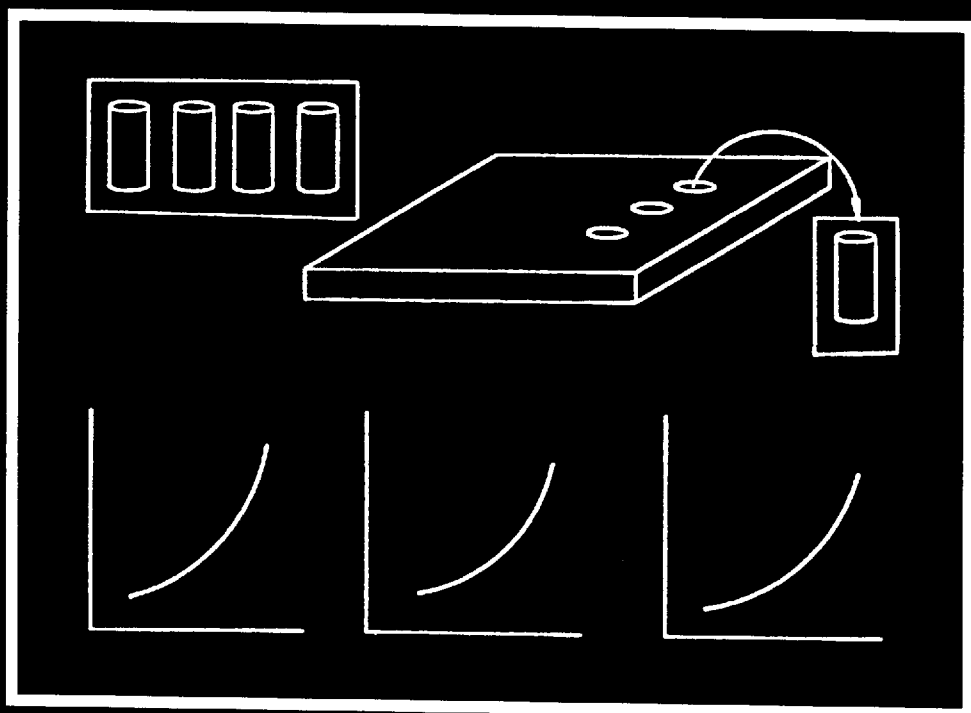


NEW EXPERIMENTAL TECHNIQUES FOR EVALUATING CONCRETE MATERIAL AND STRUCTURAL PERFORMANCE



David J. Stevens
Mohsen A. Issa
Editors



SP-143

PREFACE

Concrete is a truly unique material, exhibiting a wide range of mechanical, physical, and chemical properties, which in turn, are affected by the type of load condition, the constituents, the local environment, the processing method, the structural application, etc. Because of this complex behavior, it is crucial that accurate and meaningful experimental methods be developed and used, in order to efficiently utilize concrete, to guarantee the public's safety, and to minimize cost. This is particularly true in the 1990s, as new and novel concretes, admixtures, and reinforcements are developed.

In recognition of the need for a coherent approach to experimental testing issues, the Technical Activities Committee granted the request of Committee 444 to be renamed and given a new charter. In 1992, Committee 444 (previously called "Models of Concrete Structures") became "Experimental Analysis of Concrete Structures," with a charter to: "Promote and facilitate application of experimental analysis for the evaluation and design of concrete structures." This special publication represents the first undertaking of the revitalized committee in support of this new mission.

This volume contains papers presented at two technical sessions held in 1993 in Vancouver, British Columbia, and in Minneapolis, Minnesota. State-of-the-art techniques, devices, and instrumentation for assessing the performance of concrete structures in the field and in the laboratory were presented. Additional papers, which were not presented, are also published herein. All papers were peer reviewed, per ACI requirements, by experts within and without the committee. These original, reviewed and revised papers constitute this ACI Special Publication.

The topics include novel methods for in-place concrete strength measurements, image processing of the topography of damaged surfaces, correlations of ultrasonic readings and damage, ultrasonic studies in fresh concrete, large scale impact testing and correlations to laboratory tests, and assessment of the break-off test. New technologies such as fiber optics and self-diagnostic carbon fiber/glass fiber reinforcement are presented. Lastly, field applications of state-of-the-art instrumentation are discussed.

