value.

However a torsional stiffness \( EK_v \neq 0 \) will be added to the open cross section, that for non-uniform torsion \( (\varphi' \neq 0) \) partly will compensate for the low value of \( GK_v \). Even the closed cross section has, due to the unbalance between the sheet thicknesses, some torsional stiffness.

**Differential equation for lateral torsional buckling**

Lateral load \( p_y(x) \)

Bending stiffness \( EI_z \) (about the z-axis)

Torsional stiffness \( GK_v \) (Saint Venant)

Stiffness \( EK_v = 0 \)

The cross section rotates \( \varphi = \varphi(x) \). The shear centre axis translates \( v = v(x) \) in the y-direction. The bending moment \( M_y = M_y(x) \) in the beam

1) \( EI_z v'''' + (M_y \varphi)'' = 0 \) (transverse load \( p_y = 0 \))

Integration twice + boundary conditions

\( v''(0) = v''(L) = 0 \) gives

\[ EI_z v'' + M_y \varphi = 0 \quad \text{or} \quad v'' = -\frac{M_y}{EI_z} \varphi \]
2)

\[
\left( (GK_y + M_y\psi)\varphi' \right) + ap_2\varphi + M_y\varphi'' = 0
\]

\[
\varphi(0) = \varphi(L) = 0
\]

\( a = \) level of force application

\[
\psi = -2\ddot{z} + \frac{1}{l_y} \int (z^3 + y^2z)\,dA
\]

(cross section constant)

Elimination of \( \varphi'' \) gives the sought equation

\[
\varphi''(x) + A(x)\varphi'(x) + B(x)\varphi(x) = 0
\]

with

\[
A(x) = \frac{\psi M_y'(x)}{GK_y + \psi M_y(x)} \quad \quad B(x) = \frac{ap_2(x) - M_y^2(x)/El_y}{GK_y + \psi M_y(x)}
\]

I could later, if wanted, solve this differential equation and the associated eigenvalue problem (homogeneous equation and homogenous boundary conditions) for a couple of load cases. Then you write this differential equation of second order as a system of two differential equations of first order and uses an already finished routine in for example MATLAB.

Define \( \phi(x) = \varphi'(x) \). Then you get

\[
\begin{bmatrix}
\phi'(x) \\
\varphi'(x)
\end{bmatrix} =
\begin{bmatrix}
-A(x) & -B(x) \\
1 & 0
\end{bmatrix}
\begin{bmatrix}
\phi(x) \\
\varphi(x)
\end{bmatrix}
\]

The cross section constant \( \Psi \) is easily calculated. The function of moment \( M_y(x) \) and its derivative \( M_y'(x) \) depends on applied loading. Consider for example uniformly distributed load \( p \).

\[
M_y(x) = \frac{x}{l} (1 - \frac{x}{l}) \frac{pl^2}{2}
\]

\[
M_y'(x) = (1 - 2\frac{x}{l}) \frac{pl}{2}
\]

The projections are simplified with \( \xi = x/L \). The eigen value parameter \( p \) is included in both the numerator and the denominator of A and B. Its lowest critical value is sought.

Instability load- a simple estimation
Consider an intact bridge beam with a vertical point load $P$ in the middle of the beam. This imagine load case is more dangerous than the real load case with uniformly distributed self-weight 1150 kN along the entire beam and uniformly distributed load from concrete of 1584 kN over the middle 33 meters of the length of the beam $L$. The equivalent load $P$ is estimated to the sum of

$$P_{egen} = \frac{2}{3}1150 = 767 \text{ kN} , \quad P_{btg} = 1584 \text{ kN}$$

Since $EI_x \gg GK_p$ it is reasonable to guess (on the safe side) that the deformation of the beam when overturning only consists of a rotation $\varphi = \varphi(x)$ of the cross section about the SC-axis. With $\varphi_m$ equal the middle ordinate of $\varphi(x)$ the wrecking outer rotational moment on the beam becomes

$$Q = Q_{egen} + Q_{btg}$$

with

$$Q_{egen} = P_{egen}(h - f + e_s)\varphi_m = 767(2.00 - 1.10 + 0.97)\varphi_m = 1434 \text{ kNm}\varphi_m$$

$$Q_{btg} = P_{btg}(h + e_s)\varphi_m = 1584(2.00 + 0.97)\varphi_m = 4704 \text{ kNm}\varphi_m$$

Write

$$Q = Q_{egen} + \lambda Q_{btg}$$

where $\lambda$ is the proportion of the concrete loading needed to be utilized for instability to occur, i.e. for the beam to overturn.

