Monitoring of a Swedish Integral Abutment Bridge

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Abstract

One of the most commonly discussed problems regarding bridges with integral abutments is the influence of longitudinal elongation of the superstructure as a result of seasonal temperature variations. A bridge built with integral abutments is often supported by a row of piles made of steel or concrete. The longitudinal elongation of the superstructure induces a displacement and a rotation at the top of the pile, which in turn may cause strains that exceeds the yield strain. Such seasonal variations may lead to low-cyclic fatigue failure in the pile. Therefore, it is of great interest to investigate the amplitude of these strains, as well as the general behaviour of the bridge. In 2005, the European R&D project, INTAB (RFSR-CT-2005-00041, "Economic and Durable Design of Bridges with Integral Abutments, 2005–2008") was started. Within the INTAB project a composite bridge was built and monitored in Northern Sweden.

Keywords: integral abutments; steel piles; composite bridges; monitoring; live load testing.

Introduction

The cost of maintenance is an ever-growing problem for road administrations around the world, and bridges are no exception. One way to reduce the need for future maintenance, as well as the investment cost, is to construct bridges without expansion joints and bearings; which in this paper is referred to as integral abutment bridges (see *Fig. 1*).

Conventional bridges are in general built with expansion joints and bearings, which can both be considered weak points in the bridge structure. Leaking joints are a common reason for corrosion problems in bridges. These joints need to be maintained, repaired and also often replaced several times during the service lifetime of the bridge. Therefore, if bridges are built without any expansion joints, it is possible to reduce the maintenance cost.

Integral abutment bridges have other benefits besides lower maintenance costs: there will be no expenditure on purchase of expansion joints and bearings. The foundation works can be simplified, which would result in lower construction costs. Furthermore, a shorter construction time saves money not only for the contractor but even more for the road users, a fact that is becoming increasingly important. No expansion joint also means less noise and higher comfort when a car enters or leaves the bridge.

As the piles are connected to the retaining walls of the bridge, the piles will follow the bridge's deformation with respect to both rotations and translations as a result of seasonal as well as daily temperature changes (see *Fig. 2*). This is also the case for deformation caused by traffic on the bridge. Hence, in order to understand the mechanisms of integral abutment bridges, it is necessary to study the effect that movements in the abutment have on the stresses in the piles.

Analysing the stresses in the piles subjected to lateral movements is complex as it contains two co-dependent elements: the flexural pile and the soil. To further complicate matters, soils are often inhomogeneous. It is possible to obtain analytical solutions only for simple cases where the stiffness of the soil is constant along the pile, and the materials feature elastic behaviour. Expressions of the cases with constant or linear varying soil stiffness are given by the theories for beams on an elastic foundation. To handle more complex cases where soil stiffness varies with depth, an equivalent stiffness can be assumed.

In 2005, the European R&D project INTAB was started.[1] In May 2006, an international workshop on integral abutment bridges with participants from eight countries was organized by invitation and held in Stockholm.[2] The goal of the workshop was to share the experiences of the participants and to further increase the understanding of the design, construction, and maintenance of integral abutment bridges. During the workshop, it became clear that various approaches to the design of integral abutment bridges exists in different
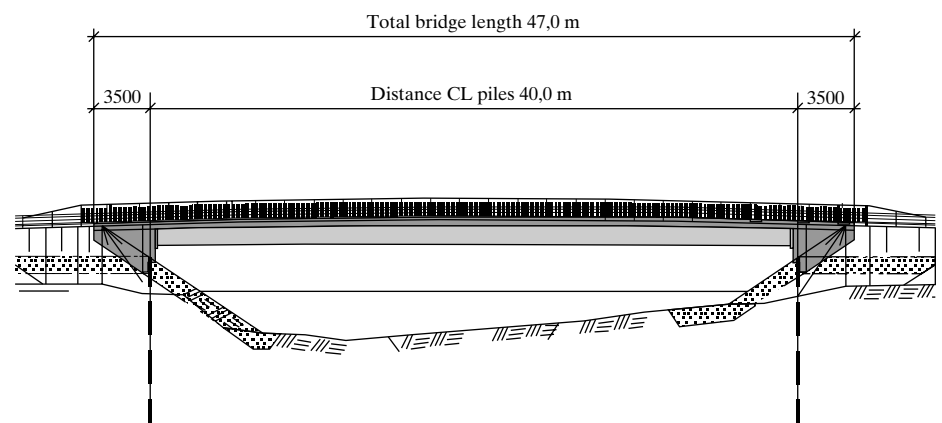