The editors (and session organizers) would like to thank the authors and presenters for submitting these papers and making this special publication possible. Their efforts are greatly appreciated. We also thank the individuals who ably and quickly reviewed the manuscripts. Special thanks

are due to the members of Committee 444 and in particular, to Mr. Kirk Marchand, Chairman of Committee 444, for his support and encouragement in producing this Special Publication and for his leadership of the committee as it undertakes its new mission.

David J. Stevens
Mohsen A. Issa
Editors

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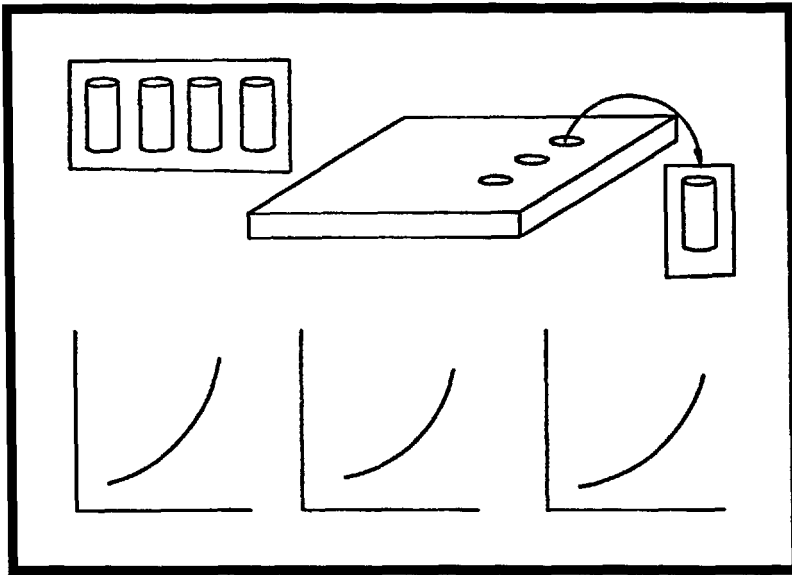
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Fiber Optic Sensor for Measurement of Strain in Concrete Structures

by M.H. Maher, B. Chen, J.D. Prohaska,
E.G. Nawy, and E. Snitzer

Synopsis: A novel fiber optic sensor was tested on reinforced concrete beams with the objective of monitoring strain due to flexural deformations. A Fiber Optic Bragg Grating (FOBG) sensor, developed recently at the United Technologies Research Center (UTRC) for monitoring of strain in structural composites, was used. The FOBG sensor was tested in beam models to measure load-induced strain. Results showed that the FOBG sensor can be accurately and effectively used to monitor strain for both existing and new concrete structures. A number of issues, such as sensor bonding to the structural component, sensor placement and practical instrumentation techniques, were addressed in this study.

Keywords: Beams (supports); deformation; fiber-optic sensor; flexure; gratings; models; reinforced concrete; strains; structures

2 Maher et al

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INTRODUCTION

The existing instrumentation techniques for monitoring the deformations of constructed structural elements, and those to be constructed, are mainly limited to the application of electrical principles using resistance strain gages, linear differential transformers and others. While these have served well, they nonetheless have major limitations in evaluating load-induced strains in facilities that have no prior instrumentation and those that are to be constructed, where remote sensing is needed. The main problems associated with these systems stem from their sensitivity to ambient electrical noise, magnetic fields and environmental degradation. This leads to low sensitivity and, at times, incorrect measurements. For example, using the available systems, monitoring of a reinforced concrete bridge deck, particularly under long-term basis and at several locations, is difficult and expensive.

Recent developments in the field of fiber optic sensors can eliminate many of the deficiencies mentioned above, and permit long-term and reliable monitoring of load-induced strains in constructed facilities. Fiber optic sensor systems may provide engineers with an effective tool for warning against impending failures, and also enable them to better predict the safe life of large structural systems. Although optical fiber strain sensors have been used in the aerospace and automotive industries [1,2,3,13], their potential use in monitoring civil engineering structures have only recently begun to attract attention [12].

Typical fiber optic strain sensors used presently for monitoring civil structural systems consist of microbend sensitive [5,6,12] and interferometric sensors [4,8,14].