The rotation stiffness $S$ [Nm/rad] in the middle point $x=L/2$, is
\[ S = 2 \frac{GK_v}{L/2} = 4 \frac{GK_v}{L} = 4 \frac{1613 \cdot 10^3 \text{kNm}^2}{65 \text{ m}} = 9910 \text{kNm/rad} \]

The similarity between required and sufficient outer torsional moment \( Q \) to cause the rotation \( \varphi_m \) gives

\[ S\varphi_m = (Q_{egen} + \lambda Q_{btg})\varphi_m \]

\[ 9910\varphi_m = (1434 + 4704\lambda)\varphi_m \]

where

\[ \varphi_m = \frac{0}{9910 - 1434 - 4704\lambda} = \frac{0}{8476 - 4704\lambda} \]

The condition \( \varphi_m \neq 0 \), i.e. \( \varphi_m = 0/0 \) gives the searched

\[ \lambda_{crit} = \frac{8476}{4704} = 1.80 > 1 \]

The result is that an intact beam during the first sequence of concreting would not fail due to lateral torsional buckling. Reductions according to standards with respect to imperfections gives \( \lambda < \lambda_{crit} \). The stiffening assumption with rotation only about the SC-axis means that \( \lambda_{real} < 1.80 \). Say \( \lambda \approx 1.20 \! \) !

Flexion due to uneven sheet temperature

Assume that incident sun irradiation during the afternoon of the day of concreting has heated up one of the side sheets and the top flange, let’s say \( T = 20^\circ\text{C} \) (could be a lot more if the weather is calm) relative to the temperature of the rest of the bridge.

\[ \alpha = 12 \cdot 10^{-6} /\text{degree} \]

Elasticity modulus \( E = 210 \text{ GPa} \)
Fixed loads (in x direction) $\alpha E AT$ (A= heated cross section area) are

\[-12 \cdot 10^{-6} \cdot 210 \cdot 10^9 \cdot 0.75 \cdot 0.0070 \cdot 20 \text{ N} = -1443 \text{ kN}\]
\[-12 \cdot 10^{-6} \cdot 210 \cdot 10^9 \cdot 2.09 \cdot 0.018 \cdot 20 \text{ N} = -1896 \text{ kN}\]

where the bending moment from reverse fixed load is

\[M_y = 1134 \cdot 1.10 + 1896 \cdot 0.10 = 1437 \text{ kNm}\]
\[M_z = 1134 \cdot 1.70 + 1896 \cdot 1.40 = 4581 \text{ kNm}\]

The uneven heating results in the center deflections $f_m$ and $\theta_m$ (take the mean values of $f_f$ and $\theta_f$).

\[
\frac{1437 \cdot 10^3 \cdot 65^2}{8210 \cdot 10^9 \cdot 0.1500} = 0.024 \text{ m} = 24 \text{ mm vertically upward towards the warm top flange}
\]
\[
\frac{4581 \cdot 10^3 \cdot 65^2}{8210 \cdot 10^9 \cdot 0.3300} = 0.035 \text{ m} = 35 \text{ mm horizontally towards the warm side}
\]

If the heating is double ($T = 40^\circ\text{C}$) double deflections are maintained. Uneven heating with another distribution could possibly give a rotation of the cross section.

Existing eccentricity at the failure 2002-06-12 decides which direction the bridge rotated at?

In what direction is the bridge located relatively the evening sun?

Beyond planar bending in the zx and xy plane according to above, this studied temperature loading gives rotational stresses $\sigma_x$ and a cross section rotation if the camber of the cross section is taken in account for. The camber $u(x, s) = -\varphi'(x)w(s)$ is considered negligible in this case.

**Conclusions**

The properties of the bridge beam and its behavior during the first stage of concreting has been identified and assessed. My conclusion is that the corrugated sheet Plannja TRP $t=0.85$
mm, prescribed by technical consultant firm 1, has to low shear strength. The situation exacerbates due to the lack of attachment of the sheet against both of the end diaphragms of the bridge.

