Monitoring of a bridge with integral abutments

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ABSTRACT: Preliminary results obtained from short term test-loading are used to illustrate possibilities of FEM used to calibrate complex interaction characteristics between a pile and soil in a bridge with integral abutments. The measurements are obtained during the winter season on the bridge over Leduån, in Northern Sweden. The bridge is built in 2006 and used for long term monitoring within the international project INTAB supported by RFCS. The main objective of the on-going research project is to propose recommendations for rational analysis and design of bridges with integral abutments.

1 INTRODUCTION

Integral abutment bridges are bridges without any expansion joints, and their largest benefits are the lower construction and maintenance costs. The abutments are generally supported on a single row of steel piles to provide the required flexibility for accommodating the longitudinal bridge movements due to daily and seasonal temperature variations. Such movements impose cyclic lateral displacements on the abutments, backfill and the steel piles. The magnitude of these cyclic displacements is a function of the level of temperature variation, type of the superstructure material and the length of the bridge.

The first bridges with integral abutments were built in the 1960's in USA and in recent years this type of bridge has gained popularity in Europe, primarily in UK and the Scandinavian countries. The maximum span width as well as the maximum bridge length has been increasing through the years. The longest concrete bridge is 358 m, located on Tennessee State Route 50 over Happy Hollow Creek. The longest steel girder integral bridge has a span of 318 m in the Colorado State (Hassiotis & Roman, 2005). It is obvious that countries such as USA, with design requirements more open towards new technologies, are encouraging implementation of innovative solutions even if they are not thoroughly examined. Although nationally accepted design specifications for integral bridges do not exist, each highway department is allowed to make decisions depending on their own expertise.

In Sweden, the maximum and minimum characteristic temperatures, which statistically occur once in 50 years, are varying between +30ºC and +38ºC respectively -20ºC and -48ºC. This rather large temperature differences decreases the upper length limit compared to USA. Some states in USA allow plastic strains to be developed in the steel piles below the abutment back walls.

Just a few bridges with integral abutments have been built in Sweden since the 1980's. The main obstacle for wider acceptance of this type of bridges is a lack of recommendations for rational analysis and design. According to the Swedish code for bridges, BRO 2004 (Vägverket, 2004), no plastic strains are allowed at serviceability limit state even if the critical section is at the top of a pile. Such high strain may occur due to seasonal temperature variation, a couple of time a year, and are limited at the very narrow region. These effects may cause low-cyclic fatigue which will be thoroughly examined in an on-going RFCS (Research Fund for Coal and

Steel) project, INTAB- Economic and Durable Design of Composite Bridges with Integral Abutments, running in period 2005-2008.

The first investigation focusing on development of rational design for steel piles was initiated at LTU in the late 1990´s where results are published in (Pétursson, 2000) and recently in (Tlustochowicz, 2005), (Hällmark, 2006) and (Nilsson, 2008). The major complexity of the integral bridges is actually in the soil-structure interaction that requires the coordinated attention of both structural/bridge engineers and geotechnical engineers. A need for a multidisciplinary team in design practice is clearly identified and recommended for finding a most economical solution. The results presented in (Kerokoski, 2006) are used in the on-going project to achieve a rational methodology which is required to determine the maximum length limit for integral bridges.

1.1 *Bridge over the Leduån River*

In the end of 2005, the Swedish National Road Administration ordered an integral bridge as a part of a research and development program. The bridge would be continuously monitored by Luleå University of Technology during a period of 18 months. The researchers were involved in an early stage in order to make sure that the design of the bridge and the construction enabled continuously field measurements.

The bridge was designed as a single span composite bridge with a span length of 40 m and a width of 5 m. Six vertical end bearing steel pipe piles, with the nominal dimensions Ø170x10, are supporting each abutment. At each support, one pile were prepared and equipped by LTU with strain gauges before installation. The soil surrounding is mainly silty fine sand to fine sandy silt. The abutments are designed with a shelf at the bottom, see Figure 1. This makes it possible for the piles to pass rather sheltered through the crushed stone used as erosion protection. The free distance between the piles and the concrete back wall is nominally 80 mm. The two meters of the piles below the bottom of the abutment were protected by a sheltering steel pipe, Ø600, filled with loose sand. The piles were also wrapped with Styrofoam sheets where they passed through the sheltering pipes.