When a microbend sensitive fiber is subjected to external pressure, a small amount of light will be lost by leakage from the core to the cladding of the fiber due to the phenomenon of microbending (Fig.1). The light attenuation by microbending can be correlated to the intensity of local stress and strain. Moreover, by using an Optical Time Domain Reflectometer (OTDR), the position of the stress or strain concentrations along the fiber may be pinpointed. The latter device determines the position and induced loss along a fiber by measuring the time delay of back reflections of a light pulse launched within a fiber of known index of refraction which governs light velocity.

Microbend sensitive fibers were used by Fuhr et al. [5] for qualitative monitoring of load-to-failure testing of a reinforced concrete beam. In this investigation, the main concern was the survivability of the embedded fiber optic sensor during the curing process of the concrete. Results of the investigation showed high correlation between fiber optic fault locations and stress-induced failure points in the concrete beam. In a related study, Fuhr et al. [6] instrumented a five-story concrete structure with the objective of monitoring stresses, vibrations and cracks. A number of issues, such as the embedment and placement of fibers in large structural elements, were addressed in this study. Microbend sensitive fiber sensors were also used by McKeehan et al. [12] for real-time monitoring of tensile strain changes in a pipeline in permafrost area. In this case, use of OTDR

enabled accurate identification of the points along the pipeline undergoing excessive settlement.

In fiber optic interferometer sensor systems, any strain induced on the fiber produces a change in the phase of the modulated optical signal propagating over its length. Thus, phase change measurements during loading can be used to calculate the strain of the fiber. Although this concept has been applied to measurement of strain in structural composites in the aerospace and auto industries, its civil engineering application has been fairly limited. While a number of researchers have demonstrated the potential of interferometric sensors in civil engineering applications [4,8,12], their use for direct and quantitative measurement of strain is yet to be investigated.

In general, research in the past few years has demonstrated the potential of fiber optic sensor systems for on-line and accurate monitoring of civil engineering structural systems. Both microbend sensitive and interferometric fiber optic systems have been tried for measurement of strain in structural elements, particularly in concrete. Experience of the authors with microbend sensitive systems has shown that a number of fundamental issues, such as fiber measurement sensitivity, repeatability and practical ruggedization, make these systems difficult to implement for measurement of strain in structural elements, such as reinforced concrete. The need for further research became clear for development of an appropriate fiber optic sensor system for strain measurements.

In search of a sensor that can measure strain without losses, particularly if remote sensing is required, the authors used the Fiber Optic Bragg Grating (FOBG) sensor system developed by Morey et al. [10,11] and Meltz et al. [9]. Measured strains, from both embedded and exposed FOBG sensors on concrete beams, showed excellent correlation with the strains obtained from electric strain gages. A description of the FOBG sensor system and the experimental program with concrete structural elements is described below.

FIBER OPTIC BRAGG GRATING (FOBG) STRAIN SENSOR

Bragg gratings were first discovered by Hill et al. [1978] in standard telecommunications optical fiber by observing a refractive index change caused by the interference of two counter propagating waves in the fiber. The first glass composition to exhibit this property was germanium-doped silica, but since then many other glass compositions have been shown to have similar properties. Meltz et al. in 1989 made it possible to make useful devices through a direct ultraviolet illumination of the fiber core from the side of the optical fiber, the transverse holographic method. The physics of the refractive index change is not completely understood but it is believed to be a photo-induced redistribution of electrons and color centers in the glass.

Since the introduction of such useful devices, FOBGs have been used to build laser cavities, Fabry-Perot interferometers, and sensors. Bragg grating in single mode fibers have been used to measure vibration, strain and temperature. Bragg gratings in two mode

optical fibers have also recently been shown to satisfy certain sensing applications [11]. High sensitivity measurements of strain have been shown by monitoring the phase shift of the Bragg reflection.

A FOBG is a longitudinal periodic variation of the index of refraction in the core of an optical fiber as shown in Fig.2. The spacing of this periodic index variation is determined by the wavelength of the light that is to be reflected. Only light which has a wavelength that is twice the optical path (spacing) is reflected. This results in a very narrow spectral line, approximately 1 Angstrom wide reflection band. The narrowness of this band provides good resolution during measurements. The condition that must be satisfied to get a reflection is

$$\lambda_{\text{Bragg}} = 2 n D$$

where λ_{Bragg} is the wavelength of the reflected light; n is the average index of refraction in the core of the fiber; and D is the spacing of the periodic refractive index variation. As strain is applied to the fiber, the spacing of the periodic variation of the refractive index changes. In addition to mechanical deformation of the grating, the average index of refraction is also changed through the strain optic effect. These two effects work together to cause a shift in the Bragg wavelength reflected for an applied strain. By differentiating the Bragg condition expression and substituting an expression for the strain optic effect, one can express the fractional change in wavelength as a constant (K) times the applied strain (ϵ).

