EXPERIENCES WITH SHM FOR LARGE CIVIL STRUCTURES

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When studying the long-term behavior of structures, it often becomes necessary to resort to monitoring of certain structural parameters. This research paper discusses two such monitoring projects which are used to verify design assumptions. A first project deals with an integral bridge that is being constructed in several separate building stages, continuously allowing traffic on parts of the bridge. In order to verify the actual behavior of the bridge, strain gauges have been applied on the beams of the superstructure, while ground pressure sensors have been attached to the back of the abutment walls. Strain and ground pressure are being monitored continuously during a one year period wherein the bridge is constructed, post-tensioned and fully opened for traffic. This project necessitated the use of a fully autonomous acquisition and data storage system since no reliable power supply was available. As a second example, this paper is concerned with the use of strain gauges as a control mechanism on the effectiveness of the post-tensioning of large concrete beams, used as part of the roof of a cut-and-cover tunnel for the high speed train link passing under the Antwerp Central Station. The emphasis is placed on the specific problems encountered when working on site. A comparison is made between calculated and measured values of concrete stresses caused by the tensioning operations, proving that the measuring method is adequate. This article gives an overview of these experiences and on the lessons learned concerning power supply and possible electromagnetic interference in harsh construction site condition. Furthermore, the most important results of both test cases are discussed in short, including the effectiveness off the integral bridge and the effectiveness of sequential post-tensioning.

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ABSTRACT: When studying the long-term behavior of structures, it often becomes necessary to resort to monitoring of certain structural parameters. This research paper discusses two such monitoring projects which are used to verify design assumptions.

A first project deals with an integral bridge that is being constructed in several separate building stages, continuously allowing traffic on parts of the bridge. In order to verify the actual behavior of the bridge, strain gauges have been applied on the beams of the superstructure, while ground pressure sensors have been attached to the back of the abutment walls. Strain and ground pressure are being monitored continuously during a one year period wherein the bridge is constructed, post-tensioned and fully opened for traffic. This project necessitated the use of a fully autonomous acquisition and data storage system since no reliable power supply was available.

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1 INTRODUCTION

In Belgium, a large number of bridges and fly-overs were built in recent years, all part of the European High Speed Train network. A considerable percentage of the large and medium span bridges amongst them were subjected to test loading [1, 2] using heavy construction lorries or test trains before completion, during which strains and or accelerations were measured for comparison with the basic design premises.

For the study of the long-term behavior of civil structures, it often becomes necessary to resort to long-term monitoring of certain structural parameters, such as the strains, temperatures, accelerations, frequencies, etc. Especially when the nature of the structures necessitates the use of an independent power source for the measurement system and prohibits the use of wired connections with the environment, a fully autonomous measurement system becomes necessary. This research paper discusses two such applications and the possible pitfalls in developing such a monitoring system.

2 MONITORING OF AN INTEGRAL BRIDGE

2.1 Introduction

Bridges which are constructed without any expansion joint and without any bearing are called integral bridges. In its simplest form, an integral bridge may be viewed as a box culvert or a portal frame structure where the road surfaces are continuous from one approach embankment to the other. The Dilbeek Bridge, which is discussed here, is actually a three span integral bridge, since no joints exist between spans or at the abutments.

Figure 1. Construction of the connection between longitudinal girders and intermediate pier of the Dilbeek Bridge.

The integral bridge concept has several advantages. The design and detail of the bridge is more straightforward since no joints, with the requirement for close tolerances are necessary. Minor misallocation of piers or abutments will not create any fit-up problems. Smooth jointless construction improves vehicular riding quality and diminishes vehicular impact stresses. Expansion joints and bearings have traditionally been used to accommodate the seasonal thermal expansion and contraction of bridge decks, typically of the order of tens of millimeters. Furthermore, expansion joints are a serious source of costly and disruptive maintenance work during the bridges' lifespan.

Although the integral bridge concept has proven to be economical in initial construction for a wide range of span lengths, as well as technically successful in eliminating

expansion joint/bearing problems, it is susceptible to different problems that turn out to be geotechnical in nature. These are potentially due to a complex soil-structure interaction mechanism involving relative movement between the bridge abutments and adjacent retained soil. Because this movement is the result of natural, seasonal thermal variations, it is inherent in all integral bridges. There are two important consequences:

1. Seasonal and daily cycles of expansion and contraction of the bridge deck can lead to an increase in earth pressure behind the abutment. This build-up of lateral earth pressures is referred to as 'soil ratcheting'. This can result in the horizontal resultant earth force on each abutment being significantly greater than that for which an abutment would typically be designed and represents a potentially serious long-term source of integral bridge problems.

