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Monitoring of Displacements on Suspension Bridges using GPS

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Summary

It is proposed in this paper a new measurement method using GPS to measure semi-static and long period movements of suspension bridge girders. The field measurements on dynamic movements were carried out using this system on the Hakucho suspension bridge during the strong wind season, and semi-static displacements of girders were successfully obtained and agreed well with numerically predicted values. Spectral analyses were conducted for the displacement data collected both by GPS and by accelerometers as well, which show good agreement in the frequencies between 0.1 to 0.4Hz. This is also confirmed by the numerical results calculated by FEM bridge model and forced vibration tests. This method is therefore proved to be useful for dynamic measurements of semi-static and ultra-long period movements of long-span bridge girders.

1. Introduction to GPS Monitoring System

The monitoring system on dynamic behaviors has been installed on major suspension bridges in Japan to assure the safety of vehicles and structures. However, vibration periods are very long on long-span suspension bridges and there exist both semi-static and periodic movements in lateral directions due to wind forces, therefore, the conventional measurement system with accelerometers are unable to catch these movements. It is proposed in this paper a new monitoring system using GPS (Global Positioning System) which utilizes electric waves from satellites to measure semi-static and long-period movements of suspension bridge girders.

The Hakucho Bridge is a suspension road bridge with main-span of 720m and two side-spans of 330m each. The girder is a streamlined steel box girder with width of 23.0m and maximum web height of 2.5m. The bridge crosses the Muroran Bay and suffers strong winds during typhoon and winter seasons. As a part of extensive dynamic tests conducted on the Hakucyo Bridge, field measurements using GPS were carried out for three weeks in March 1998 just before the bridge opened for vehicles in June.

The real-time monitoring of 'kinematic GPS' was used in this measurement. Two sets of GPS antennas and receivers were placed at mid-span of the girder and at the office on land with distance in between about 1 km. Both antennas simultaneously received electric waves from several satellites, and the displacements of three directions, vertical, lateral and longitudinal, were obtained by the phase difference of both waves. A pair of telemetry links between the main and reference receivers were installed, and the measured data were analyzed by a processing software run on a laptop PC. Sampling time was 1.0sec.

Before the field measurement, a test was carried out to evaluate errors caused by this GPS system. The main antenna was temporarily fixed on the tower base, and forced to move 10cm in three directions and measured. It showed that the error was within 1.6 cm in the lateral and longitudinal directions and within 0.5 cm in the vertical direction.

2. Measurement Results and Spectral Analyses

A typical record measured from 1:00 a.m. to 7:00 a.m. on March 1998 showed longitudinal displacements Dx, lateral displacements Dy, vertical displacements Dz, wind speed U and wind angle Ur. The wind data was obtained by a ultrasonic anemometer on the tower top. Dominant wind direction Ur was WNW-NW, which was slightly diverted from the girder transverse angle W. The wind speed U varied between 5m/s to 20m/s in this data.

Lateral displacements Dy consisted of both semi-static components and fluctuating components. The semi-static displacements varied with wind speed U and there is strong correlation between Dy and U. The maximum Dy reached about 20cm when wind speed is about 20m/s, which agreed with the wind tunnel test results. Vertical displacements Dz and longitudinal displacements Dx consisted of only fluctuating components, and there seemed no correlation on Dx and Dy against U.

The data were divided into two minute data, and the relation of averaged Dy versus lateral components of wind speed Uy were compared with the theoretical values. These values were predicted by three dimensional FEM using drag coefficients C_D of 0.75 for girders and 0.70 for cables, which were obtained by the wind tunnel tests. The measured data Dy agreed very well with the numerically predicted values.

The girder displaces up and down because of the temperature change of the main cables, and the vertical displacements Dz measured for 24 hours clearly showed this periodic movements. The measured displacement range in the day was nearly equal to the calculation based on the maximum temperature change measured by the thermometer installed inside of the cable.

The data were analyzed by Fourier transform. Fourier spectrum of Dx, Dy and Dz showed clear frequency peaks. Accelerometers were also placed at the same position of GPS antenna and the acceleration data was collected at the same time. Peak frequencies obtained by GPS data and those obtained by accelerometers agreed very well. The bridge was later forced to vibrate by exciters and natural frequencies were obtained. The peak frequencies obtained by GPS agreed well with these natural frequencies and also with those calculated by three dimensional Finite Element bridge model. This proves that GPS method is reliable for dynamic measurements of ultra-long period movements in lower frequency range.

3. Conclusions

A new measurement method using GPS was applied to measure displacements of suspension bridge girders. Field measurements were carried out using this system on the Hakucho Bridge during the strong wind season. This study proves that GPS method is efficient for measurements of semi-static and long period movements of long-span suspension bridge girders. During stormy weather, it is difficult to judge only by wind data when a bridge should be closed for vehicles. However, monitoring girder displacements by GPS can give useful and appropriate information for the judgement. This monitoring system also helps us to find structural abnormal phenomena in early stages, and consequently contributes to long-term durability of structures.

Traffic Load Models and Weigh-In-Motion Data

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Summary

During the evaluation of an existing bridge, measurements at the site can be used to update the traffic load model. Measurements can be made with either temporary Weigh-In-Motion (WIM) sensors installed on the road surface or with strain gauges installed on the structure. The aim of updating a traffic load model is to avoid costly interventions on an existing bridge that are often proposed as a result of an assessment using a design traffic load model. This paper presents the measurement of traffic loads and effects at a bridge site in Switzerland. Simultaneous measurements with temporary WIM sensors placed adjacent to a bridge and strain gauges on bridge elements were used to study vehicle characteristics and traffic load effects. The results of this study provide valuable information about the effects of actual traffic on bridges and the use of WIM data for bridge evaluation.

Keywords: Road bridges, traffic loads, weigh-in-motion, strain measurement.

Strain measurements

Static and dynamic load tests were carried out during a bridge closure using two 3-axle test vehicles with a total weight of approximately 25 tonnes each. Continuous dynamic measurements under normal traffic conditions were subsequently made over two periods totalling 18 days. Strain measurements during the static load test enabled the behaviour of the bridge to be assessed. These measurements showed that traffic load effects were lower than assumed during design, thereby illustrating the advantage of carrying out measurements on a structure as opposed to using a default structural model.

By processing the strain data from continuous measurement under normal traffic, it was possible to filter out noise, dynamic effects and temperature effects in order to identify vehicle loading events and to determine the peak static effect of traffic actions during each event. Figure 1 shows a histogram of peak stresses obtained by processing data recorded over a period of three days.

WIM Traffic survey

The vehicle survey was carried out using temporary Weigh-In-Motion sensors installed on the road surface at a distance of approximately 200 metres from the bridge. The portable WIM system consists of inductive loops to detect the presence of a vehicle and to measure its speed and length combined with WIMstrip capacitative sensors that weigh half of each axle. More than 30 000 heavy vehicles were measured using the WIM system over a period of 18 days. However, the system is sensitive to vehicle speed and experiences problems measuring axle groups. For this reason, erroneous measurements were later filtered out and approximately 10,600 heavy vehicles were retained for further analysis. A histogram of heavy vehicle total weight is shown in Figure 2.

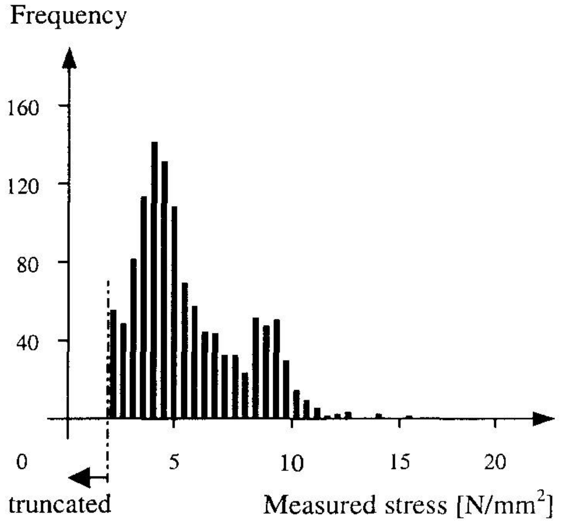

Fig. 1. Histogram of peak stresses measured under normal traffic conditions

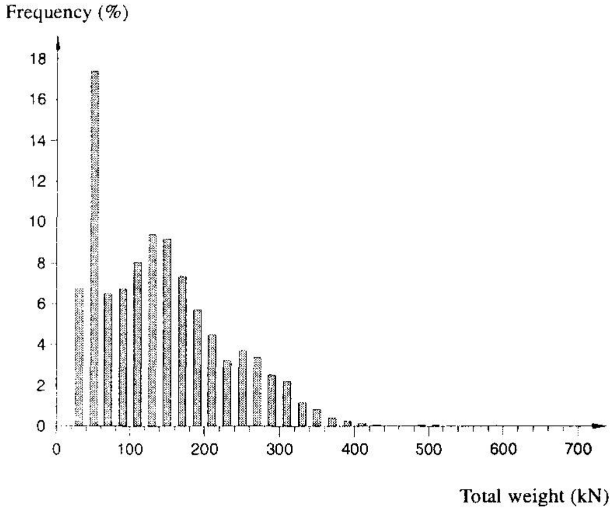

Fig. 2. Histogram of heavy-vehicle total weight

Discussion

The two approaches to updating traffic load models involve different amounts of effort and yield results of different quality. The continuous measurement of strains in a bridge requires expensive equipment and the use of expert technicians as well as complex data post-processing. On the other hand, the use of temporary WIM sensors is relatively straightforward and real-time processing produces vehicle data that is easy to use. In terms of financial investment in equipment and man-time, strain measurement is approximately twice as expensive as the use of temporary WIM sensors. This comparison of cost should in turn be balanced against the value of the data collected. In this respect, this study has shown that strain measurement yields precise data about traffic action effects and bridge behaviour, whereas vehicle data collected using temporary WIM sensors is subject to significant uncertainty. A decision about which approach to adopt must therefore be made by considering the purpose of measurement. If vehicle counting and a general idea of average vehicle loads is required then temporary WIM sensors suffice, otherwise a more costly permanent installation of WIM sensors is necessary for weight data of greater accuracy. If dynamic traffic action effects and bridge behaviour are of interest then strain measurement is preferable.