$$\Delta\lambda/\lambda = K \epsilon$$

In the simplest configuration, the spectral shift of the Bragg reflection corresponds to an applied strain or temperature change. This can be measured by using a broad band LED source at 1300 nm launching light into a 3dB coupler with one end of the coupler connected to the fiber with the FOBG (Fig.3). The return end of the coupler is connected to a spectrometer which monitors the spectral shift of the reflection band (Fig.4).

EXPERIMENTAL PROGRAM

Prior to testing of FOBG sensors in concrete structural members, an experimental program was conducted using both single mode and multimode micro-bend sensitive fibers to assess their applicability as strain sensors in concrete. Micro-bend sensitive fibers were embedded in concrete by mounting the fibers on the reinforcing bars. They were also placed on the exterior of the bottom flanges of the concrete beams as exposed sensors. Various methods of fiber mounting on reinforcing bars were tried. These included straight mounting and coiling of the fibers around the reinforcing bars. The concrete beam models were 6"x6"x1'-8" in dimension with #3 size steel bars as reinforcement. The beams were subjected to third point loading for a constant moment zone, and the induced tensile strain in the steel reinforcement was measured by both micro-bend sensitive fibers and strain gages.

Although micro-bend sensitive fibers function well in places where discrete stress concentrations are clearly present, such as those shown in Fig.1, their

performance as direct strain sensors along the length of reinforcing bars in concrete was found to be less than satisfactory. Results of this experimental program showed that issues such as fiber sensitivity, ruggedization and mounting, need to be further investigated if this type of optical fiber sensor is to be successfully used in concrete structures.

Following the testing of micro-bend sensitive fibers, the authors searched for a more practical sensor for application to concrete structural elements. This search resulted in selection of FOBG sensor as a potential strain measurement sensor in concrete. Following some preliminary tests, an experimental program was conducted to assess the feasibility of using these sensors in concrete. Details of this experimental program are described below.

Structural Elements Tested

Fiber Optic Bragg Grating sensors were used to measure strain in concrete beams. Three-point bending tests were performed on small (2.5"x5"x2'-0") and large scale (8"x12"x10'-0") prismatic high strength reinforced concrete beams with embedded FOBG sensor and electrical strain gauges attached to their reinforcing bars. A Fiber Optic Bragg Grating sensor was also tested in an exposed position, mounted on the underside of the large concrete beam, with the objective of evaluating the sensor's performance on existing structures. A schematic diagram showing the cross sectional properties of the tested beams is presented in Fig.6.

The concrete for the beam models was proportioned for a 7-day compressive strength of concrete of approximately 11,000 psi by using Type III cement, and silica-fume as a mineral aggregate. The water-cement ratio was 0.28. The coarse aggregate was crushed stone of 3/8 in. maximum size and the fine aggregate was a natural local sand. Slump was about 6-7 in.

Fiber Optic Bragg Grating Sensors Application

The fiber optic Bragg gratings used were standard single mode telecommunication fibers with Germania-doped cores with the gratings being about 10mm (0.4 in.) in length. In all the embedded cases tested, the FOBG sensors were attached to the reinforcing bars in the concrete beam by placing them in a small V-groove cut on the bar and mounting with epoxy. Ideally bonding of the sensor to the bar should provide good strain compatibility between the two. However, due to the low elastic modulus of epoxy compared with those of glass and steel, some loss in strain can occur during the strain transfer process. In order to evaluate the strain transfer process, attachment of the FOBG sensor to the reinforcing bar was modelled by finite element method (FEM). The axisymmetric finite element model consisted of 150 solid elements depicting the actual geometry of FOBG sensor-epoxy-reinforcing bar system. Results of the evaluation showed that there was some loss in strain transfer from the reinforcing bar to the FOBG sensor (Fig.5), mainly due to the lower modulus of the epoxy in comparison to the moduli of steel and the FOBG sensor, which was glass. The loss, however, can be minimized by choosing epoxy compounds with high elastic modulus, and reducing the thickness of epoxy film between the sensor and the reinforcing bar.