With, according to my opinion, reasonable assumptions and designed initial deformations and designed eccentricity and unbalance for the loading from concrete, I find that the corrugated sheet will start to fail in the longitudinal splices close to the ends of the beam before the entire load from the first sequence of concreting is applied. A rough estimate gives that, in the according to me designing case, approximately 44% of the concrete load would initiate failure.

Through failure as mentioned above at the ends of the beam, the beam in total loses its rotational stiffness fast and will be subjected to lateral torsional buckling. I find that an intact bridge beam would not have been subjected to this even after the entire loading from the concreting would have been made.

My answer to Anders Reldén’s question four is:

“The failure 2002-06-12 would have occurred regardless of the changes made by construction company 1 referring to changes of sheets and their attachments”.

8. Technical consultant firm 1 have through technical consultant 2 got the mission for undersigned to give an announcement about the failure of the Y1504 Bridge over Gide älv.

Three of my valued colleagues (Professor 1, Professor 2 and Professor 3) have through calculations tried to explain the failure and reached different results. All three of them well know about the theories applicable and I have not noted any severe errors even though details may be discussed or are not presented. The calculations by Professor 3 are very pedagogically presented and easy to follow, but does not treat the properties for the critical plate. For its properties he uses the results from Professor 1, which I find more credible than the ones from Professor 2. Other than that I have no reason to question the calculations, but only the assumptions they are based on.

The reason to the separated results are that they have used different assumptions about measurement deviations and circumstances at the time of failure. It will not bring the issue forward if I make my own calculations. Instead I will use what has already been made and make, if needed, corrections of the assumptions.

General comments

construction company 1 built the bridge for SRA and hired Technical consultant firm 1 to plan according to agreement. There is nothing unusual in the agreements and especially for the construction stage it says that “separate working instructions” is included in the mission. These are often quite limited and usually include to forward demands from the contractor and to present assumed casting order. In the current case this is presented on a drawing with a normal level of detailing. Concerning the critical trapezoidal plate there is a reference to a separate working instruction. This is in connection to detail D20 on the same drawing. A connection between end diaphragms and the plate is predicted but an oral explanation from the constructor Peter Collin, says that that was not his intention. His intention was that the forces was to be transferred to the diaphragms through the attachments in the top flanges. The calculations made by Professor 1 shows that the connection would have lasted but I would in the future not recommend a bar welded to the flanges for attachment to the plate.

Since the work was made on a turnkey contract you have to find technical solutions and working methods that leads to intended quality to the smallest cost. A turnkey contract is meant to promote innovations but the way the building industry works today the development is not progressing systematically but in connection to other projects. This means that the budget for the development shall fit in to the project budget and it will therefor become small. The used solution with trapezoidal sheets as permanent form work and stabilizing element is innovative and was as far as I know first used by Tore Lundmark at Ramböll, Luleå year 1992 and has after that used successfully in a lot of different projects as well by Ramböll as of other constructors. One is used to that the solution is working and have not seen any reason for any deeper analysis. The bridge over Gide älv was longer than earlier bridges and had essential deviations from the drawings made for the constructing. Even construction company 1 was used to that is was usually working and saw the possibilities in saving money, without being aware of the consequences. With this I would like to point out that technical development is
and will always be associated with failure. I don’t agree with Professor 3 that says that the construction “invites to failure”. With a better analysis the stabilizing trapezoidal sheet would probably have been made stiffer but even with the drawn solution it would have been possible to build the bridge, which I will get back to. If I also should moralize, I think it’s a pity that you don’t invest money on systematically product developing in the building industry instead of replacing failed bridges and argue about who will pay.

Would it have been possible to build the bridge?