2. The second consequence is the soil deformation adjacent to each abutment. It has been postulated that settlement troughs occur as a result of the soil slumping downward and toward the back of each abutment.

The three spans of the Dilbeek Bridge, as well as both sides (driving directions) of the bridge are constructed separately in order to allow traffic crossing the construction zone at all times. This complicates the integral construction, as shown in Fig. 1. In order to verify that integral bridge behavior occurs and that ground pressures do not rise to problematic values, a monitoring setup was installed during construction.

Figure 2. Installation of the ground pressure sensors behind the front wall of the Dilbeek Bridge.

2.2 Measurement set-up

At two locations behind one of the abutments, two sets of five special ground pressure sensors have been installed. Total earth pressure cells are designed to measure stress, acting on plane surfaces and are constructed from two circular stainless steel plates, welded together around their periphery. The annular space between these plates is filled with de-aired glycol and connected to a transducer. Five of these cells are installed at a

with a vertical distance between sensors of 1m as is shown in Fig. 2, allowing for a clear image of the build-up of ground pressures as a function of the depth.

Since no reliable electrical power was available during the monitor period, the entire system was designed for as low as possible power consumption. Everything can operate for about a month on 6 lithium-ion D-cells. In addition, a half bridge setup was for the Wheatstone bridges of the strain gauges in order to reduce the influence of heating of the strain gauge and zero drift.

2.3 Results

Initial results of the ground pressure cells are shown in Fig. 3. The filling of the installation pit is clearly visible. The pressure cells are numbered starting from the surface. After an initial settlement period of about 3 weeks, ground pressure remained more or less constant, daily variation due to temperature notwithstanding. In order to filter out this effect, all pressure cells are equipped with temperature registration sensors.

Figure 3. *Evolution of the ground pressure behind the front wall of the Dilbeek Bridge during the first few weeks after installation.*

3 POST-TENSIONED CONCRETE BEAMS ANTWERP

3.1 Introduction and setup

Part of the Antwerp North-South railway link constitutes of a carrying beam structure where post-tensioning is applied. During the construction, the eastern part of the beams is built first and temporarily supported on bored piles. The second part is constructed afterwards, thus allowing two tracks to be in service during the execution of the works. After the tensioning operations, the beams lift from the temporary supports and become simply supported at both ends on slurry walls. The latter become the final walls of the underground structure, extending several levels deeper. These reinforced-concrete walls are made by the slurry wall method and go deep into the Boom Clay. Some deeper parts of the walls are built later on during further construction phases. The temporary and

permanent supports of the crossbeams are shown in Figure 4, as well as the construction joint between the two construction phases. Due to the many track switches in the tunnel before entering the station and to obtain sufficient room for this, the intermediate supports must be eliminated once their function has become obsolete.

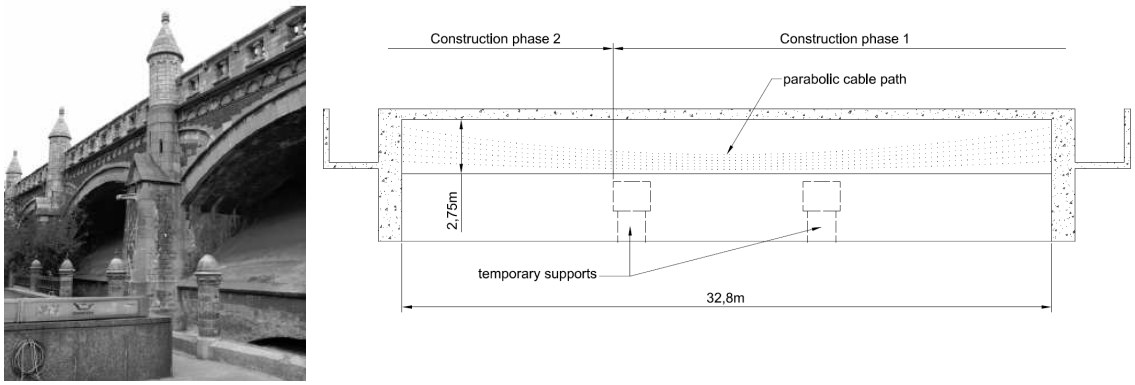

Figure 4. *Protected masonry retaining walls flanking the existing tracks(left) and a drawing of the largest crossbeam, showing the permanent and temporary supports, as well as the 2 construction phases (right).*

During the design stage it was thought that the many successive building phases would influence the efficiency of the post tensioning. The beams are furthermore rigidly connected to the walls, thus causing a deformation of the walls when the cables are tensioned. This can easily be accounted for in the design. There exists however a substantial ground mass behind the slurry walls. This, partly stirred, ground mass will be excavated later on, which can reduce part of the applied post-tensioning. The uniform compression by the post-tensioning was not taken into account in the calculation model due to the uncertainties described above. Only the lifting forces, caused by the curvature of the cables were considered. It was believed that the combination of all precautions taken formed an easier and at least as reliable alternative for the use of replaceable post-tensioning. However, to allow for quick decision-making about the necessity of the extra post-tensioning, there was a need for quickly feasible and reliable measurement data.