Conclusions

The measurements carried out as part of the study presented in this paper clearly demonstrate the relative advantages and disadvantages of two approaches to updating design traffic load models for bridge evaluation.

A traffic survey with a temporary WIM system has shown that measurements are subject to significant dispersion. The prediction of extreme traffic loads, in particular, is therefore subject to high uncertainty. Furthermore, traffic action effects calculated using traffic data are dependent on an assumed bridge model, thus introducing further uncertainty.

Strain measurement provides accurate information about the actual traffic action effects on a structure, but is limited to instrumented elements. Direct measurement provides information about load distribution within a structure, the combined effect of vehicles and dynamic effects. However, the volume of strain data recorded should be reduced by processing in real time.

Bridge Deformation Monitoring with Fiber Optic Sensors

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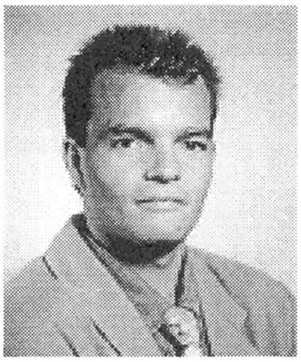

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Abstract

The security of civil engineering works demands a periodical monitoring of the structures. In many civil structures like bridges, tunnels and dams, the deformations are the most relevant parameter to be monitored. Fiber optic deformation sensors, with measurement bases of the order of one to a few meters, can give useful information both during the construction phases and in the long term.
SOFO is a structural monitoring system using fiber optic deformation sensors. It is able to measure deformations between two points in a structure, which can be from 20 cm up to 10 meters (or more) apart with a resolution of two microns (2/1000 mm) even over years of measurements. The system is composed of optical deformation sensors adapted to direct concrete embedding or surface mounting, the cable network, the reading unit and the data acquisition and analysis software. The system is particularly adapted to precise short and long-term monitoring of deformations. The SOFO system is successfully used in a number of bridges, tunnels, dams and geostructures.
This paper briefly summarizes the measurement principle of the SOFO system and presents two examples of application to the monitoring of existing and refurbished bridges. The Lutrive concrete bridge was instrumented with 40 sensors to monitor its curvature variations under static, dynamic and thermal loading. From the curvature measurements and using a double-integration algorithm it was possible to retrieve the vertical displacements of the bridge. The Versoix bridge was instrumented with more that 100 sensors to monitor the different phases of its enlargement and refurbishment. The sensors were used to characterize the interaction between concrete of very different ages and to follow the horizontal and vertical displacements of the bridge during construction, static loading testing and in the long term.
The combination of adequate monitoring techniques and numerical simulations is a powerful tool that enables the understanding of complex structural phenomena. In this way the design of more durable structures can be enhanced.

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Monitoring of Concrete Structures with Integrated Bragg Grating Sensors

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1. Introduction

Worldwide a lot of research is carried out concerning the (remote) monitoring of civil engineering structures to improve their durability and safety. This paper focuses on the use of optical fiber sensors for the purpose of remote monitoring. The main advantages of optical fiber sensors are their insensitivity to corrosion and electro-magnetic interference and their small size. Some optical fiber techniques even have the possibility to use more than one sensor in one fiber which enables quasi-distributed sensing or even continuous measurements. The sensor used for the tests reported in this paper is a Bragg grating sensor which consists of a periodical change in refractive index written in the core of part of a single mode optical fiber. When broadband light is coupled into the fiber one will receive a small reflection peak centered around a certain wavelength called the Bragg wavelength. This wavelength depends on the period of the grating which will change due to mechanical and/or thermal loading. This paper shows the results of tests to determine the temperature effect on the sensor and will give a comparison between strains measured with an electrical strain gauge and strains measured with Bragg sensors. The executed tests are tensile tests on steel rods and a bending test on a concrete prism with an integrated Bragg sensor. In order to demonstrate remote measurement capabilities, as required in many civil engineering applications, a remote monitoring test set-up has been implemented. In this set-up the optical spectrumanalyser (used to measure the reflection peak), the test object and the monitoring PC are connected to each other via optical links of about 200 m. The exact location of each element is basically irrelevant as long as they are linked to each other. It is clear that such a set-up is an essential step towards a permanent remote monitoring system of civil engineering structures in combination with a novel method of strain measuring.

2. Temperature sensitivity of a Bragg grating sensor

When a Bragg grating is subjected to strain the peak wavelength will change linearly. This enables us to use a Bragg grating as a sensor. However a change in temperature will change the refractive index of the grating and will also cause a linear change of the peak wavelength. In order to be able to distinguish both types of wavelength shift during a strain measurement on a test object in an environment with changing temperature, it is necessary to determine the shift of a free Bragg sensor caused by temperature effects. Tests were performed on free Bragg gratings as well as on a Bragg grating attached to a rebar. By determining the shift of the free grating caused by a changing temperature the coefficient of thermal expansion of the rebar could be checked. The test gave a coefficient of thermal expansion of $10.2 \cdot 10^{-6}$ /°C, which is in good agreement with the normally assumed coefficient of thermal expansion for steel of $10 \cdot 10^{-6}$ to $11 \cdot 10^{-6}$ /°C.

3. Tensile tests on steel rods

In order to compare strains measured with a Bragg grating to strains measured with electrical strain gauges simple tensile tests on steel rods were performed. In order to have a controlled manner of attaching the fiber to the rebars a groove of 1 mm width and 1 mm depth was milled in the rebars. The sensor could easily be put in this groove and the groove was then filled with epoxy resin. Attention should be paid that the resin shows no visco-elastic behavior and no hysteresis. An example of a tensile test (loading and unloading) is given in figure 1.

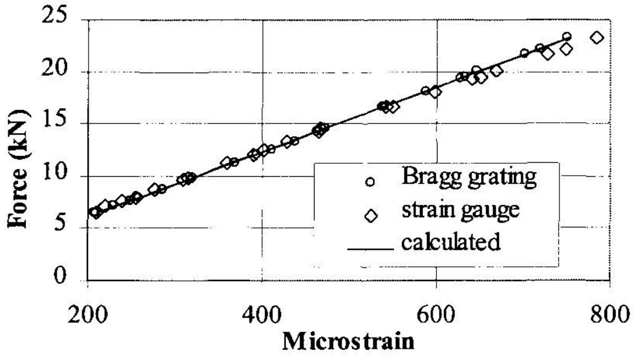

Fig.1 Tensile test on a rebar (ϕ = 14 mm)

4. Bending test on a concrete prism

After performing a tensile test in the elastic zone, a rebar was integrated in a concrete prism of 600 × 150 × 150 mm. The prism was subjected to a 4 point bending test. The strains measured by the grating and the ones measured by the electrical strain gauge were almost identical during the load cycle. A slight difference occurred when the prism was unloaded. The prism was subjected to several loading cycles. The signal of the Bragg grating after each loading cycle was constant while the signal of the strain gauge varied within a range of a 110 $\mu\varepsilon$. The first load cycle is shown in figure 2.

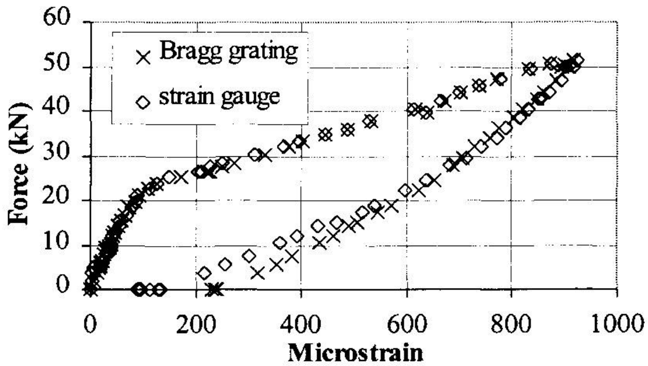

Fig.2 Four point bending test on a concrete prism

5. Conclusion

This paper has shown the results of tests to determine the temperature sensitivity of a Bragg sensor. The results of strain measurements with Bragg gratings during tensile tests on steel rods and a bending test on a concrete prism show that gratings are adequate strain sensors allowing integration in concrete structures. All test were performed in a remote monitoring set-up.