In the case of the exposed FOBG sensor attached to the underside of the concrete beam (Fig.6), the sensor was wrapped and stretched between two steel cylinders (1 in. diameter) which were mounted on the underside of the concrete beam approximately 3 ft apart. The attachment concept was similar to the mounting design concept used in testing of fibers in direct tension using 1.5 in. mandrels. For both the embedded and exposed sensors, electric strain gages were used for verification of strain measurement correlation.

An optical spectrum analyzer (ANDO AQ-6310B) was used for measurement of strain-induced wavelength shift in the FOBG sensor. During the loading process, load was applied in increments and strain measured at each increment.

RESULTS

Tension Test on Bare FOBG Sensor

A number of direct tension tests were performed on FOBG sensors to evaluate the relationship between direct tensile strain and the fractional wavelength shift. In order to avoid grip fracture, the tension tests were performed by wrapping the fibers around two 1.5 in. mandrels which resulted in a firm non-slip fiber mount. Tension tests were within the elastic range of the fiber and were repeated for a number of cycles. Results of the tension tests, as presented in Fig.7, showed excellent correlation between strain and fractional wavelength shift. Moreover, repeated (cyclic) tension tests had no effect on the relationship between the strain and fractional wavelength shift.

Tension Test on FOBG Sensor attached to #3 Steel Bar

Fiber Optic Bragg Grating sensors were also tested in tension attached to a #3 reinforcing bar. The fiber sensor was placed in a small V-groove cut on the bar, bonded with epoxy, and then tested in cyclic tension. Result of this test, as presented in Fig.8, showed excellent correlation between the strain measured with electrical strain gages attached to the bar and the fractional wavelength shift of the sensor.

Sensors Embedded in Small Beam Models

In order to evaluate the performance of FOBG sensors for monitoring of deformation of concrete structural members, the sensor was attached to the reinforcing bar of a small prismatic concrete beams (1' 8" span) subjected to third-point loading tests to failure. The tensile strain induced on the reinforcing bar was measured with both the FOBG sensor and the electrical strain gages attached to the bar. Results of this test as presented in the form of strain vs. moment ratio, M/M_u , showed good agreement between strain measurements from FOBG sensor and strain gauges (Fig.9). A photograph showing the set-up of this test is presented in Fig.10.

Sensors Embedded in Large Beam Models

Results of the FOBG sensor measurement of strain in the large scale concrete beam (9'-6" span) is presented in Fig.11. As shown in the figure, excellent agreement was observed between measurements obtained from the FOBG sensor and those obtained from electrical strain gages. In this test, the FOBG sensor was attached to one of the reinforcing bars for direct

measurement of strain. A number of strain gauges (3 gages) were placed on the reinforcing bars for verification of FOBG sensor measurement. A photograph showing the test set-up is this test is presented in Fig.12.

Exposed Sensors Mounted on Large Beam Models

Results of strain measurement obtained from the exposed FOBG sensor mounted on the underside of the large concrete beam (9'-6" span) are presented in Fig.13. The FOBG sensor measurement was verified by a number of embedded strain gages attached to the reinforcing bars, and a set of LVDTs placed on the beam. Test results showed good agreement between all the sensors considering the different elevations at which the sensors were placed.

CONCLUSIONS

It can be concluded from past research and the findings from this study, that fiber optic sensor technology can be applied effectively to monitor strain in concrete structures. The main issue is the selection of an appropriate sensor type having the capability to measure the induced strain. The selection criteria includes sensor sensitivity, repeatability, and potential for ruggedization. Based on such criteria, the Fiber Optic Bragg Grating (FOBG) sensor has shown to be an effective tool for direct and quantitative measurement of strain in concrete structures. The conclusions reached in this study are as follows:

1. In reinforcing bar tensile tests, the fractional wavelength shift in the Fiber Optic Bragg Grating sensor, which is an indication for strain measurement, showed excellent correlation with tensile strain measured using strain gages.

2. Good bonding of sensor to structure is imperative for maximum transfer of strain. High elastic modulus epoxy, applied in thin layers between the sensor and the structure, provides the optimum bond.

3. The fractional wavelength shift in Fiber Optic Bragg Grating (FOBG) sensor showed good correlation with tensile strain in reinforcing bars embedded in a small concrete beam specimen (2.5"x2.5"x2'-4").