The critical question is if the bridge would have been possible to build if the drawings would have been followed and on that question, the performed calculations have given different results. For the real performance both reality and the calculations has shown that the bridge would fail. This happened when the bridge was loaded with a small part of the concrete. My assessment is that the failure not would have been avoided with the real performance how precise they had tried to balance the concrete during the continued casting. This is indicated through that the critical load for lateral torsional buckling was too low as a consequence to the by construction company 1 chosen trapezoidal plate and its attachment. The meaning of this change is of interest for the liability issue and a way to describe it is through the change of the rotational stiffness of the box. The rotational stiffness is the parameter with the greatest significance for the stability of the bridge during the casting. The rotational stiffness for the chosen performance is 1/3 to 2/3 of the drawn performance. The lower value is from Professor 1 and the higher from Professor 2. The lower value is probably closer to reality since it is based on more accurate calculations. With the lower value, the lateral torsional buckling load is reduced to 60% of the one for the drawn performance. Another measurement of the significance is the decrease of the resistance of shear force in the side over-lap, which was critical for the failure. The resistance in the chosen performance is 32 to 43 % of the drawn. Again the lower value is from Professor 1 and the difference is conditioned by the fact that Professor 2 makes a debatable reduction of the thickness for the plate in the drawn case. That plate has never been realized and its thickness is not familiar. It is then reasonable to assume that is has predicted computational thickness.

For the drawn performance, the calculations by Professor 3 shows that the bridge would fail. He has then amongst other things assumed that the bridge has a side-way crookedness of 98 mm. In the drawing it is given a tolerance of side-ways crookedness of 20 mm. According to the report by technical consultant 1 dated 2002-08-12, the straightness of the steel box was within given tolerances and there is no reason to assume a bigger crookedness for the analysis of the failure. With this change, calculated shear flow for the trapezoidal plate on page 7 in the report made by Professor 1, dated 2004-08-10 6.3 kN/m instead of 15 kN/m. Further, Professor 3 means that in addition to this shear flow, what is required for bracing the top flange of the box according to the construction documents shall be added. The calculation of the bracing force in the construction calculations relates to a case where the flange only is braced in one point and not as is the current case where it is continuously braced. With 20 mm of crookedness in the shape of an arc and a constant pressure, a shear flow in the plate of 3.1 kN/m is obtained. Together 9.4 kN/m is obtained, which is less than the resistance that is assumed to be 12 kN/m in the report by Professor 3, dated 2004-08-10. Further in the same report, the shear flow is given to increase close to the ends of the bridge since the trapezoidal plate is not attached to the end diaphragms. The increase is estimated to be 1.4 times, but it is not told why the increase should occur. I’m having a hard time telling the reason and the
computer calculation in the report by Professor 1, dated 2002-11-08 is not showing this increase. Instead, normal forces arise in the trapezoidal plate that causes an increase of the forces in the longitudinal direction of the plate on its attachments to the flanges. It is shown in the report by Professor 1 that the forces can be absorbed.

In the reports made by Professor 2, dated 2003-04-11, by Professor 1, dated 2003-08-15 and by Professor 2, dated 2004-04-28 there is a discussion about the wind load. Professor 2 and Professor 1 agree that at the time of failure the wind is so low that it could not have contributed to the failure. In the report by Professor 3, dated 2004-08-10 there is a discussion about the impact from the uneven temperature. The bridge is located in bearing 60° and is therefore about parallel with the afternoon sun. It is obvious in the report from Technical consultant firm 1, dated 2002-07-06 that the casting started in the afternoon and that the sun was shown when they started casting the concrete. From this a conclusion can be made that uneven temperature did not affect the failure.

Above it is clear that it should have been possible to build the bridge if the drawings were followed and if the concrete would have been placed reasonably centric. In the calculations in the report by Professor 3, 2004-08-10 a rotational moment due to unevenly applied concrete of about 77 m$^3$ is taken in account for, which is a part of the calculated shear flow above, of 9.4 kN/m. The characteristic resistance is with respect to the side joint screws 11.3 kN/m and before it reaches the rotational moment load could be increased with 40 kNm. It should here be noted that the calculations from Professor 3 does not take in account for any second order effects to the fullest, but on the other hand it doesn’t either take in account for the contribution from the top flanges to the resistance of the plate for forces in its plane. If we suppose that these effects evens each other out, the required rotational moment force to reach characteristic resistance corresponds to for example 6 m$^3$ of concrete with 1 meter eccentricity. It should have been possible to limit the unbalance to less than this already with a perceptiveness and if you further had a levelling instrument on site.