New Applications of Fiber Optic Sensors for Structural Monitoring

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Introduction

The civil structures should be constructed and/or reinforced by materials that are significantly lighter, tougher and longer lasting, as FRP (Fiber Reinforced Polymer). Such materials is brittle, so they doesn't forewarn his failure, then it is important that such structures are continuously remotely monitored in order to communicate their performance under environmental and loading conditions. This allows engineers to check on a daily basis for a possible message of warning or of damage that could have a cost both in human live and in economical field. This paper describes an experimental program in order to assess the possibilities and the limits of fiber optic sensors (FOS) in structural testing and monitoring.

Experimental

The fibre optic sensor employed is designed around a FPI: Fabry-Pérot interferometer. The experimental program carried out consists in tests in both static and dynamic field, in order to achieve information relative to the performances of FPSs, compared to traditional strain gauges. Specimens made of steel and composite materials (CFRP) have been equipped by FOSs and electrical strain gauges (ESG). Tensile and flexural tests were carried out, in order to compare the two types of strain sensors. Dynamic tests too have been accomplished, with the aim to verify the possibilities and the limits of FOSs in the dynamic low frequency field. Finally, a beam of a RC bridge has been equipped with a FOS applied to a rebar, in order to check the capability of such sensors in monitoring the effect of temperature variations.

Results and Discussion

In figg. 1 and 2 we can see the data from different loading tensile cycles carried out on the steel plate and on the CFRP laminate, respectively.

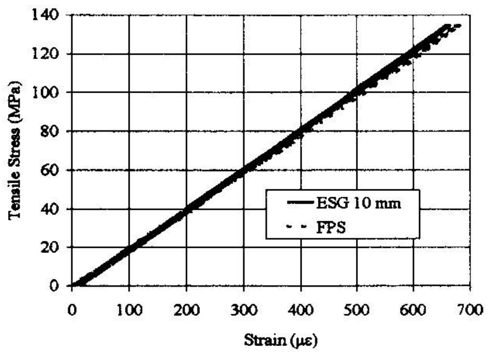

Fig. 1 Stress-strain curves for the steel plate

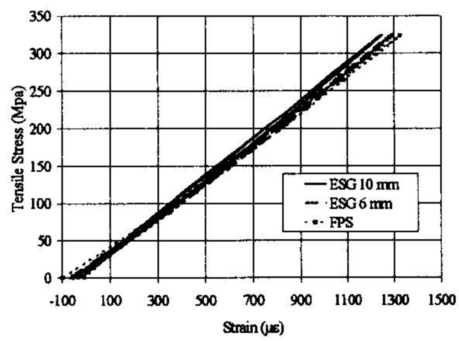

Fig. 2 Stress-strain curves for the CFRP laminate

Figures 3 and 4 reports the absolute value of the strain Vs the total applied load, in bending tests. The optic fiber device used in this experimental work wasn't designed for measures in a dynamic range, because of his low sampling rate (10 Hz). Nevertheless we carried out some tests in a low frequency range, which is very common in a wide range of civil structures. In Fig. 5 you can see the time domain signal of the FPS, compared to the one by the ESG. In the frequency domain (see Fig. 6) we can observe the same values given by the ESG, if we avoid the first frequency peak, which is affected by aliasing errors. When the natural frequency is lower than 5 Hz, equal to a half of the sampling rate of the device, the test becomes significant .

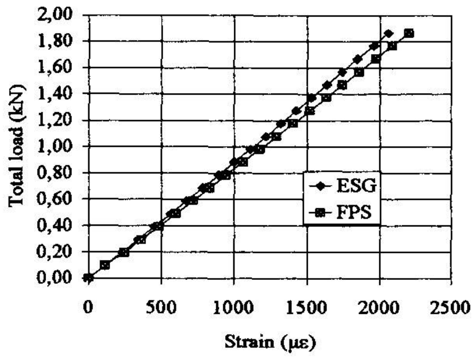

Fig. 3 Absolute value of the strain measured by FPS and ESG, when the FPS is in tension

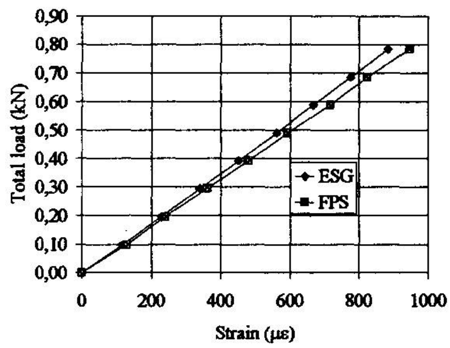

Fig. 4 Absolute value of the strain measured by FPS and ESG, when the FPS is in compression

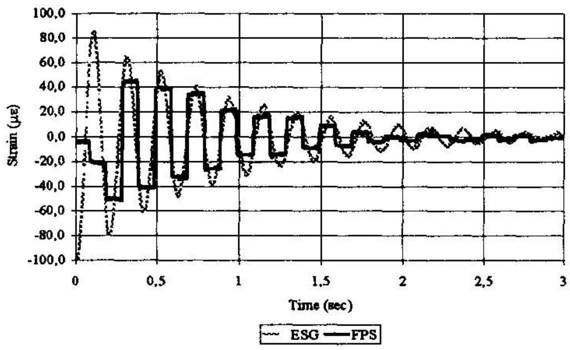

Fig. 5 Time domain signal of the FPS

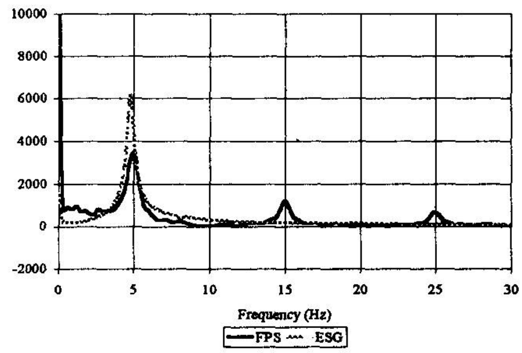

Fig. 6 Frequency domain signal of the FPS

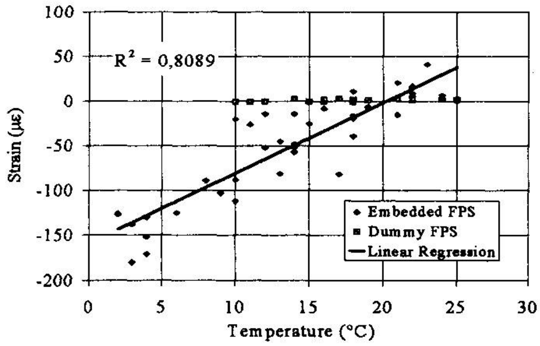

Fig. 7 Monitoring of a steel rebar of a bridge RC beam

Fig. 7 shows the results of the monitoring carried out on the bridge RC beam. The correlation between strain and temperature is linear with a good regression coefficient. The dummy FPS is not influenced by the temperature variations. The repeatability is very good. This is an important result, if we take into account that the device were disconnected and reconnected to the sensor at each reading. This is due to the very good coupling system, and is an important property of this type of sensor.

Conclusions

Fabry-Pérot Sensor (FPS) strain gauges has been tested in several conditions. Tension tests carried out on a steel specimen showed the agreement of the results between FPSs an traditional Electrical Strain Gauges (ESG). Identical tests on a CFRP specimen showed some differences between the two types of sensors, especially when 10 mm ESG are used. Also bending tests on a steel specimen showed good agreement between the two types of sensors, both in tension and in compression. Dynamic tests in a low frequency range confirm the possibility of using FPSs as vibration transducers, if an acquisition system with suitable sampling rate is used. As monitoring system, the fiber optic based system has demonstrated very good performance in terms of sensitivity to strain, repeatability of the readings and insensitivity of the sensor to thermal effects.

Monitoring of a Highway Bridge Reinforced and Prestressed with CFRP

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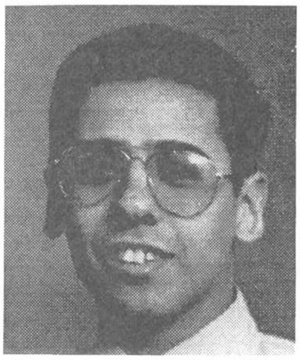

Summary

This paper discusses a comprehensive system used to monitor a highway bridge in Canada reinforced with fibre reinforced polymer (FRP). Taylor Bridge, the longest smart bridge in North America, includes four girders, the deck slab and the barrier wall reinforced using carbon and glass FRP materials. The Taylor Bridge is remotely monitored using fibre optic sensors embedded in the girders, the deck slab and the barrier wall to provide continuous information on the structural performance of the bridge. The signals obtained from the optical sensors are transmitted through a telephone line, allowing an office-based engineer to monitor the stresses and strains via a computer anywhere in the world. The paper discusses the expert system program used to reduce the data collected from the bridge into engineering information which can be used to assess the performance of the FRP material and the behaviour of the bridge. Design philosophies and construction techniques used for the bridge to handle these new materials are also presented.