Fig. 1: The integral abutment bridge over Leduån (Units: m)

Displacement caused by seasonal temperature changes
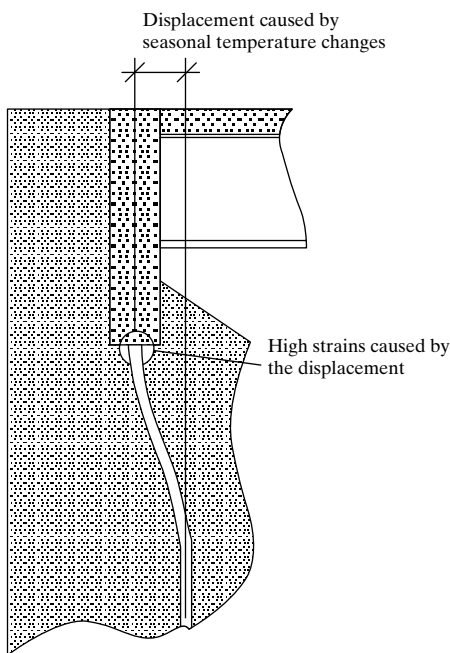
High strains caused by the displacement

Fig. 2: Pile deformation due to seasonal temperature variation induces strains in the steel pile

countries. Results of a European survey to illustrate the design criteria were compared to the best practice of transportation agencies in the United States.[3] There are many similarities in design assumptions and construction practices. Yet, there are also significant differences, especially regarding limits for the bridge length. Furthermore, the type of piling as well as the detailing of bridges differs much, from country to country.

Bridge over Leduån

Presently, there are no specific design rules existing for integral abutment bridges in Sweden and most European countries. In the future, bridges will be designed according to the Eurocodes. A common design criterion for the serviceability limit state is to restrict the allowable stresses to the yield limit (see EN 1993-2 7.3). This will curb the possibility to build longer integral abutment bridges founded on steel piles. EN 1990 3.4 states that "The limit states that concern: (a) the functioning of the structure or structural members under normal use; (b) the comfort of people; (c) the appearance of the construction works, shall be classified as serviceability limit states". None of these criteria are violated because of the fact that the pile experiences some inelastic strains. Some might consider the appearance of construction work as violated. But the part is hidden under the ground surface and will not make anyone using the bridge uncomfortable, and

it therefore need not be considered a violation of the rule. A structure's functionality can be endangered during normal use of piles only if cracks were to form in the steel piles. The bending strains in the piles are not necessary for the bridge to transfer the loads to the soil and would virtually vanish in a hinged pile, but a mechanism of this kind is both more expensive and susceptible to frequent maintenance problems. A pile joint without a hinge mechanism is therefore preferable in practice.

In order to investigate the stresses in the piles, the bridge behaviour as well as the accuracy of design models, a bridge was instrumented and monitored (see *Fig. 3*). The influence of both seasonal temperature variations and short-term traffic loads were studied.

Design and Construction

The bridge over Leduån is a 40 m single span integral abutment bridge. The composite superstructure comprises two I-shaped welded steel beams, and a one lane slab of concrete with characteristic compressive strength f_{ck} = 40 MPa (C40/50) (see *Fig. 4*). The superstructure is supported by end-bearing piles of steel pipe RR170 × 10 in S440 grade. Six piles are placed in a single row under each abutment. The piles were driven into the soil along a straight line perpendicular to the longitudinal axis of the bridge. The bearing piles were sheltered to a depth of 2 m under the lower end of an abutment by wider steel pipes of diameter 600 mm. Styrofoam plates were inserted inside the sheltering pipes and the remaining space was filled with loose

sand. A detailed description is given in the works of Nilsson[4] and Hällmark.[5]

The following sequence was used when constructing the bridge in order to reduce the effects of thermal movements on fresh concrete and to control moments induced into the supporting pile system:

1. The soil was excavated down to a level at which soft soil was wanted.
2. The piles were driven down in the ground.
3. The sheltering steel pipes and styrofoam plates were placed. The space between the steel piles and the steel pipes were filled with loose sand (see *Fig. 5*).
4. Blasted rock was filled around and up to the level of the top edge of the steel pipes, and then compacted.
5. The pile caps were poured to the required bridge seat elevation. Temporary bearings were installed.
6. The wing-walls were poured (see *Fig. 6*).
7. The beams were set and anchored to the abutment on temporary bearings. The bearings allow wide room for further dead load rotations.
8. The bridge deck was poured in the desired sequence, excluding the abutment retaining wall and the last portion of the bridge deck of length equal to the retaining wall width. In this manner, all dead load slab rotations occurred prior to locking the superstructure to the abutment, and no dead load moments were transferred to the supporting piles.
9. The retaining walls/pile caps were poured to full height. As no backfilling had yet been placed at this point, the abutment was free to move.
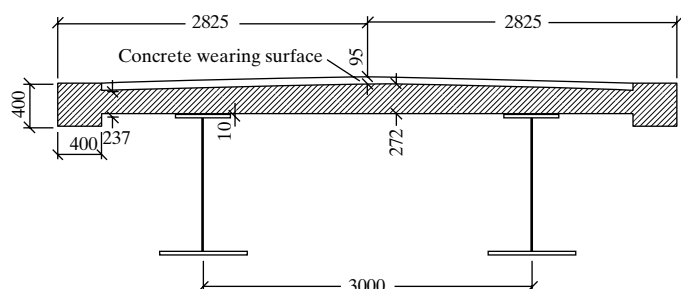

Fig. 3: The bridge over Leduån in Northern Sweden

Fig. 4: Cross section of the bridge (Units: m)

Fig. 5: Piles have been driven. The pile to the right was later used for monitoring. The picture shows the loose sand and styrofoam plates that were used to achieve a less stiff surrounding for the upper part of the piles

Fig. 6: The bridge superstructure before it was launched in place

10. The soil behind the retaining walls was filled and compacted simultaneously behind both abutments of the bridge.

Instrumentation of the Bridge

The bridge was monitored during a period of 18 months. Totally 34 gauges were placed on the bridge as shown in *Figs. 7 and 8*. Strain-gauges were welded to the bridge girder and to the piles. The strains in the piles were measured at five different levels with two strain-gauges at each level as shown in *Fig. 8*. The two strain-gauges in each section (position 5–9 in *Fig. 7*) were oriented and placed in such a way that the maximum strains of the cross section could be measured. At the upper four levels, the difference in strain between two strain-gauges was stored (bending strain). Signals obtained from both pairs of strain-gauges at the fifth level were stored separately and an estimation of the axial force in the pile was made (*Table 2*).

The movement of the retaining walls was measured with level indicators, two at each side of the bridge (gauges 3 and 4 in *Fig. 7*). The level indicators were placed in a vertical plane along the centre line of the bridge at a vertical distance of 1,5 m between gauges on each abutment. With a known geometry of the abutment, rotation and displacement of the pile cap could be estimated from these measurements. Two strain-gauges (gauges 1 and 2 in *Fig. 7*) were welded at the steel girders' flanges, close to the south abutment, to get an indication of the moment constraint obtained at the bridge end. Strain-gauges were also welded to the upper and lower flanges at the mid-span of the bridge (position 10 and 11 in *Fig. 7*) for an estimation
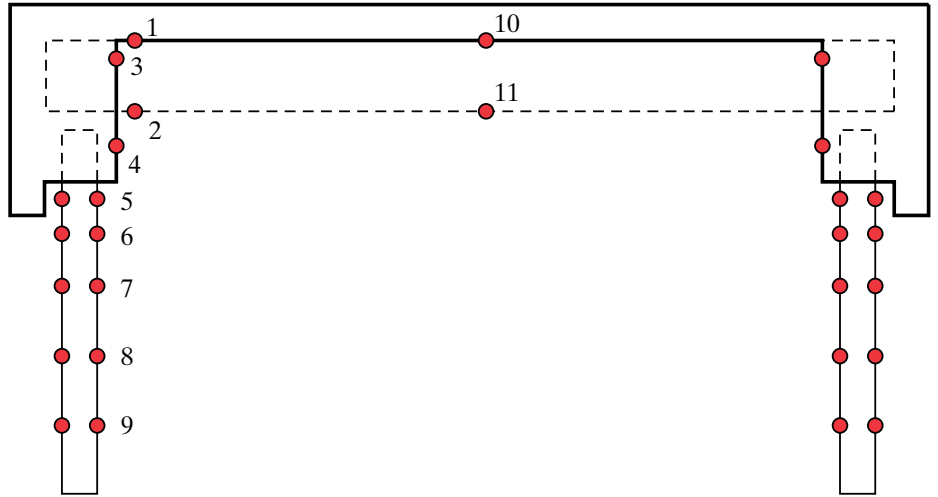