So why did Professor 2 come to the conclusion that the bridge would have failed even though the performance was made according to the drawings? In the report by Professor 2, 2003-04-11 a calculation is made based on the basis of the time of failure but with the right trapezoidal plate and attachments. It is based on a side-way crookedness of about 32.5 mm instead of 20 and a too low characteristic resistance for the side over-lap screws. In the report by Professor 1, 2003-08-15, Professor 1 says that the latest mentioned and shows that with the right value, the resistance should not have been exceeded according to his calculations. In the report by Professor 2, 2004-04-28, Professor 2 is correcting the resistance but contains the assumption of the too big crookedness and the bridge just fails. Also, a calculation with wind leading to failure for a lower concreting load is presented. Professor 1 and Professor 2 is getting closer to each other when it comes to the analysis of the failure, but because of different assumptions it still remains that they come to different conclusions.

The difference between the results in the different calculations depends on different assumptions. If the calculations were corrected to the current conditions at the failure they show, with small variations, that the casting would have been possible to proceed with and be fulfilled and that the side over-lap screws then should have been used almost to their characteristic resistance. For the interpretation of this I would like to remind you of what the characteristic resistance mean. The characteristic resistance is decided from attempts so it is
corresponding to the 5% fractile of the ultimate load from a great number of attempts. It means that if a hundred of screws are loaded with the characteristic resistance once at a time, 5 will break and 95 will not break. For the current case, failure in the border of the screw holes, this definition has not been applied strictly but the resistance have been made lower with respect to the deformation limitation. Here this is ignored and assumed that it is a 5% fractile for each screw. In one side over-lap there is however 19 screws working together. If you assume that the variation coefficient in the resistance in 0.1, then the 5% fractile becomes 13.0 kN/m instead of 11.3 kN/m which is used above. It is allowed that this is not often considered at design but it is a factor making it more likely for the bridge not to fail.

The loads in this case are well known with good accuracy. The insecurity lays within the eccentricity of the concreting load. As already mentioned it would have been possible to keep the eccentricity that low, so that the casting would have been possible to fulfil. This assumes that the contractor would have been aware of the meaning of keeping the eccentricity down. From the description in the report by technical consultant 1, 2002-08-12, that seems to have been the case. How big the eccentricity was at the failure is not known and even less how big it would have been if the bridge was performed according to the drawings.

Conclusion

The reason of the failure is according to me, lack of communication at several occasions. The purchaser of construction company 1 was not aware of the meaning of the trapezoidal plate. The purchaser should on the contrary have asked Technical consultant firm 1 if the change was OK. The plate supplier should have understood that that the close screwing of the side over-lap joint had to do with diaphragm action and not have recommended a sufficiently weaker screwing without asking the constructor. It didn’t get any easier when the plate was delivered too thin. The impact of this change was devastation and in this context the responsibility lies on construction company 1. They changed a great utilized construction to one that was impossible to build.

In the end it is a legal assessment of the responsibility issue that is not for me. I believe that the decision from the contractor to change the plate and its screwing was decisive for the failure. Without this change the bridge would have been possible to build, even though with a very low safety.

Finally I would like to agree with Professor 3 that we shall not do this again. On the other hand I think that the construction solution is cost-effective and it can be done stiffer and stronger by small means.
9. Failure of Y1504 Bridge over Gide älv in Kärrsjö

Would the bridge have failed if it was performed according to the drawings?

In a letter 2004-01-12 from Anders Relden, White & Case Advokat AB it is questioned if the bridge would have been as crooked as it was if it was performed according to the drawings made by Technical consultant firm 1 s.

In my calculations 2002-11-08 on page 19 the side deflection of the bridge after the first stage of concreting has been calculated to \( u = 28.7 \) mm. The level difference between the flanges is not written out but it is 31.9 mm. The level difference and the side deflection are based on an eccentricity for the concreting of 200 mm. One can see that according to the assessment of Professor 3 this eccentricity is on the high side.

Spread out over the width of 3.4 meters this is not that much considering that the roadway in the middle of the field is crowned with 2.5% tilting from the middle.

Spånga 2005-02-02

Professor 1
Appendix II
Calculations of required stiffness according to Eglimez et al.