Bridge Description and Design Philosophy

Due to a lack of codes and standards on the use of fibre reinforced polymer (FRP) as reinforcement and prestressing materials for concrete bridges, an experimental program included testing of a large scale model of a bridge girder totally reinforced with carbon FRP and a full scale portion of the deck slab at the University of Manitoba. The results were used to design the Taylor Bridge which opened to traffic in October 1997. To obtain continuous information on the behaviour of the bridge and the performance of FRP as reinforcement and prestressing tendons, the bridge is monitored to provide essential data for long-term behaviour durability. The Taylor Bridge is considered to be the world's largest highway bridge reinforced by FRP and monitored using fibre optic sensors. The 165.1m-long bridge consists of 40 prestressed concrete AASTO type girders. Four girders of the Taylor Bridge were prestressed by two different types of carbon fibre reinforced polymer (CFRP) material using straight and draped tendons. The girders were also reinforced by CFRP stirrups protruded from the AASHTO type girders to act in composite action with the bridge deck. A portion of the deck slab is reinforced by CFRP reinforcement. Glass fibre reinforced polymer (GFRP) was also used to reinforce the barrier wall. The barrier wall is connected to the deck slab with double headed stainless steel bars.

Monitoring of Taylor Bridge

FBG sensors were used to monitor the strains in the CFRP reinforcement of the girders and the deck slab of Taylor Bridge and in the GFRP reinforcement of the barrier wall. Selective girders reinforced by conventional steel reinforcement were also instrumented using FBG sensors. The FBG sensors used in the Taylor Bridge were fabricated by ElectroPhotonics Corporation, Toronto, Canada, and had a full range of 10,000 microstrain. The FBG sensors produced by ElectroPhotonics Corporation were used in concrete structural models and calibrated in several tests at the W. R. McQuade Laboratory at the University of Manitoba.

Preliminary results recorded by ISIS Canada researchers at the University of Manitoba are reported in this paper. The collected sensors' data addresses the following stages:

a-Construction stage
b-Load testing of the bridge
c-Long-term behaviour due to the temperature effect

Conclusion

A sophisticated network of fibre grating sensors has been successfully deployed in the Taylor Bridge, Headingley, Manitoba. The optical sensing system is used to remotely monitor the bridge structure, giving the bridge engineer a warning signal if abnormal conditions should occur. This project provides an example for the practical issues of design and implementation of such a system for long-term structural monitoring. The FBG sensors' data are processed using an interactive software package, specially implemented for this purpose. Preliminary data collected from the bridge shows that such a monitoring system can prove to be a very effective tool for the bridge engineer. Monitoring of the Taylor Bridge provides essential data related to the short-term and long-term performance of FRP material used to reinforce the bridge members. In summary, the monitoring system can provide a profile of the bridge, with detailed information on its structural behaviour, as well as the applied loads and environmental effects.

In Situ Stress Evaluation of Reinforced Concrete Elements

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Summary

In the structural assessment of concrete structures, the evaluation of the existent stresses is essential for the quantification of the structure's residual strength and for the optimisation of a future repair operation. This paper presents the "Reinforcement Release Method", a method developed at IST for the assessment of in situ stresses in reinforced concrete elements, based on the stress release of reinforcement bars.

Keywords: In situ assessment; reinforced concrete; columns; stress release.

1. Introduction

Structural assessment is performed in concrete structures if anomalies occur (cracks, excessive strains, deterioration) or if the strengthening of the structure must be performed due to changes in the design conditions. In these cases the evaluation of the existent stresses is particularly important for the design and construction engineers since the actual level of the total stresses is one of the major parameters in the assessment of the structural residual capacity.

The existing methods for experimental assessment of in situ stresses in concrete structures are essentially based in two techniques [1]: local and partial release of the stresses followed by a controlled pressure compensation (direct method); relief of stresses around either cylindrical inclusions or cores extracted from the structure (indirect method).

The "Reinforcement Release Method", presented in this paper, was developed to quickly perform the assessment of in situ stresses in reinforced concrete elements. The use of the technique is mainly restricted, in this initial phase, to columns where compression stresses due to the axial loads are paramount.

2. The Method

The proposed technique is based essentially on the determination of the axial force in columns through the local and total stress release in one of the reinforcement bars of the column. This release begins with the opening of a groove in the concrete, around the bar. This operation must expose completely some centimetres of bar. Afterwards, the exposed reinforcement bar is cut with an appropriate disk *(Fig.1)*. The strain released in the bar during the cut is recorded through electrical strain gauges, previously glued, in a position close to the cut section. The existing total stress in the bar is evaluated from the change of strain. The technique is completed by welding the bar and the groove is closed with mortar to guarantee the ultimate strength of the analysed column.

To obtain, more accurately, the stress in the bar due to the existing applied loads (external component) it is necessary to perform a complementary analysis. This includes an evaluation of the self-balanced stress components due to the thermal-hygrometric behaviour of the concrete, and of the components associated with disturbance due to the concrete opening.

The technique was calibrated in reinforced concrete specimens that were subjected to compression under a known axial force, that was later compared with the axial force evaluated through the method [1].

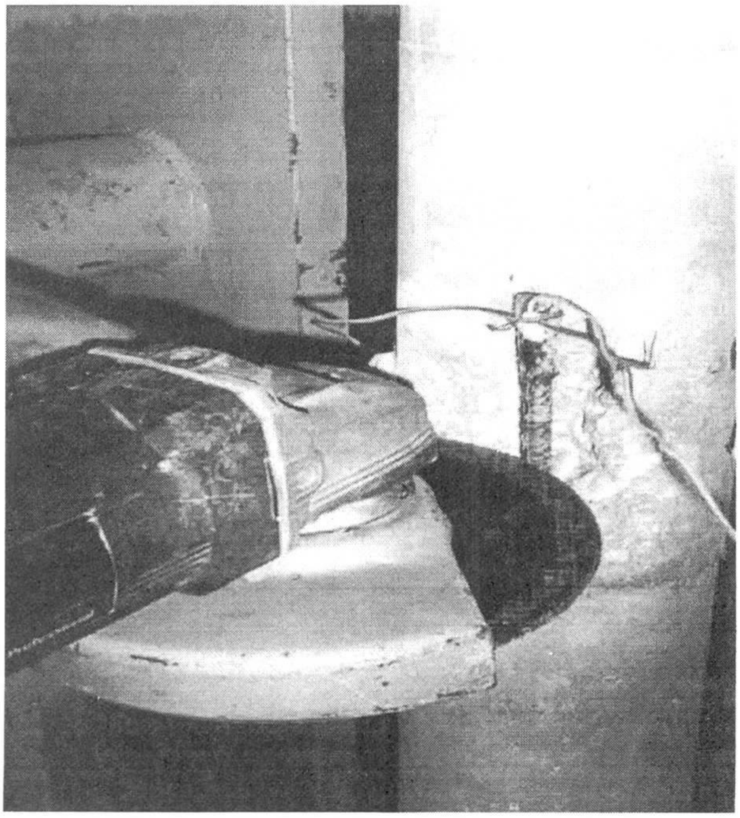

Fig.1 Electrical saw cutting the bar [1]

3. Concluding Remarks

The major conclusions of the "Reinforcement Release Method" are:

1) The stresses obtained with the direct measurement of the strain variation while the cutting a reinforcement bar are a maximum of the existing stresses;
2) More accurate results can be obtained if the E_c value is measured in situ;
3) More accurate results can be obtained if creep and shrinkage effects are considered. The opening area effect can also be considered;
4) The great advantage of this technique is the technology simplicity.

4. References

[1] Santos, J. R. "In Situ Stress Evaluation of Reinforced Concrete Columns", *MSc. Thesis in Structural Engineering*, 1997, Technical University of Lisbon, Lisbon.

Thermovision - An Efficient Tool for Monitoring of Steel Members

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Summary

The monitoring of structures involves the measuring of aspects of their performance. For the prediction of future damages (cracks) the detection of high stress concentration areas is essential. The thermographic stress analysis with thermovision is a full-field experimental technique, which can be applied for this task. The method allows a fast overview about the principal stresses in a cyclically loaded steel member, the detection of dangerous stress concentration areas and the monitoring of the carrying behaviour of the member. The paper reports experience from 4 tests carried out at the Brandenburg Technical University, Cottbus (tension bar with opening, necked hot-rolled beam, plate girder with slender web, rosette press joint).