Figure 1. Piles are wrapped with Styrofoam.

A conventional concrete bridge had also been designed as an alternative to the integral abutment bridge. That design demanded an internal support in the river, including a more complicated foundation work. The cost estimation for the integral abutment bridge showed that it in this case was economically superior compared to the conventional design. This cost estimation was made without taking the maintenance costs into account, which would have made the integral abutment bridge even more competitive.

The construction procedure from the foundation work to the finishing of the superstructure, can be described as followed:

- The area where the piles shall be driven is excavated down to about 2 m below the abutment.
- The piles are driven in accordance with the construction drawings. The tolerances in the longitudinal direction are set as ±100 mm. The piles have 12 m sections and are butt-welded in end-to-end connections.
- The pile inclination and straightness are measured, and the results are forwarded to the bridge designer.
- Sheltering piles, Ø600, are placed around the piles with the piles in the centre of the pipe. Piles are wrapped with Styrofoam sheets and loose sand is filled in the pipe.
- The foundation work is then made up to the bottom level of the abutment.
- The lower part of the abutment back walls is cast at the same time as the wing walls.
- The supports below the wing walls are removed, resulting in a pile moment in the opposite direction of the moments acting on the finished structure. The moment from the wing walls can bee seen as a prestressing of the pile.
- Steel girders are mounted on temporary steel bearings.
- Bridge deck and the upper part of the abutment back walls are cast in an ordinary way. The concrete in the upper part of the back wall shall be cast last.
- Pavement and rails are placed on the bridge.

Figure 3. The bridge over the Leduån River

2 THE ON-GOING RESEARCH AT LTU

The main research activity is carried out within a European R&D project, INTAB-Economic and Durable Design of Composite Bridges with Integral Abutments, financially supported by RFCS and SBUF (Development Fund of the Swedish Construction Industry). There are two major project objectives, a concept saving investment and maintenance costs. The project includes international comparisons, theoretical studies, in-situ and laboratory testing of bridges and the preparing for guidelines for design of such bridges. The major objectives of the project are the development of cost effective, environmental friendly and sustainable bridge structures. The project is focused on composite bridges and the improvement of their durability in order to make them even more competitive.

The partners involved in the project are:

- RWTH Aachen, Germany
- University of Liège, Belgium
- ProfilARBED, Luxembourg
- Ramböll Sverige AB, Sweden
- Luleå University of Technology, Sweden
- Schmitt Stumpf Fruehauf und Partner, Germany as an associated partner

2.1 *Monitoring program*

To gain knowledge about the overall behavior of the bridge a total of 34 measurements will be constantly recorded and stored in period of at least 15 months, starting from autumn of 2006. Strain-gauges were welded to the bridge girder and to the pile, as shown in Figure 4, Figure 5 and Table 1.

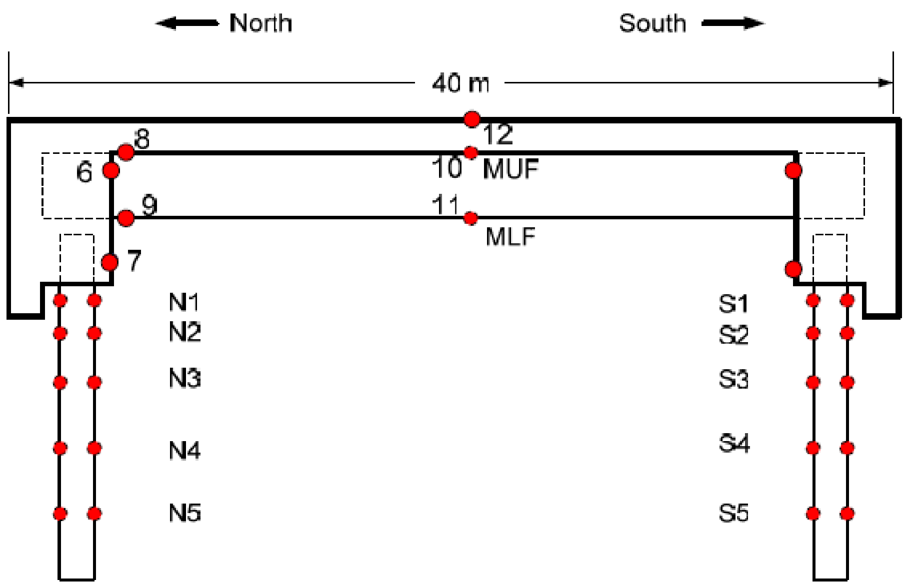

Figure 4. Position of long term monitoring gauges at the Leduån bridge.