The required shear stiffness $G'$ for metal deck to work as lateral bracing is decided according to calculations below.

\[ G' = \frac{4(M_u - C_b \cdot M_g)}{m \cdot s_d \cdot d} \]

where

\[ M_u = 19.2 \text{ MNm} \]

\[ C_b = 1.14 \]

\[ C_b^* = \frac{C_b}{1.4} = \frac{1.14}{1.4} = 0.814 \]

\[ M_g = \pi \frac{l}{L} \sqrt{E I_y G J + \frac{\pi^2 E^2 I_y C_w}{L_b^2}} \]

where

\[ L = 65 \text{ m} \]

\[ E = 210 \text{ GPa} \]

\[ I = 274354 \cdot 10^6 \text{ mm}^4 \]

\[ G = 81 \text{ GPa} \]

\[ j = \sum \frac{t^3 h}{3} = 26.39 \text{ mm}^4 \]

\[ C_w = 9.481 \cdot 10^{16} \text{ mm}^6 \] (according to Professor 1 for closed cross section)

\[ L_b = 7.2 \text{ m} \] (average distance between diaphragms)

\[ m = \text{(with intermediate bracing, } \frac{h}{t_w} = 116 \text{ and top flange loading)} = 0.375 \]

\[ s_d = \frac{n_g^{-1}}{n_g} \cdot s = \frac{11-1}{11} \cdot 6500 = 5909 \text{ mm} \]

\[ d = 2000 \text{ mm} \]

\[ G' = \frac{4(M_u - C_b \cdot M_g)}{m \cdot s_d \cdot d} = \frac{4(19.2-0.814\cdot16.95)}{0.375\cdot5909\cdot2} = 4876 \text{ kN/m} \]
Appendix III
Calculations of strength requirements on fasteners according to Eglimez et al.

Forces acting on fasteners

\[ M'_{br} = 0.001 \frac{M_{ul}}{d^2} = 0.001 \cdot \frac{19.27}{2^2} = 33.6 \text{kNm/m} \]

\[ F_{V,P1} = \frac{V_{br}}{n_e} = \frac{2M_{br}}{n_e L_d} = \frac{2M_{br} \cdot w_d}{n_e L_d} = \frac{233.6 \cdot 0.75}{4 \cdot 2.85} = 4.42 \text{kN (PEVA45 with one fastener per profile bottom)} \]

\[ F_{V,P2} = \frac{V_{br}}{n_e} = \frac{2M_{br}}{n_e L_d} = \frac{2M_{br} \cdot w_d}{n_e L_d} = \frac{233.6 \cdot 0.75}{8 \cdot 2.85} = 2.21 \text{kN (PEVA45 with two fasteners per profile bottom)} \]

\[ F_{V,T} = \frac{V_{br}}{n_e} = \frac{2M_{br}}{n_e L_d} = \frac{2M_{br} \cdot w_d}{n_e L_d} = \frac{233.6 \cdot 0.95}{5 \cdot 2.85} = 4.48 \text{kN (TRP45 with one fastener per profile bottom)} \]

\[ F_{M,P1} = M'_{br} \cdot \frac{1}{1.11} = \frac{33.6}{1.11} = 30.27 \text{kN} \]

\[ F_{M,P2} = M'_{br} \cdot \frac{1}{1.5} = \frac{33.6}{1.5} = 22.4 \text{kN} \]

\[ M_{br} = F_{M,T} \cdot \frac{w_d}{2} + F_{M,T} \cdot \frac{w_d}{2} + F_{M,T} \cdot \frac{w_d}{4} + F_{M,T} \cdot \frac{w_d}{4} = 1.25 F_{M,T} \cdot w_d \rightarrow \]

\[ \rightarrow F_{M,T} = \frac{M_{br}}{1.25 \cdot w_d} = \frac{M'_{br}}{1.25} = \frac{33.6}{1.25} = 26.88 \text{kN} \]

\[ F_{tot,P1} = \sqrt{F_{V,P1}^2 + F_{M,P1}^2} = 30.6 \text{kN} \]

\[ F_{tot,P2} = \sqrt{F_{V,P2}^2 + F_{M,P2}^2} = 22.5 \text{kN} \]

\[ F_{tot,T} = \sqrt{F_{V,T}^2 + F_{M,T}^2} = 27.3 \text{kN} \]
Appendix IV

Calculations of stiffness according to SDI

TRP45

Units according to American system of measuring

- $e = 1.57$ in
- $f = 1.57$ in
- $d = 7.48$ in
- $w = 2.17$ in
- $h = 1.68$ in
- $t = 0.0297$ in