Fig. 7: A sketch showing the gauges that were used during the monitoring of the bridge

Part	Dimensions (mm)	Material
Upper flange	25 × 500/25 × 600	S460M
Web	13 × 1221/11 × 1234	S355J2G3
Lower flange	36 × 800/40 × 800	S460M

The length of each end part is 11,2 m and the middle part is 18 m. The height of the steel girder varies from 1073 to 1299 mm.

Table 1: The dimension of the bridge girders, end part/middle part

Gauge number according to *Fig. 4*	
1	Strains in upper flange at support
2	Strains in lower flange at support
3 and 4	Horizontal displacement of retaining wall, gauges placed on both abutments
5–8	Bending strains in the pile were measured with two gauges and the differences were recorded; strain-gauges were placed on one pile in the northern and one in the southern abutment
9	Strains in the pile measured with two gauges, signals from both were recorded; strain-gauges were placed on one pile in the northern and one in the southern abutment
10	Strains in top flange at mid-span
11	Strains in bottom flange at mid-span

Table 2: Explanations of what the gauges are used for

of the overall bridge behaviour. Temperature was measured at three locations in the concrete slab, in the steel flange and in the air. Detailed accounts of the gauges are given in the work by Nilsson.[4]
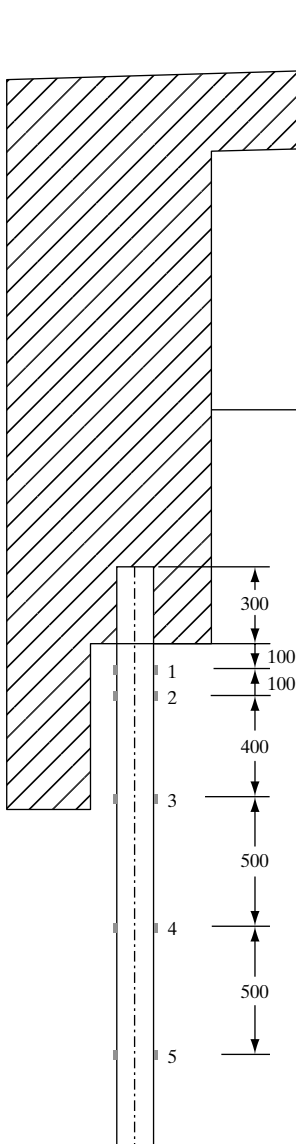
Fig. 8: Two strain-gauges were placed at five positions along the piles. For the upper four gauges the difference in strain between the two gauges was recorded (Units: mm)

Results from Monitoring

Long Term Monitoring

During the 18 months of monitoring the sampled data from strain measurements at the northern pile and the southern pile show a similar trend of variation. However, a clear difference in amplitudes exists. The maximum measured bending strain amplitudes are 881 μ-strain for the northern pile and 518 μ-strain for the southern pile (difference between two gauges in one section; see *Fig. 9*). With the assumption that Young's modulus is 210 GPa the corresponding bending stresses are ±93 and ±54 MPa for each pile, respectively. The measured temperature variation in the concrete deck was 43°C, which is much lower than the expected 50 years' maximum of 80°C (*Table 3*) according to Bro 2004.[6] Such a difference has to be statistically expected in a case where measurements are available only for a limited time period.