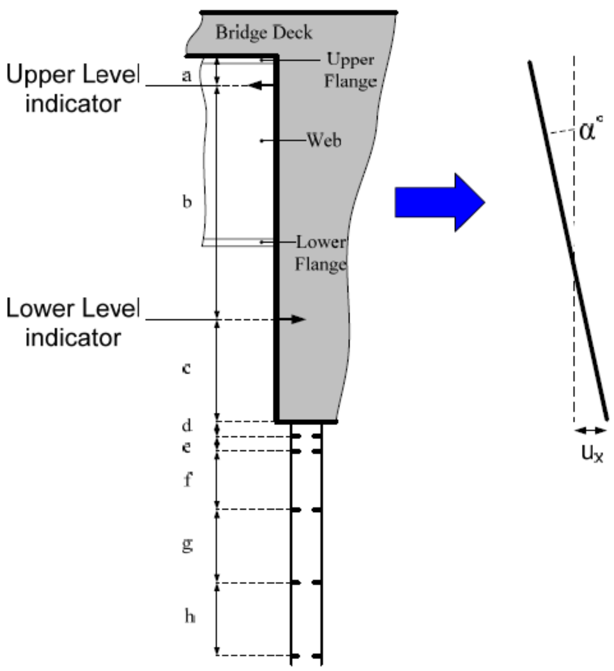

Figure 5. Measurements of the abutment rotation and its effect on the pile.

Table 1. Position of gauges at the end of the bridge.

Notation	Distance [mm]
a	200
b	1500
c	356
d	100
e	100
f	400
g	500
h	500

Strains in the piles were measured at five different levels with the two strain-gauges at each level. The strain-gauges position was such that the maximum strains were measured. At the upper four levels, called N1-N4 and S1-S4, for the north and south side of the bridge respectively, the stored signal consists of the difference between two strain-gauges. Signals obtained from both pairs of strain-gauges; at the fifth level, N5 and S5, were stored separately. An estimation of the axial force in the pile was possible from the measurements at the fifth level.

The movement of the abutment walls is measuring with level indicators, two at each side of the bridge. Their position is indicated by numbers 6 and 7 in Figure 4. The level indicators were placed in a vertical plane along the centre line of the bridge at a distance of 1.5 m. With known geometry of the abutment, rotation and displacement of the pile top is estimated from these measurements.

Two strain-gages were welded at the steel girder, close to the south abutment, to get an indication of the moment constrained obtained at the bridge end. Strain-gages were placed at the upper and lower flange, pos. 8 and 9 respectively. Strain-gages were also welded to the upper and lower flanges at the mid-span of the bridge, pos. 10 and 11 for the estimation of the overall bridge behavior.

Effects of temperature seasonal variation is very important effect that influence strains at the top of the pile. Therefore, the air temperature as well as temperatures at three positions of the bridge; on the upper flange, on the lower flange, in the concrete, position 10, 11 and 12 respectively; were stored.

2.2 *Evaluation of the short term test-loading*

A lorry which has a mass of 24,4t has passed over the bridge at an average speed of 1m/s, which was used to obtain effects referred in the text as the short term test-loading. Similar loading has been done on four other occasions. Possible seasonal differences of the soil conditions and its material properties will be estimated from these measurements. The first short term test-loading was performed in January 2007. The air temperature at the bridge location was as low as -20^{o}C in a period of a couple of days prior to the measurements.

The pile top is embedded in the abutment wall. Therefore, the lateral displacement and the rotation at the pile top are obtained from the measured rigid body movement of the abutment; pos. 6 and 7.