The shear stiffness $G'$ is decided according to calculations below

$$G' = \frac{Et}{2.62^2 + 6D_d + C}$$

where

- $E = 29500$ kip/in$^2$
- $s = 2e + 2w + f = 3 \times 1.57 + 2 \times 2.17 = 9.07''$
- $\varnothing = 1$

$$C = \frac{24 \cdot E_t \cdot L \cdot S_f}{a} \left( \frac{n_{sh} - 1}{2 \alpha_1 + n_p \alpha_2 + 2\alpha_3} \frac{S_f}{S_s} + \frac{1}{2 \alpha_1 + n_p \alpha_2 + n_e} \right)$$

where

- $L = 9.51'$
- $S_f = 1.3 \cdot 10^{-3}/t^{0.5} = 7.54 \cdot 10^{-3}$ in/kip
- $S_s = 3.0 \cdot 10^{-3}/t^{0.5} = 17.41 \cdot 10^{-3}$ in/kip
- $a = 2559''$
- $n_{sh} = \frac{L}{w} = 69$
- $n_p = n_e = 0$
- $\alpha_1 = \frac{\sum X_e}{w_{sh}}$

where

$$\sum X_e = 44.88$$
\( w_{sh} = 37.40 \text{ in} \)

\[
\alpha_1 = 1.20
\]

\( n_s = 20 \)

\[
C = \frac{24 \cdot 29500 \cdot 0.0297 \cdot 9.51 \cdot 7.54 \cdot 10^{-3}}{2559} \left( \frac{68}{2 \cdot 1.2 + 2 \cdot 20 \cdot \frac{7.54}{17.41}} + \frac{1}{2 \cdot 1.2} \right) = 2.39
\]

Calculation of \( D_n \)

\( WT = 4f^2(f + w) = 37.18 \)

\( WB = 16e^2(2e + w) = 211.22 \)

\( PW = 1/t^{1.5} = 195.13 \)

\( A = 2e/f = 2 \)

\( V = 2(e + w) + f = 9.07 \)

\( D1 = \frac{1}{3} h^2(2w + 3f) = 8.53 \)

\( D2 = D1/2 = 4.27 \)

\( D3 = \frac{h^2}{12d^2} [V(4e^2 - 2ef + f^2) + d^2(3f + 2w)] = 2.42 \)

\( C1 = 1/(D3 - D2/2) = 3.52 \)

\( C2 = 1/\left( e \left( \frac{D2}{f} \right) + D3 \right) = 0.15 \)

\( C4 = A/[e(D1/f) + D2] = 0.16 \)

\( D4(1) = (24f/C1) \left( \frac{C1}{WT} \right)^{0.25} = 5.95 \)

\( D4(2) = (24f/C2) \left( \frac{C2}{WT} \right)^{0.25} = 63.63 \)

\( D4(4) = (48e/C4) \left( \frac{C4}{WB} \right)^{0.25} = 79.79 \)

\( G4(1) = D4(1) = 5.95 \)

\( G4(2) = 2[D4(2)] + A[D4(4)] = 286.84 \)

\( DW1 = G4(1)(f/d)PW = 244.41 \)

\( DW2 = G4(2)(f/2d)PW = 5891.87 \)

\( D1 = \frac{244.41}{12L} = 2.14 \)

and \( D2 = \frac{5891.87}{12L} = 51.6 \)

\( G1' = \frac{29500 \cdot 0.0297}{2.6 \cdot 9.07 + 2.14 + 2.39} = 114.2 \text{ kips in} = 19994 \text{ kN/m} \)

and \( G2' = \frac{29500 \cdot 0.0297}{2.6 \cdot 9.07 + 51.6 + 2.39} = 15.3 \text{ kips in} = 2687 \text{ kN/m} \)
PEVA45
Units according to American system of measuring

\[ e = 1.64 \text{ in} \]
\[ f = 1.77 \text{ in} \]
\[ d = 7.38 \text{ in} \]
\[ w = 2.92 \text{ in} \]
\[ h = 1.77 \text{ in} \]
\[ t = 0.028 \text{ in} \]