Short Term Monitoring and Comparison with Finite Element Analysis

Finite element analysis (FEA) was used to interpret the results of short-term measurements. The short-term measurements were made approximately every three months. A lorry with a maximum mass of about 25 t was used as test load (see *Fig. 10*). The strains were also measured on the bridge before loading, and after the

truck loading of the bridge. It was thus possible to get the strains due to just the truck load.

The soil characteristics were estimated by calibrating results of a simple two-dimensional finite element (FE) model with data from short-term measurements. A limited geotechnical investigation was made in-site and used to check the credibility of FEA.

Springs were used to model the support behind the retaining as well as the effects from the soil surrounding the piles (*Table 4*).

Recommendations from BRO 2004[6] were used as the starting value for the definition of the spring stiffness.

The spring stiffness, k, is given as:

$$k = k_k \cdot A_{\text{spring}} = k_k \cdot d \cdot s \qquad (1)$$

where A_{spring} is the projected pile-soil contact area related to one spring, d is the outer pile diameter, s is the distance between two springs and k_k (MN/m^3) is the sub-grade reaction modulus at the depth z. For friction-type soil the sub-grade reaction modulus is given by:

$$k_k = \frac{n_h \cdot z}{d} \qquad (2)$$

The constant of sub-grade reaction modulus, n_h (MN/m^3), can be found in BRO 2004.[6] According to the geotechnical investigation, the soil surrounding the piles was sand with a very low consistency. Thus n_h was taken as:

	Assumed Values in Design	Measured Values
Low temperature	−40°C	−16°C
High temperature	+40°C	+21°C
Temperature range	80°C	37°C
Stress range	269 MPa	93 MPa (Northern pile)
MPa/°C	3,36	2,58

Table 3: Calculated and measured temperatures and stresses in the concrete slab

Measured strain difference level 1, north side
18 October 2006 – 23 October 2007

Measured strain difference level 1, south side
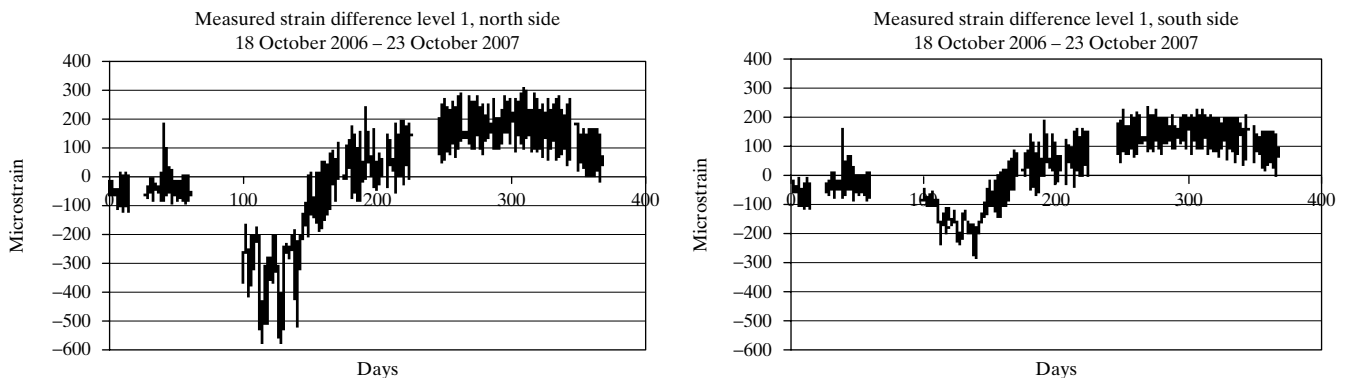18 October 2006 – 23 October 2007