2.2.1 *Measurements at the piles*

Results from the measurements obtained from one pile at the north and one pile from the south abutments shows similar pattern, after the lorry has passed over the bridge small strains, at the level of 10 micro strains, remains in the pile indicating an axial compressive force and a small curvature. These preliminary results indicate a good quality of measurements and expected behavior of the pile. Interpretation of the measurements is on-going work and a preliminary analysis of the soil characteristic is given below.

2.2.2 *FE Model of the pile*

FEA is used to interpret the behaviour of the measurements. One of the parameters that are rather difficult to measure is the properties of the soil. This issue is indirectly estimated calibrating results of a rather simple 2D FE model with the measured data. The pile is modelled by beam elements, using elasto-plastic hardening material and von Mises yield criterion. The soil influence is modelled by elastic two node spring elements. The spring elements connect the pile nodes to fixed reference points and provide only horizontal support. Totally 69 spring-elements were used, more tightly distributed at the top of the pile, where changes of the slopes occur. The distances between the springs are in a range between 0.05 m to 0.15 m. Every spring has a particular property allocated to produce a reaction force equivalent to a soil layer centred at the point of connection.

Recommendations from the Swedish bridge code (Vägverket, 2004) are used in the design for the definition of the spring stiffness. The spring stiffness, k, is then:

$$k = k_k \cdot A_{spring} = k_k \cdot d \cdot s \tag{1}$$

Where A_{Spring} is the projected pile-soil contact area related to one spring, d is the pile outer diameter, and s the distance between two springs, k_k [MN/m^3] is the sub grade reaction modulus at the depth z. For friction type soil, no difference is made between short term and long term stiffness and the subgrade reaction modulus is given by:

$$k_k = \frac{n_h \cdot z}{d} \tag{2}$$

The constant of sub grade reaction modulus, n_h [MN/m^3], can be found in a table. According to geotechnical analysis, the soil surrounding the piles is sand with a very low consistency. Thus n_h is taken as:

- 2,5 MN/m^3 above the ground water level
- 1,5 MN/m^3 below 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 it then remains constant. For sand with a very low consistency, these limits are, $(k_k.d)_{max}$:

- 4,28 MN/m^2 above the ground water level
- 2,57 MN/m^2 below the ground water level

In a process of calibration the FE model, the characteristic soil properties were considered. The ground water level is supposed to be at the top of the pile. The depth at which the stiffness stops increasing and remains constant can be derived from k_k.d and n_h:

$$z_c = \frac{k_k \cdot d}{n_h} = \frac{4.28}{2.5} = 1.71m \tag{3}$$

and the corresponding stiffness is:

$$k_c = (k_x \cdot d)_{\max} \cdot s \tag{4}$$

Results of FE calculations with soil properties used for design according to (Vägverket, 2004) gives lower strains than those measured on site. FEA indicates a higher curvature that must be due to a stronger lateral support of the pile from the surrounding soil. It is important to notice that the short term test-loading was performed in winter as temperatures were very close to the minimal and the soil was probably frozen. It is know from (Kerokoski, 2006) that the soil stiffness may be increased approximately by a factor 30 compared to unfrozen soil below ground water level.

For the northern pile the results of FEA fit the measurements best for a soil with a constant stiffness 20 to 30 times higher than that of the unfrozen soil below the ground water down to a depth of 2m. Below that level the characteristic soil of the norm is used. The behavior of the southern pile is different. The soil parameters appear to be higher only by factor 5 to 10.

The results of the monitoring collected from measuring places at the bridge girders will be compared to the complete bridge model results. A 3D FE model is currently under construction, which will be presented in another paper.

3 CONCLUSIONS

The preliminary results obtained from the monitoring of the Leduån Bridge indicate that a rather simple FE model, suitable for design practice, may be used to calibrate unknown parameters of the soil under traffic load. Results from the measurements, obtained in January 2007, indicates stronger stiffness of the soil in a range between 10 to 20 times more compare to stiffness of an unfrozen soil. The strains from the bending moment in the piles caused by the traffic load of the 24t lorry are very low, where the maximum strain measured was about 60 micro strains.

3.1 *Acknowledgements*

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