Keywords: monitoring, experimental stress analysis, thermovision, infrared detection.

1. Introduction

For controlling the performance of a structural element, a full-field stress pattern is an efficient tool, because the difference with respect to the predicted stress distribution can be observed rapidly. The conventional methods of experimental stress analysis (e.g. strain gauges, photoelasticity, etc.) require complex preparation, and are not always effective for complex surfaces. For this reason a contactless technique was chosen, a thermal emission method with conventional thermocamera. The possible applications of this method to monitoring purposes were tested and the results were compared with data from strain gauges and FEM calculations.

2. Experimental procedure

The emitted thermal radiation of the surface of the loaded specimen is detected by sensitive infrared detectors or cameras. For our experiments the AGEMA Thermovision System 900 was used. The analysis was based on the thermal difference image between the fully loaded and unloaded state of the specimen in one load cycle. The calculation of stresses was done according the classical equation of the thermoelasticity.

3. Applications

The four test represent some possible applications of the method. The tested specimens are shown in *Fig. 1*. (*a*) On the conventional tension plate with central circular hole the stress sensitivity of the system was compared with stresses measured by strain gauges. (*b*) The welded girder with necking was used to compare the geometrical resolution of the thermocamera with FEM calculations through full-field stress maps. (*c*) The ultimate behaviour of the loaded plate girder with slender web was followed and observed with the thermocamera.

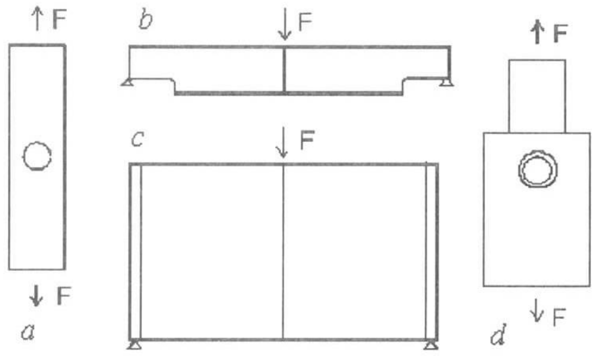

Fig. 1 Tested specimens

(*d*) The investigations on a complicated structural detail confirmed the usability of the thermographic method for complex surfaces. The stresses resulted from thermovision and calculated by FEM are shown in *Fig. 2*. The failure of the specimen was demonstrated in coloured thermal pictures.

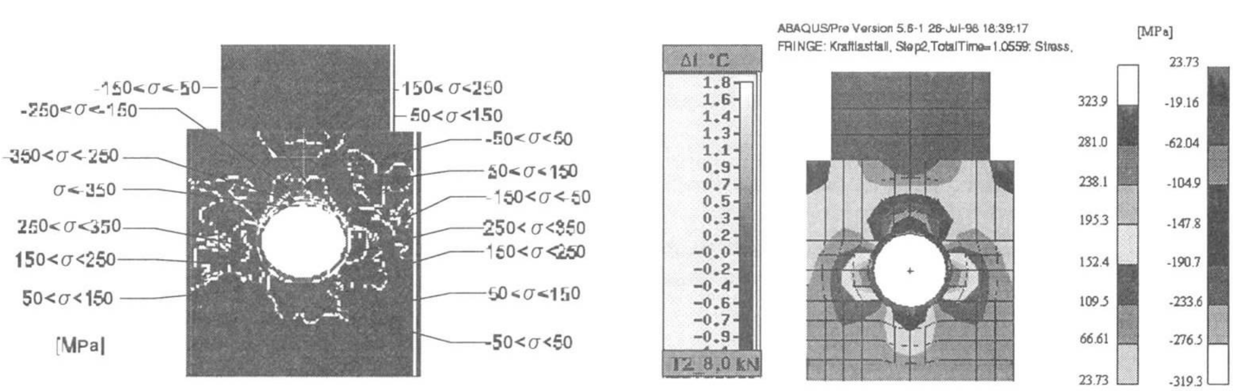

Fig. 2 Stresses from thermocamera and FEM in a rosette joint (Fig.1d) at a load level of 8,00 kN

4. Conclusion

The advantages and disadvantages of the thermographic method according to our experience has been briefly summarised. The thermographic stress analysis with conventional thermocamera can be an efficient tool for the monitoring of steel members. The experimental analysis of structures and details can be performed rapidly. The structural elements can be analysed in full-scale and in their operational environment, no modelling and only small surface preparation is needed. The sensitivity of a conventional thermocamera is sufficient to obtain good qualitative results from the whole structure, and acceptable quantitative results from smaller areas. For the refinement of the method further investigations are necessary (random loading, on-site effects, plastic behaviour).

Monitoring to Become Wiser: A Case Story

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Summary

The last two decades have revealed an enormous development of computers and computer software. This development has given structural engineers very powerful tools such as finite element (FEM) programmes which allow very detailed analyses of new as well as existing structures. FEM analyses are based on assumptions to some extent e.g. static boundary conditions often has to be assumed or the loading is based on codes or qualified assumptions.

The tremendous development of computers and computer software have fortunately also been to the benefit of hardware and software for monitoring and measurement equipment such as dataloggers. Performing measurements on complicated structures, which has previously been analysed by FEM, can provide valuable information to verify whether calculation assumptions. Adjustments to the structure or the service manual can be implemented before the structure is taken into service.

Performance tests on new structures can help prevention of unexpected interruptions of the structural service.

Performing short and uncomplicated measurement routines on existing and new structures exposed to fatigue can give a more reliable prediction of fatigue life.

The increase in transport by railway as well as automobile has been the reason for the strengthening of many existing bridges. By performing measurements, strengthening can in some cases be avoided or postponed due to a clarification of the structural behaviour and/or the loading. Measurements can also serve as documentation for the efficiency of a performed strengthening.

This paper will be based on the following three case stories, which illustrate the benefit from measurements.

- New railway expansion joints on the Great Belt Bridge before being put to service.
- Fatigue life predictions on a bascule bridge which had failed due to fatigue
- Efficiency of a strengthening of a 90 years old riveted railway bridge will be demonstrated. Integrated use of three dimensional non-stationary dynamic FEM analyses will be presented in relation to the performed measurements.

Conclusion

Based on the briefly presented case stories we find that monitoring of structures can be beneficial in the following cases:

- Existing structures which do behave in an unpredictable way. In this case the behaviour can be clarified and the test results can be evaluated e.g. in the same way as if the information has been obtained by FEM analyses.
- Existing structures which have to be upgraded. In this case complicated FEM models can be either calibrated or confirmed by measurements.
- The effect of a strengthening of an existing structure. Performing a measurement before and after a strengthening has taken place, the effect of the strengthening can be evaluated or documented.
- Design assumptions can be confirmed or even determined before the final design is completed.
- The performance of a new structure can be clarified or documented before hand over. E.g. deformations can be documented to be with in specified limits etc.

Monitoring System of Kao-Ping-Hsi Cable Stayed Bridge

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Abstract

This paper presents the items and configurations of monitoring project of Kao-Ping-Hsi Cable Stayed Bridge (Fig.1). The monitoring of critical structural performance during the service life is an essential part of the maintenance program. The instrumentations of monitoring system, not only provide some important information such as wind speed and direction, seismic acceleration, and pylon movements for the traffic control center when the bridge opens to traffic, also provide data of the bridge behaviour during construction stages.

In order to monitor the bridge behaviour during construction and service stages, the items included are the strains of concrete and reinforcing bars in PC girder and pylon, the strains of steel girder, the strains of cable anchorage zone (Fig.2), cable forces (Fig.3), acceleration of ground and the selected locations of structure under seismic, the speed and direction of wind; the displacement of pylon and the variation of forces in earth anchor at abutment, etc.

During construction, the data are collected manually, and after the bridge opens to traffic, they will be collected by automatic data acquisition system. The monitoring system will send the data periodically to traffic control center and the remote research center. In case of unusual structural performance occurred due to the actions of earthquake, strong wind and larger movement of pylon, the automatic data acquisition system will be triggered using shorter scanning period to collect data immediately. The outline of instrumentations of this project is presented.

The measured strains of concrete and steel at the various locations of pylon reasonably reflect the behaviour of pylon during each corresponding construction stage. The temperature distribution in the selected section of the pylon is also presented. The variations of force in the six load cells installed with some of the earth anchors in the abutment were also well correlated with the in-situ conditions. The above data collected during the construction stages are also included.
The Kao-Ping-Hsi Cable Stayed Bridge crossing the main riverbed is one of the special projects of the Second Freeway being constructed and is the largest cable stayed bridge built in Taiwan.