Working with measurement equipment built into the beams did not seem very realistic, when taking into consideration the difficult working conditions on the building site. All beams were constructed with an extremely dense placement of the normal reinforcement bars, thus further complicating this option. The high working speed of the construction site necessitated a swift measurement method, leaving virtually no time for interpretation of the results, notwithstanding the need of a quick evaluation of the results. Because of the impossibility to work with built-in systems, a permanent monitoring system was also excluded.

Measuring the strain directly on the concrete surface may seem contradictory at first, because of the non-homogenous composition of the base material, however it has certain advantages. There is no need to convert the measured strain values acquired from gauges glued directly on the passive reinforcement bars to the strain of the surrounding concrete or of the concrete surface. This conversion is in most cases quite dubious, since the exact location of the strain gauge after the pouring of the concrete and the thickness of the concrete cover is guess work at best. When applied directly on

the reinforcement bars, the strain gauges are also highly sensitive to damage, as is the experience of this research group on previous occasions when strain gauges were used in a much more controllable environment such as a pre-tensioning factory.

3.2 Results

The performed measurements are: Zero Measurement, after applying 75% of the total tensioning force, after applying 100% of the total tensioning force, 14 days after the previous measurement, 2 months after the previous measurement, at the completion of the whole project

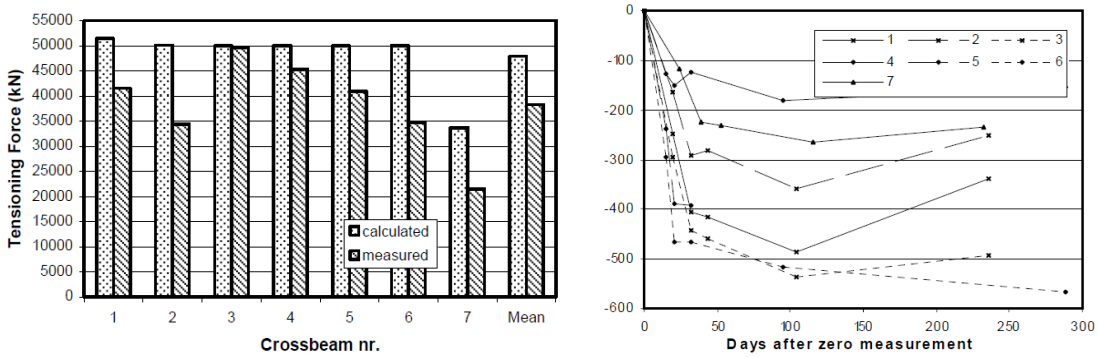

Figure 5. *Comparison of calculated and measured tensioning forces after the first tensioning operation, in the middle of the beams (left) and strain evolution at the middle of the undersides of the crossbeams (right).*

After the second measurement operation it is possible to calculate the existing stresses in the beams, using the occurring strains, a Young's modulus of 34000MPa and a Poisson's ratio of 0,2. This modulus was obtained by testing a cube of the concrete mixture.

The separation of the bending stresses from the normal stresses allows for the calculation of the tensioning forces and bending moments to which the beam is submitted. This separation can be done after the second, as well as after the third measurement operation, allowing for the calculation of the bending moment and normal force, caused by the first tensioning operation and by the tensioning of the last 4 cables, derived from the difference between the second and third measurement.

The comparison between calculated and measured values is made difficult because of the following independently occurring effects:

- The influence of the remaining soil between the slurry walls. The calculation model assumes that the sidewalls are elastically supported from approximately 2 meters beneath the beams;

- The noticeable influence of the massive quantities of passive reinforcement bars, thus reducing the influence on the concrete of the post-tensioning;

- Before the post-tensioning operation the beams aren't solely supported by the sidewalls, but also by 2 additional intermediate supports. The weight of the roofs is then carried by all four supports, while the ballast is placed only partially at this early stage. Because of the tensioning operation, the beams will lift up from

the intermediate supports. The total amount of ballast used has a direct relation to the size of this lift effect and was determined at 0,7 to 0,8m.