Fig. 9: Measured strain difference between the northern and southern piles, for a period of 12 months
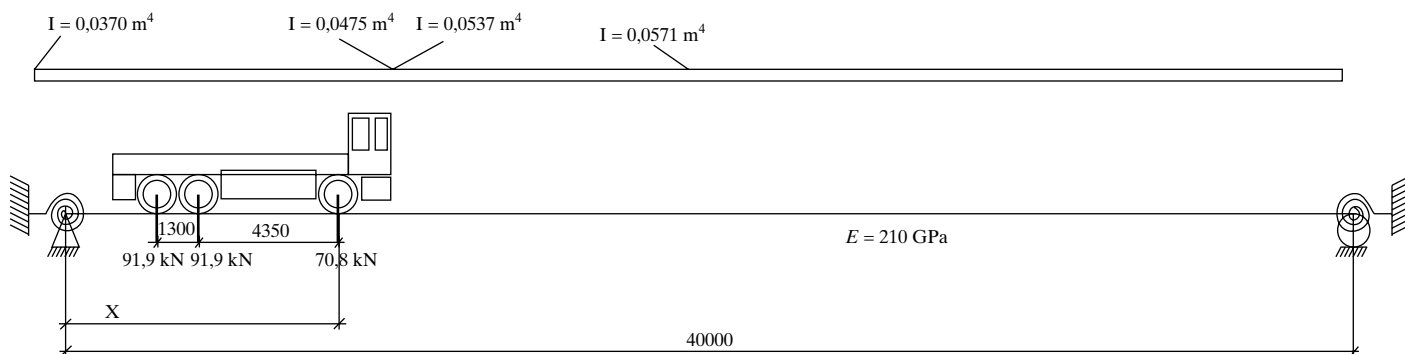
Fig. 10: The truck used for short-term monitoring. The theoretical moment of inertia of one composite bridge girder is also shown above (with the assumption that the ratio between Young's modulus is $E_{steel}/E_{concrete} = 7,2$)

Below ground water level	Linear increase 0,00–1,71 m	$n_{hd} = 1,5$ MN/m^3
	Constant stiffness 1,71–6,00 m	$k_{kd} = 2,57$ MN/m^2

Table 4: Distribution of the soil properties along the pile, according to Bro 2004

- $n_h = 2,5$ MN/m^3 over the ground water level;
- $n_h = 1,5$ MN/m^3 under the ground water level.

In the considered soil model, the soil stiffness increases linearly with the depth until a maximum value of the product $k_k d$ is reached and then remains constant.

For sand with a very low consistency, these limits are:

- $(k_k d)_{max} = 4,28$ MN/m^2 over the ground water level;
- $(k_k d)_{max} = 2,57$ MN/m^2 under the ground water level.

Calibration of the FE model was made by varying the characteristics of soil properties.

The ground water level was assumed to be at the top of the pile.

The depth, z_c, at which the stiffness stops increasing and remains constant can be derived from $k_k d$ and n_h:

$$z_c = \frac{k_k \cdot d}{n_h} = \frac{4,28}{2,5} = 1,71 \text{ m} \qquad (3)$$

and the corresponding stiffness was:

$$k_c = (k_k d)_{max} \cdot s \qquad (4)$$

Results of FE calculations with soil properties used for design according to BRO 2004[6] indicated larger strains than the measured values, which could be explained by the fact that in the real bridge the piles are not rigidly fixed at the pile cap (see Figs. 10 and 11).

The influence of springs supporting the piles is almost negligible for the vertical displacement of the bridge measured while the empty lorry crossed the bridge (see Fig. 12).

Short-term measurements indicate that the deflections are asymmetric, as the deflections are consistently larger on the eastern side of the bridge, but the variations are quite small, 10–15%. The unsymmetrical deflections could probably be explained by some eccentricity in the loading and varying response of the back fill behind the abutment, or a combination of these factors.

The strains in the upper part of the piles (gauge 5 in Fig. 7) were measured in both summer (October) and winter (January) conditions, using the same lorry with a total weight of 24,0 t. In October, the maximum strain amplitudes were 95 μ-strain and 84 μ-strain for the northern pile and southern pile, respectively (Table 5). These strain differences correspond to the stress amplitude on each side of the pile of 20 MPa and 18 MPa, for each pile, respectively.