The shear stiffness \( G' \) is decided according to calculations below

\[ G' = \frac{E \cdot t}{2.6 + \phi \cdot \rho + c} \]

where

\[ E = 29500 \text{ kip/in}^2 \]
\[ s = 2e + 2w + f = 10.90'' \]
\[ \phi = 1 \]

\[ C = \frac{24 \cdot E \cdot t \cdot L \cdot S_f}{a} \cdot \left( \frac{n_{sh}^{-1}}{2 \alpha_1 + n_p \alpha_2 + 2n_s \frac{S_f}{S_s}} + \frac{1}{2 \alpha_1 + n_p \alpha_2 + n_\epsilon} \right) \]

where

\[ L = 9.51' \]
\[ S_f = 1.3 \cdot 10^{-3} / \epsilon^{0.5} = 7.72 \cdot 10^{-3} \text{ in/kip} \]
\[ S_s = 3.0 \cdot 10^{-3} / \epsilon^{0.5} = 17.82 \cdot 10^{-3} \text{ in/kip} \]
\[ a = 2559'' \]
\[ n_{sh} = \frac{I}{w} = 68 \]
\[ n_p = n_\epsilon = 0 \]
\[ \alpha_1 = \frac{\sum X_e}{w_{sh}} \]

where

\[ \sum X_e = 30.75 \text{ in} \]
\[ w_{sh} = 29.53 \text{ in} \]

\[ \rightarrow \alpha_1 = 1.04 \]
\[ n_s = 9 \]
\[ C = \frac{24 \cdot 29500 \cdot 0.0283 \cdot 9.51 \cdot 7.72 \cdot 10^{-3}}{2559} \cdot \left( \frac{86}{2 \cdot 1.04 + 2 \cdot 9 \cdot 7.72/17.82} + \frac{1}{2 \cdot 1.04} \right) = 5.31 \]

Calculation of \( D_n \)

\[ WT = 4f^2(f + w) = 58.92 \]
\[ WB = 16e^2(2e + w) = 268.39 \]
\[ PW = 1/t^{1.5} = 209.53 \]
\[ A = 2e/f = 1.86 \]
\[ V = 2(e + w) + f = 10.9 \]
\[ D1 = \frac{1}{3}h^2(2w + 3f) = 11.65 \]
\[ D2 = D1/2 = 5.83 \]
\[ D3 = \frac{1}{12}h^2 \left[ V(4e^2 - 2ef + f^2) + d^2(3f + 2w) \right] = 3.34 \]
\[ C1 = 1/(D3 - D2/2) = 2.36 \]
\[ C2 = 1/ \left[ e \left( \frac{D2}{f} \right) + D3 \right] = 0.11 \]
\[ C4 = A/[e(D1/f) + D2] = 0.11 \]
\[ D4(1) = (24f/C1) \left( \frac{C1}{WT} \right)^{0.25} = 8.07 \]
\[ D4(2) = (24f/C2) \left( \frac{C2}{WT} \right)^{0.25} = 78.03 \]
\[ D4(4) = (48e/C4) \left( \frac{C4}{WB} \right)^{0.25} = 101.0 \]

\[ G4(1) = D4(1) = 8.07 \]
\[ G4(2) = 2[D4(2)] + A[D4(4)] = 343.46 \]
\[ DW1 = G4(1)(f/d)PW = 405.66 \]
\[ DW2 = G4(2)(f/2d)PW = 8635.89 \]

\[ D_1 = \frac{405.66}{12L} = 3.55 \]

\[ D_2 = \frac{8635.89}{12L} = 75.64 \]

\[ G_1' = \frac{29500 \cdot 0.0283}{2.6 \cdot 7.318 + 3.55 + 5.31} = 114.2 \text{ kips/in} = 11526 \text{kN/m} \]
and $G'_2 = \frac{29500 \cdot 0.0283}{2.6 \cdot 10.9 + 75.64 + 5.31 \text{ in}} = 15.3 \frac{\text{kips}}{\text{in}} = 1727 \text{ kN/m}$