Figure 5 (left) shows the calculated and measured tensioning forces for the last seven crossbeams: the measured tensioning force in all beams is lower than the calculated one. The discrepancies between measured and calculated values for certain beams can only be explained by the above mentioned reasons and specifically the unknown influence of the soil mass behind the slurry walls. While this soil can be densely compacted in some areas, because of the site traffic, this isn't necessarily true for the entire length of the slurry walls.

However, bearing in mind that the theoretical model cannot fully take into account all occurring effects and will in most cases overrate the actual values and since this is in fact the case for all considered beams, one can conclude that the measurements constitute a safe and adequate control method of the tensioning forces.

All measured strain values on the lower side of the mid-span sections of all measured crossbeams are shown in figure 5 (right). Additional measurements on one of the beams show that one should be weary of the linear connection between the first two measurements. Notwithstanding, a slight shrinkage effect, all strains remain close to zero before the first tensioning operation. The different tensioning operations are clearly visible. Another trend on the charts, between measurements 2, 3 and 4, can be explained by a different temperature and shrinkage as well as creep effects. In view of these effects, it may seem illogical that all strains show an upwards tendency, a stress reduction in all beams, at the end of the measurement period. This, however, can be explained by the excavations of the soil between the slurry walls in the intervening months. These ground works reduce the outward pushing ground forces, resulting in a slight reduction of the tensioning forces. In some beams this effect is intensified by a change of the ambient temperature, thus in a small positive strain.

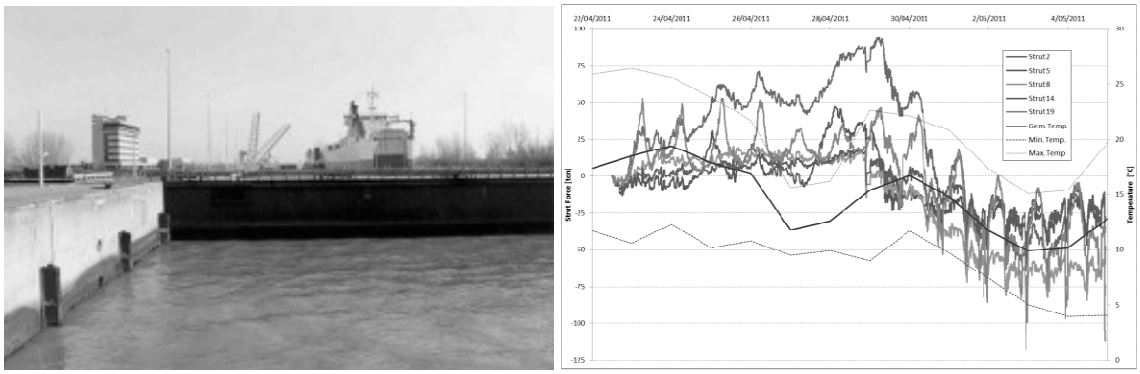

Figure 6. *Lock doors of the Van Damme Lock in Zeebrugge, Belgium (left) and initial results (right)*

4 FUTURE MONITORING PROJECTS

4.1 Van Damme lock doors

During the renovation of the lock doors of the inner harbor of the Zeebrugge Port, Belgium, shown in figure 6 (left), a measurement system has been installed to measure the normal force in the struts between the walls of the lock room. Initial results, shown

in figure 6 (right), indicate that the influence of temperature variations on the normal force is much larger than anticipated in the design.

4.2 Haeren viaduct

Another project that is installed during summer 2011 deals with the Haeren viaduct where the special focus will be the distribution of breaking forces over the piers of a viaduct consisting of simply supported decks and the follow-up of creep in the concrete decks. Some photographs of the steel and concrete parts, with 'Iris'-shaped piers are shown in figure 7.

Figure 7. *Steel bridge decks (left) and precast concrete piers of the concrete viaduct in Haeren, Belgium*

5 CONCLUSIONS

This article gives a quick overview of experiences with monitoring projects during the construction of large civil infrastructure. It lists some of the problems and the lessons learned concerning harsh construction site condition. While the use of strain gauges, applied directly to the concrete surface, as a controlling mechanism on the effectiveness of the post-tensioning of large concrete beams, on site may not seem the most obvious solution, the paper delivers a case study stating the validity of this method. Indeed, when the practical execution of the measurements is carried out with the utmost precision, this method delivers reliable and precise results, taking into account the specific working conditions. . Furthermore, the most important results of both test cases are discussed in short, including the effectiveness off the integral bridge and the actual post-tensioning efficiency on-site of the tunnel.

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