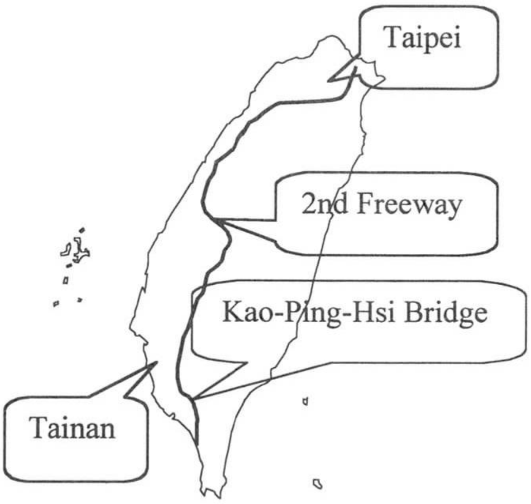

Fig.1 The location of the Kao-Ping-Hsi Bridge

It has a 183.5m high inverted Y shape pylon with varying cross sections and a semi fan type single plane stays with 14 paired cables on each side of pylon. The girder is 45 meters above the riverbed. It is featured with 6-lane 34.4m wide girder which is composed of 180m prestressed concrete (PC) box girder on the side span and 330m steel box girder on the main span. The PC box girder is designed with cast in place concrete and constructed by the advanced shoring technique, whereas the steel box girder is designed with in-situ whole welding for all segments. It has been constructed since April, 1996 and is scheduled to be finished in October. 1999.

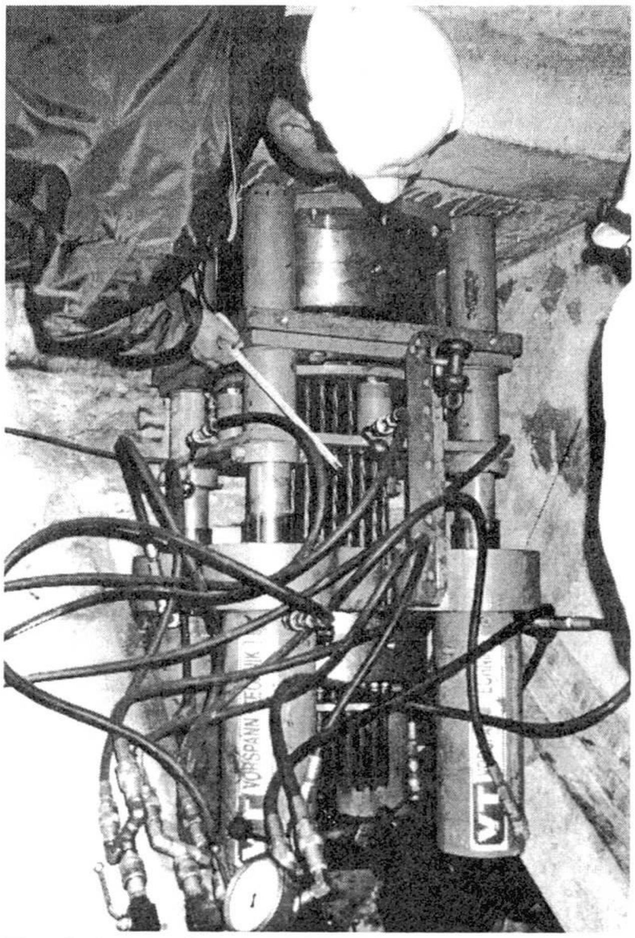

Fig. 2 One-end prestressing of stayed cable in PC box girder

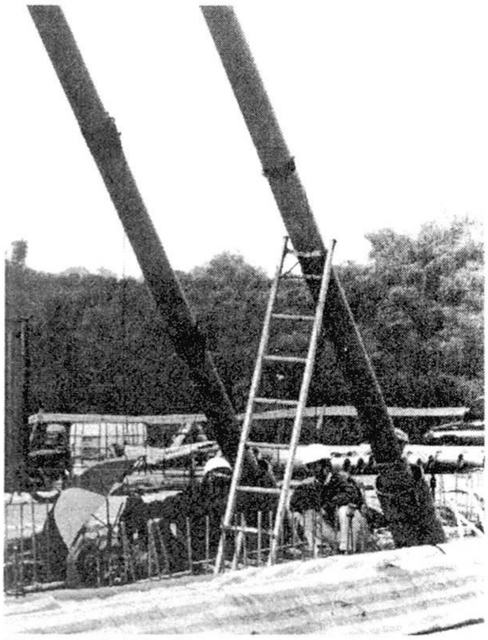

Fig. 3 Cable forces checked by ambient vibration method

Dynamic Test of a Pedestrian Bridge as Part of Safety Assessment

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Summary

The results of the dynamic tests carried out on the approach pedestrian bridge to Civita di Bagnoregio, Italy, are shown. The structural elements were in bad conditions due to the carbonation of concrete. The results of the experimental campaign allowed to state that the structural behaviour of the bridge was quite similar to that we could expect. Under ambient vibrations the structure behaved as a whole, formed by the piers linked at their tops by means of the beams.

Keywords: bridges, experimental dynamic analysis, system identification.

1. Description of the bridge

The approach pedestrian bridge to Civita di Bagnoregio is composed by 14 spans, which are simply supported on the piers. These are spaced by about 19.00 *m*. Starting from Civita we first find a 20% steep slope, which interests five spans. The heights of the piers from pier 1 to pier 5 vary from 11 to 15 *m*, pier 4 being the highest. The viaduct between pier 5 and pier 9 is almost horizontal while in the last part, from pier 9 to pier 13, it shows a 6% slope. Piers from 6 to 13 are much shorter than the others. The girder is composed by three pre-stressed concrete beams and a concrete slab. The total height of the cross-section is 80 *cm*. The horizontal distance between the beam axes is 90 *cm*, the total width of the bridge is 2.50 *m*, included the two longitudinal parapets. Each span has a length of about 16.70 *m*, but the first one is 16 *m*. The beams are supported by the upper plate of the piers by means of leaden pads. All the piers are composed by 4 circular pillars, whose diameter is 50 *cm*, and by an upper plate of 65 *cm* height. The pillars start from a rectangular foundation plate, supported by concrete piles with a maximum length of 25 *m*. The structural elements were in bad conditions due to the carbonation of concrete. All the external beams were damaged, the phenomenon being favoured by the combined action of rain and wind. Several cracks could be seen in the pillars, where the concrete cover was split and the reinforcement bars was uncovered. The upper plates of the piers were damaged too.

2. Experimental dynamic analysis

Recently, the bridge showed vibrations of high amplitude during a funeral, when it was very crowded. The SGM Engineering S.r.l. of Perugia (Italy) carried out, on behalf of Bagnoregio Town, a series of experimental campaigns on the structure, in order to analyse the static conditions and the dynamic characteristics of the bridge and to find out any structural damages. The dynamic tests of the structure revealed a strange behaviour of the piers. The authors, involved in the interpretation of the results of the tests, decided to carried out another experimental campaign, in order to better

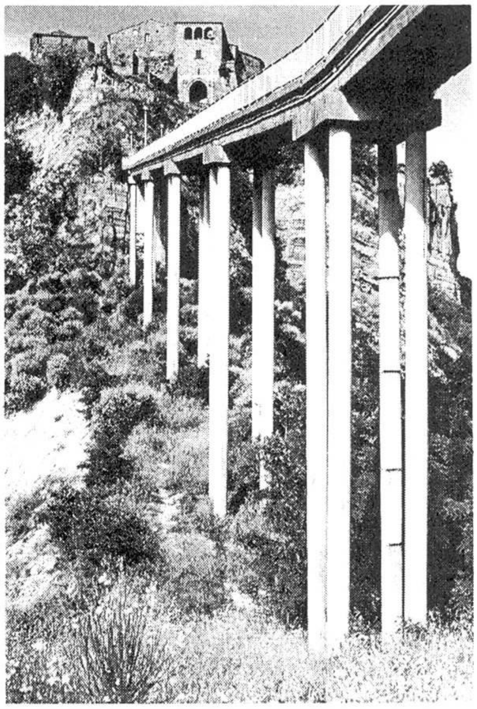

Fig. 1 *Piers 1 to 5*

characterise the dynamic behaviour of the piers. The experimental set-up was composed by eight seismometers Kinemetrix SS1, an HP3566A signal conditioner and a HP laptop. Measurements were carried out on June 1997. Sensors were deployed in several configurations. Both ambient and forced vibrations were considered, the latest being caused by the passing of pedestrians or vehicles of small size. The recorded data were analysed both in the time domain and in the frequency domain.
The recorded data allowed to analyse the behaviour of the viaduct, with particular attention to its part between pier 1 and pier 5. The following frequencies are particularly apparent in the spectra: 0.95, 1.38, 2.07, 2.83 *Hz*. This occurrence suggested that the viaduct behaved as a whole, the records at the piers presenting the same features. Records at the basement of each pier were always in phase with those at the top, at all the found resonance frequencies. Peaks at the same frequencies are apparent in all the cross spectra between the sensors at the tops of the piers, with significant values of the phase factor and the coherence function.
The analysis of the phase factors allowed to find out the modal shapes associated to these resonance frequencies. We concluded that the girder behaved as a beam supported by horizontal elastic restraints at the piers. In the longitudinal direction vibration amplitudes were much lower.
Piers 2 and 3 were particularly analysed. The already observed resonance frequencies were found. We could also state that no torsional modes were associated to these frequencies. The experimental results were compared with those obtained from the analysis of a finite element model. The girder was modelled by means of a spatial beam, having the same geometrical and mechanical properties of the bridge and elastically supported by the piers in the transversal horizontal direction. Vertical displacements and torsional rotations were not allowed in correspondence of the piers. The stiffness of the elastic restraints were equal to the stiffness of the piers. The first four modal shapes of such a model are very similar to the experimental ones. Instead, the resonance frequencies are higher. This occurrence demonstrated that the actual stiffness of the piers are lower than the original ones, relative to the undamaged structure, but the stiffness decrease was almost uniformly distributed along the bridge and not concentrated in a particular element.