4. The strain measured with the Fiber Optic Bragg Grating (FOBG) sensor attached to the underside of a large concrete beams (10"x12"x10'-0") showed good correlation with strain gage measurements on the embedded reinforcing bar, and LVDT measurements outside the beam.

5. Future issues to be addressed include sensor ruggedization, refinement and simplification of the instrumentation, and large scale field implementation.

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NOTATION

λ_{Bragg} = Reflected wavelength of the light,
 n = average index of refraction in the fiber core,
 D = spacing of the periodic refractive index variation,
 $\Delta\lambda$ = shift in wavelength,
 K = constant,
 ϵ = tensile strain (in./in.).

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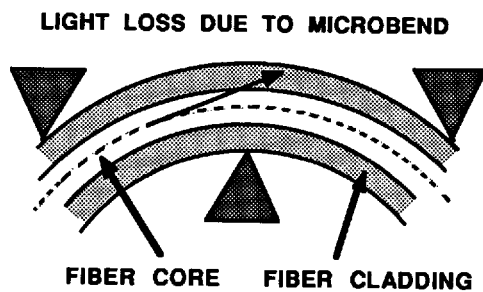


Fig. 1—A typical microbend configuration

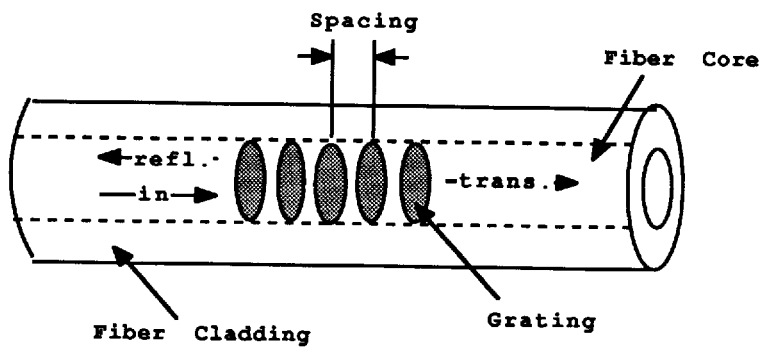


Fig. 2—Schematic diagram of gratings on a FOG sensor

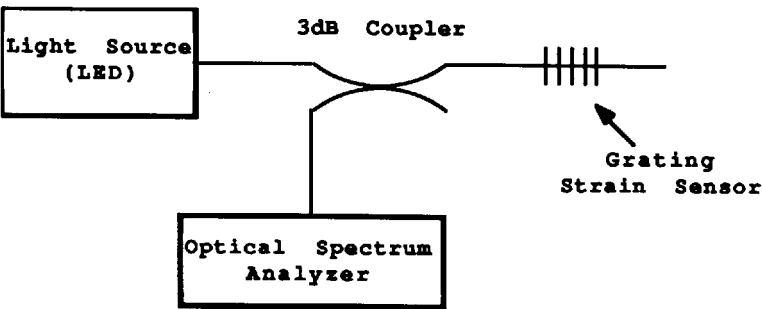


Fig. 3—Schematic of instrumentation for FBG sensor (Maher et al., 1992)

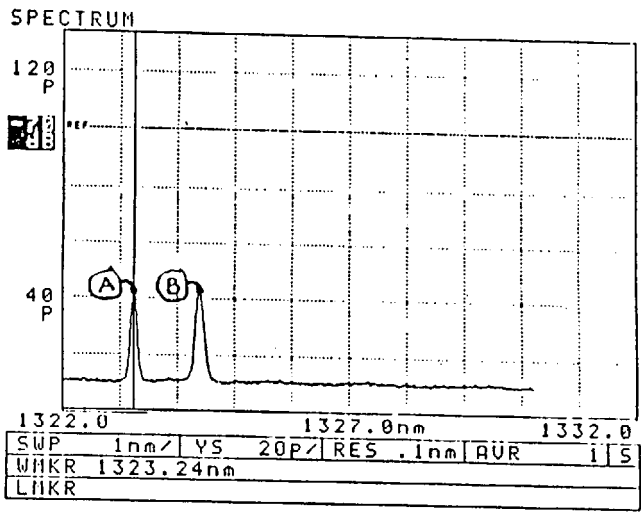


Fig. 4—Typical wavelength shift (from point A to point B) as a result of strain on FBG sensor