In January, the maximum strain amplitudes were 47 μ-strain and 45 μ-strain for the northern pile and southern pile, respectively. These strain differences correspond to the stress amplitude on each side of the pile of 10 MPa and 9,5 MPa, for each pile, respectively. The bending stresses in the piles during winter conditions were, according to the monitoring, 46 to 50% lower than for summer conditions (Tables 6 and 7; see also Fig. 9).
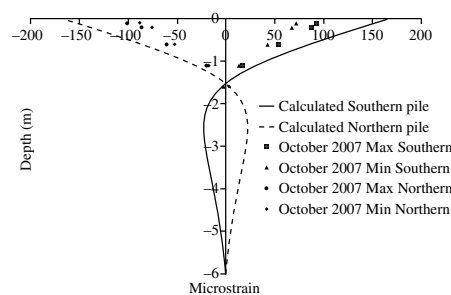

Fig. 11: Comparison between strains calculated by FEA and measured in the pile. The bridge is loaded by a truck according to Fig. 10
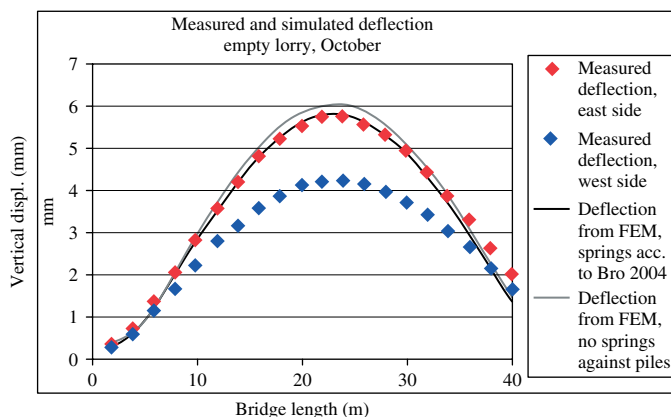

Fig. 12: Measured deflection compared to deflection modelled by FEM. The bridge was loaded by a truck as shown in Fig. 10

	Northern pile (μ-strain)	Northern pile (MPa)	Southern pile (μ-strain)	Southern pile (MPa)	Deflections (mm)
October 2007	94,5	19,9	83,9	17,6	8
January 2008	46,9	9,9	45,3	9,5	5,8
Difference (%)		50,4		45,9	27,5

Load according to *Fig. 10*.

Table 5: Measured strains in the piles, gauges N5 and S5

	Upper flange (μ-strain)	Upper flange (MPa)	Lower flange (μ-strain)	Lower flange (MPa)
October 2007	−3,8	−0,8	74,8	15,7
January 2008	−4,2	−0,9	53,3	11,2

Load according to *Fig. 10*.

Table 6: Measured strains in the flanges at mid-span, gauges 10 and 11, at maximum deflection

	Upper flange (μ-strain)	Upper flange (MPa)	Lower flange (μ-strain)	Lower flange (MPa)
October 2007	−3,8	−0,8	−26,3	−5,5
January 2008	−6,1	−1,3	−31,4	−6,6

Load according to *Fig. 10*.

Table 7: Measured strains in the flanges at support, gauges 1 and 2, at maximum deflection

For the configuration giving maximum deflection in mid-span, the measured bending stresses are 28% larger for summer conditions compared to winter conditions. Also the deflections are larger in October, which could be explained by a larger restraint on the back wall when the soil around the bridge is frozen.

Conclusions

In this paper, results obtained while monitoring the Swedish bridge over Leduån have been presented. For this bridge, the response in the piles and the superstructure was measured for both thermal and traffic loading. Furthermore, a procedure for erecting integral abutment bridges, minimizing the bending stresses in the slender steel piles, has been described.

Part of the problem with integral abutment bridges is that, for all their simplicity of construction, they are complicated structural systems. To thoroughly analyse a given structure, the designer must not only design for primary loads (dead load, live load, wind load, etc.) but must also accurately account for secondary loads (creep, shrinkage, settlements, temperature effects, etc.). To further complicate the analysis, the response of the structure to a given set of forces is much dependent on the geometry, materials, configuration, soil interaction and construction details of the individual system. In order to avoid this complicated analysis, integral abutment bridges should typically be designed by using conservative methods and by building on field experience.

Despite these mentioned uncertainties, the measured values from Leduån with respect to deflection and pile stresses show a fair agreement with the FEA carried out with soil parameters according to BRO 2004.[6] The published research can be used as a knowledge base to determine reasonable guidance for the safe design of integral abutment bridges.

It is believed that integral abutment bridges will continue to gain ground in Europe. Although Eurocodes do not contain specific information on how to construct and design integral abutment bridges, they make it easier for engineers to transfer knowledge on the subject.

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