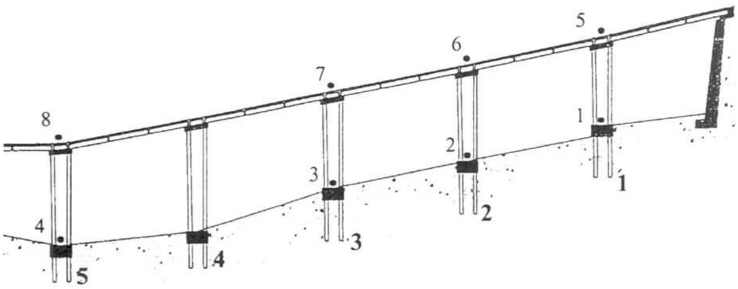

Fig. 2 *Sensor locations in configuration A*

3. Conclusions

The structure showed a very high deformability, more evident in the part where are the highest piers. This characteristic, due to the slenderness of the vertical elements was emphasised by the structural damages, which resulted in a reduction of the effective cross-sections of the pillars. Therefore the viaduct was very vulnerable to wind actions, which are particularly insidious in the area. Besides, when the bridge is very crowded the natural wind flow is obstructed, the vertical structures very stressed and the vibrations could be amplified very much.
We suggested that all the piers should be repaired, in order to be suitable to support the vertical and horizontal loads that can act on the structure. No works were needed on the foundation structures. These may be damaged in the future because of the landslides and the continuous erosion of the soil surface. Therefore they should be monitored or frequently tested.

Monitoring of the New Tagus Bridge in Lisbon

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Summary

The Vasco de Gama Bridge in Lisbon, was opened to traffic in March 1998. The structure is 12.3 kilometres long, including 9 kilometres over the Tagus estuary. The monitoring of such a large bridge requires a design phase necessary to adapt the procedures and instruments to the goals of this monitoring. Analysis carried out by the Contractor and the Designers has led to a system based on networks and standard data acquisition systems and protocols linked to the supervisory control system of the bridge, design which is well known in the industry, but quite new in civil engineering.

Keywords: monitoring, bridges, viaducts, instruments, data acquisition, networks, standardisation.

1. The Vasco de Gama Bridge

The new Tagus crossing, called the "Vasco de Gama Bridge", has been built to cope with increasing traffic bringing the 30 years old « Ponte 25 de Abril » to saturation. It allows to deviate the regional and national traffic around Lisbon linking the North highway A1 to Porto with the Coina Ring and highway A2 to Algarve and Spain. The design of earthquake has been specified to 0.45g horizontal acceleration, representing 4.5 times the assumed acceleration that occurred during the historical earthquake of November 1st, 1755. This specification led to a considerable increase of the capacity of foundations, piers, bearings and expansion joints ; and thus, various seismic devices (buffers, limiters, lateral restraints) have been incorporated in the design.

The 12 300 m continuous bridge is divided into five structures:

- The North viaduct which is 488 m long, 11 spans of 45 m average, is a multiple 3.40 m high T beam deck, with a width varying from 60 to 37 m;
- The Expo viaduct which is 672 m long, 11 spans increasing from 45 m to 62 m, is a segmental double box girder, same height as the North viaduct;
- The Main Bridge, cable stayed, is 820 m long with a 420 m central span over the main shipping channel. It is a totally suspended twin deck (2.60 m high) with longitudinal concrete beams and light steel cross beams every 4.40 m.;
- The Central viaduct is 6531 m long including 9 viaducts of 9 spans of 80 m average length;
- The South viaduct is 3825 m long and has a similar cross-section to the one of the North viaduct.

2. Design of the monitoring

2.1 Objectives and priorities

Objectives of the monitoring of the Vasco de Gama Bridge are multiple : the safety of the users of the bridge, the verification of the consistency between the state indicated by computations and the measured state, the verification of the state of the work after a major event, that is to say an earthquake, a shock of boat or a storm and finally, the confirmation of the period of which it will be necessary to restress the stays of the main bridge, determined by the computation.

Two techniques of measurement are used to answer these targets: high precision topography and sensors (electric sensors or manually read instruments).
The seven types of measurements carried on the bridges and viaducts, consist in meteorological measures, topographic readings, accelerations during earthquakes, concrete strains, temperatures, rotations, and opening of expansion joints between viaducts.

2.2 Topographic survey

The topographic measurements use high precision levelling and triangulateration (angles and distances) and DGPS.

2.3 Sensors

Vibrating wire extensometers for the measure of concrete strain (169), biaxial inclinometers (16), sensors of temperature (54), are linked to networks of data acquisition and can be read with the help of a microcomputer. Sensors of opening of expansion joints (6) are read automatically by computers of the central station that uses a fibre optic network. A specific data processing (removal of temperature effects) is applied to the readings of the opening of the expansion joints.

2.4 Earthquakes monitoring

The earthquakes monitoring includes triaxial strong motion accelerometers (27) that continuously supervise movements of the bridges and viaducts. This data acquisition unit assures the monitoring of accelerations and their recording in case of the threshold of a programmed acceleration.

2.5 Data Acquisition System

Industrial standard procedures and well documented protocols have been adopted. They simplify the design, the installation and the data processing. On the other hand, modules for the specific vibrating wire instruments have been especially developed. Data acquisition and recording use the software ItelTage, a specific module of the modular suite ADAMAS Itelos97, a suite designed for advanced telemonitoring of buildings, bridges, dams, natural slopes, power plants, environmental parameters, research tests and industrial facilities.

The table summarises the types of instruments, data acquisition units, networks used in the Monitoring System.

	Data acquisition unit	Recording and data processing	Link
Meteorological instruments	Industrial modules	Central PC	Fibre optic
Temperatures	Industrial modules	PC NoteBook*	Local network (wires)
Rotations	Industrial modules	PC NoteBook*	Local network (wires)
Concrete strains	Industrial modules	PC NoteBook*	Local network (wires)
Expansion joints	Industrial modules	Central PC	Fibre optic
Temperature joints	Industrial modules	Central PC	Fibre optic
Earthquakes monitoring	Specific data acquisition unit	internal PC NoteBook*	no

Legend: PC Note Book* used to transfer and record data.

The results of the measurements during operation are recorded in a data base used for easy data management and interpretation. The load tests and the first survey readings confirm the advantages of the structure adopted here, which can be easily upgraded or modified at every moment.

Monitoring of Maslenica Bridge during Construction

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Summary

The analysis of Maslenica concrete bridge monitoring has been carried out. The bridge was finished in 1997. The main girders of the superstructure and the arch were analysed during different construction phases. It appears that there was a considerable shrinkage of concrete in girders after their pouring because of a relatively small moisture level at summer time when the measurements were undertaken. This caused compression in the reinforcement. The stiffness of girders was determined by means of the modulus of elasticity, by measuring their natural frequencies, and by means of concrete compression strength. By measuring stresses and displacements during construction and by comparing it with the values obtained by means of NELIN program which includes the material and geometrical non-linearity, a solid qualitative correspondence between theoretical and experimental research results has been established.

Keywords: monitoring, non-linear analysis, modulus of elasticity, stresses, displacements

1. Introduction

The new Maslenica bridge was built approximately 1,5 km west from the old destroyed bridge, over Zdrelac strait near Maslenica. The bridge completely satisfies all the requirements of a normal highway cross section. The main structure of the bridge is a concrete arch with a span of 200 meters and a rise of 65 meters. This paper includes the analysis of stress conditions, deflections and the stiffness of superstructure main girders as well as the conditions of arch stresses and displacements.