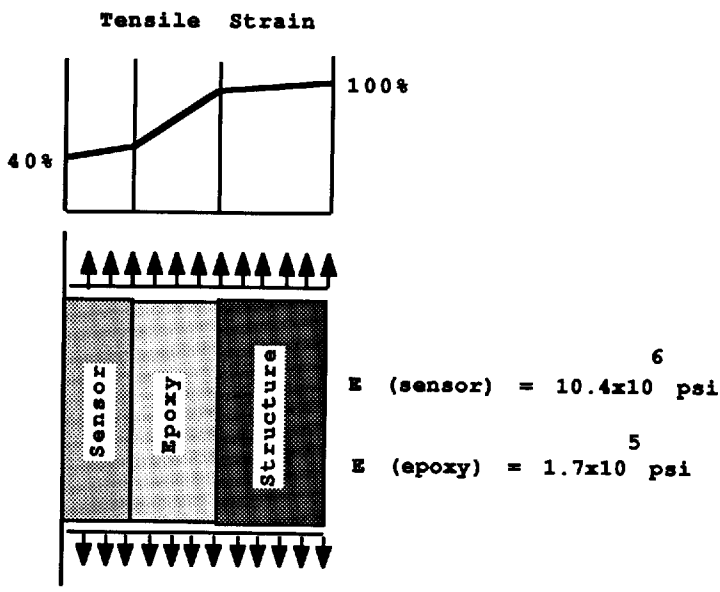


Fig. 5—Strain transfer between FOBG and the structure

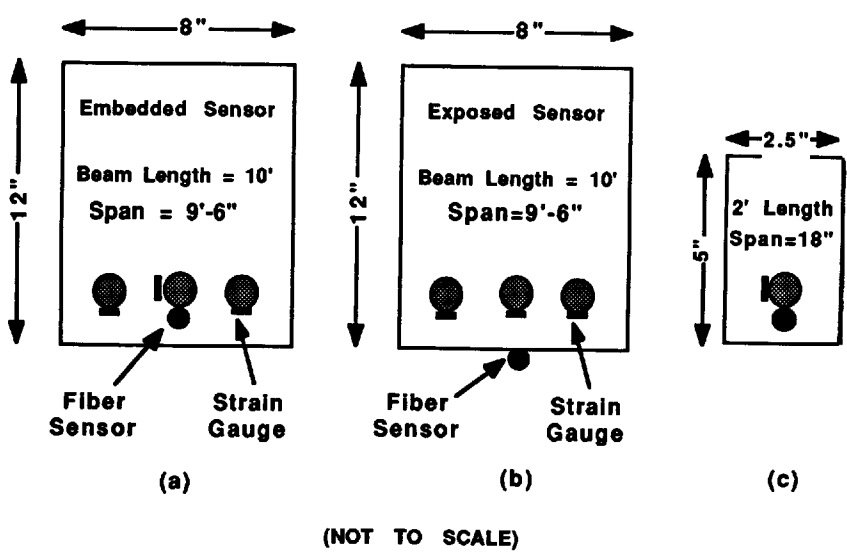


Fig. 6—Schematic diagram of the concrete beams cross sections

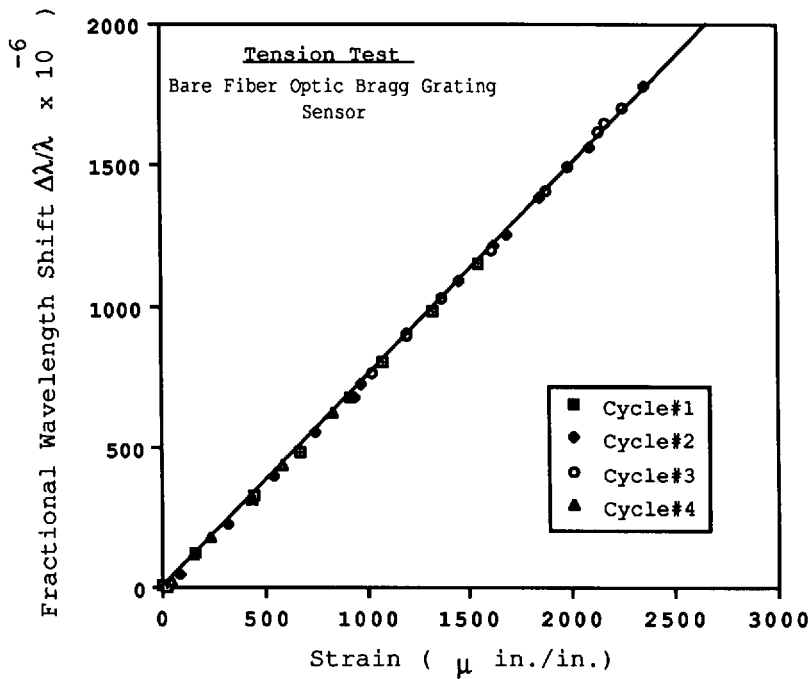