2. Monitoring of prestressed girders

Figure 1 shows the changes of stresses, the temperature of the girders' reinforcement and the temperature of the environment immediately after the assembly of a reinforcement cage and during casting and prestressing of superstructure main girders. Before the casting of girders, compression in the reinforcement increases with the fall of the air and reinforcement temperature. Within that period of time, the changes of the reinforcement temperature follow the changes of the environmental temperature. Because of the intensive process of concrete hardening during casting, the relief of thermal energy occurs in a relatively short period of time, which causes a quick rise of the concrete and reinforcement temperature. In the presented case, the reinforcement temperature gradient was approximately 15°C degrees, which caused the increase of compression in the reinforcement for approximately 55 MPa. These stresses return to the starting condition with cooling down of concrete after the hardening process. After the concrete hardening process with the fall of extreme air temperatures, compression in the reinforcement increases. In

Dalmatia, relatively high environmental temperatures often appear already in May with regularly less than 50% moisture, which causes a considerable shrinkage of concrete. This process of concrete shrinkage and the phenomenon of compression in the reinforcement have been registered from the course of stressing after girders' casting and the concrete hardening process. From the function of built-in stresses that is shown bellow, one can notice that about 15% of the allowable stresses were registered before prestressing. The analysis of air temperature gradients and the hardened concrete temperature gradient shows that there are considerably smaller daily temperature changes of the reinforced concrete structure because of slow warming up and cooling down of concrete. Therefore, for example, on 16 May 1996, the measured air temperature differed from 18 to 27°C degrees (ΔT_{air}=9°C), and the concrete temperature from 20,5 to 24,5 C ($\Delta T_{concrete}$= 4°C).

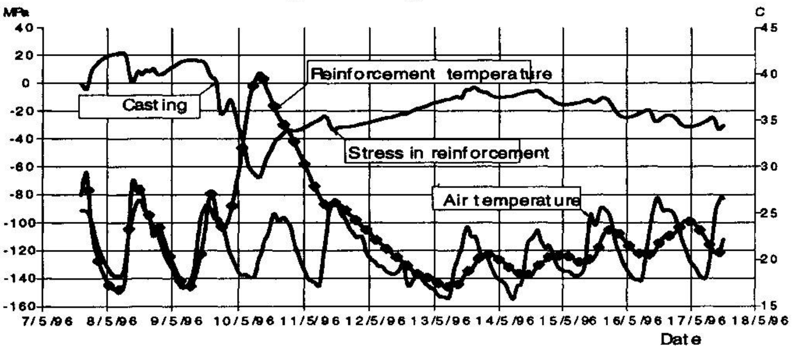

Figure 1. Changes of stresses, girders reinforcement temperature and environmental temperature from the assembling of the cage to the prestressing of girders

This paper contains the analysis of the stiffness of superstructure main girders. The bending stiffness of main girders can be determined by means of the dynamic modulus of elasticity. This modulus was obtained by measuring the frequencies (f) of simple spanned girders from the following equation:

$$E_d = \frac{4f^2qL^4}{\Pi^2 Ig} \tag{1}$$

The dynamic modulus of elasticity was compared with the tangent modulus which was determined on the basis of the tested and designed concrete strength.

3. Arch displacements

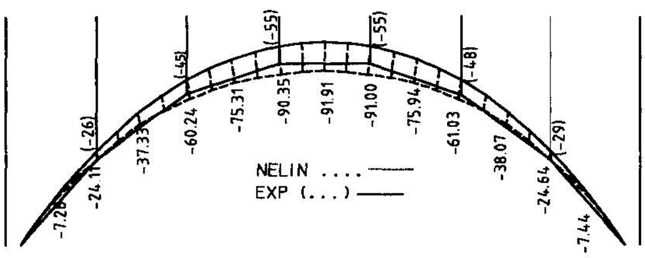

Figure 2 shows differences in displacements between the completed arch and the end of the bridge construction. The displacement difference is determined geodetically and by NELIN program. Comparing experimental and theoretical values of displacements at the all places of the pier and arch junction, averagely 28% higher theoretical valueas have been obtained.

Figure 2. Arch displacements between the completed arch and the end of bridge construction, determined geodetically and by means of NELIN program

4. Conclusion

By measuring displacements and stresses during construction and by comparing the values with the ones that were obtained by means of NELIN program, a solid qualitative correspondence between theoretical and experimental research results has been established. When comparing experimental and theoretical values, one should take into consideration that the theoretical calculations were not completely following all the construction stages of the arch.

Monitoring of a Bridge over the Paraná River during the Launching Phase

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Abstract

The railway bridge spanning the Paraná river, located between the states of São Paulo and Mato (Brazil), plays an important socioeconomic role for a vast region of the country and is one of the largest national engineering works of the last few years.

The road/railway bridge spanning the Paraná river located between the states of São Paulo and Mato Grosso do Sul (Brazil) is the major bridge combining road and railway systems to be built in Brazil, it consists of a 2,600m long steel structure (twenty-six 100m spans), besides two 720m and 450m concrete access viaducts located, respectively, on its left and right sides.

Of the total of 29,000 cubic meters of concrete used, 25,000 were used for the foundation, which consists of large 25 to 62m water column concrete-filled steel pipe piles. The entire 20,650 ton steel structure was made of high-strength low alloy steel (high resistance to corrosion).

The steel structure is made up of two truss beams, with the railway and roadbed, respectively, at the upper and lower chords. Four parts in earth, two 600m long (six spans) and two 700m (seven spans) long, were built on the ground and then push out to their definitive position.

In the launching phase, the ends of the spans reached a 100m cantilever, and exerted a strong force that was concentrated on the lower chord at positions far from the nodes. One of the spans was instrumented using strain gages and, during the launching operation, strains were measured and the stresses evaluated at several points of the structure and compared against the theoretical values.

One of the parts was monitored with strain gages. Strains were measured and stresses were evaluated at several points of the structure during the truss launching phase and then compared to the theoretical values.

The objective of the experimental analysis was to evaluate the strains in the steel structure during the launching phase, comparing the results with those obtained in the theoretical analysis, thus verifying the suitableness of the theoretical model and the launching procedure.
Six of the truss bars were instrumented with strain gages to measure the strains of the steel structure described in table 1. A temperature compensating strain gage was placed on each bar, fixed to a small plate made of the same material as the structure. Figure 1 indicate the position of the bars and the instrumented points.

Table 1 – Description of the instrumented bars

Bar	Description	Number of points instrumented
1	Lower chord – left truss	17
2	Upper chord – left truss	12
3	Diagonal – left truss	12
4	Lower chord – right truss	11
5	Upper chord – right truss	6
6	Diagonal – right truss	6
Total		**64**

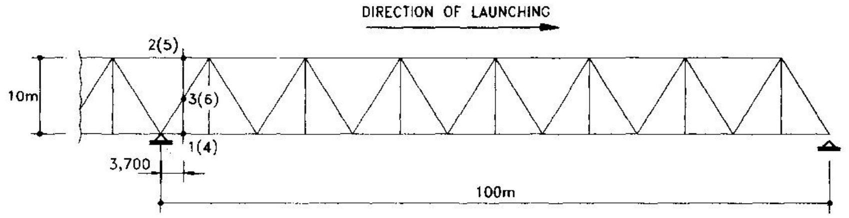
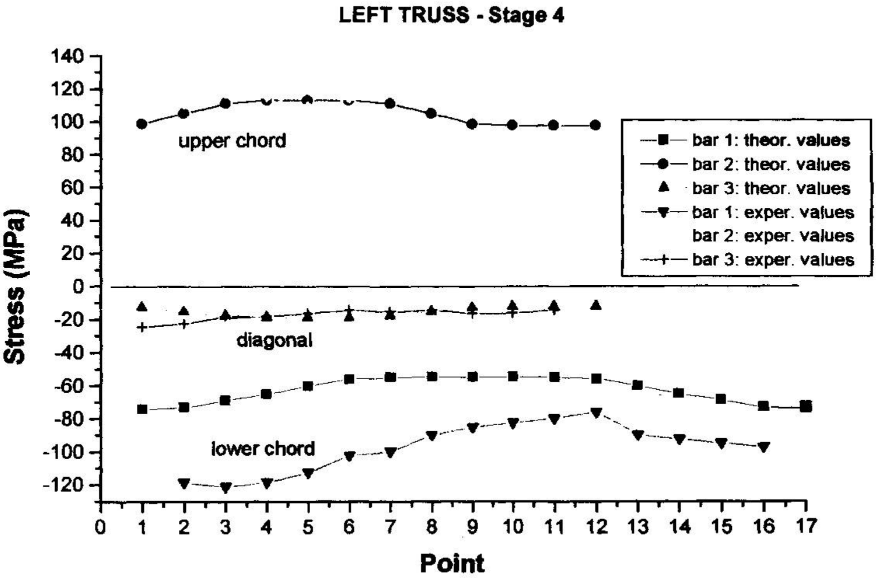


Fig 1 – Location of the instrumented bars (1 to 3: left truss / 4 to 6: right truss)

The stress were evaluated based on the measured strains, assuming the steel presented a linear elastic behavior, with modulus of elasticity E = 205.000MPa. Graph 1 illustrates the theoretical and experimental values for strain in the three bars of the left truss.



Graph 1 - Theoretical and experimental strain values in stage 4 - left truss

To conclude, it can be stated that the stress evaluated experimentally in the launching phase of the steel structure were below those of the limit of proportionality of USI-SAC 50 steel, and that its average values presented relatively small differences in relation to the theoretically evaluated average values. This leads us to believe that the procedure employed to launch the steel structure was entirely satisfactory and did not jeopardize the structure's safety.