Fig. 7—Fractional wavelength shift versus tensile strain on a bare fiber

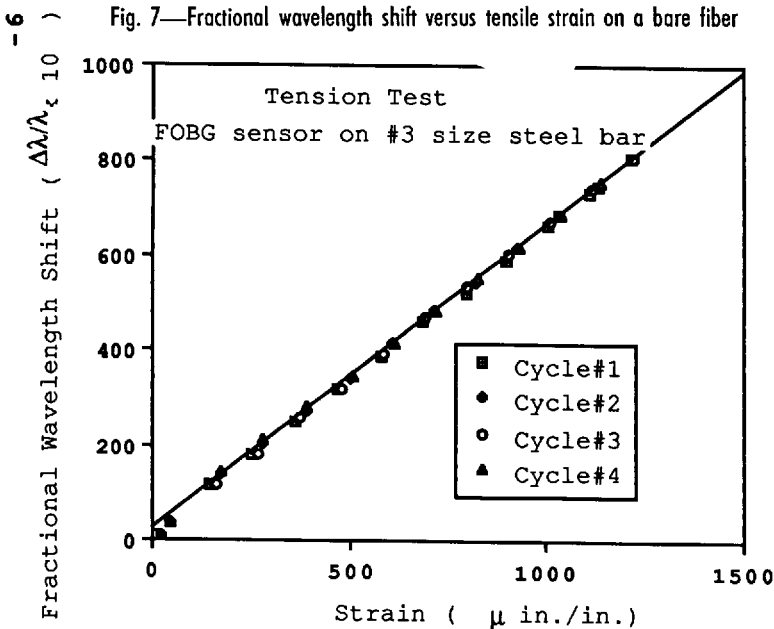


Fig. 8—Fractional wavelength shift versus tensile strain on FOBG sensor attached to a #3 size reinforcing bar

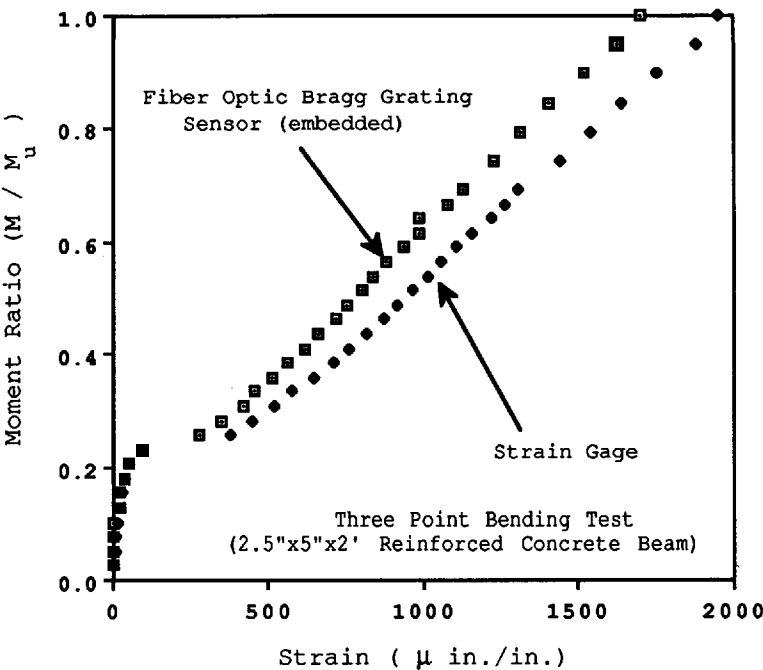


Fig. 9—Moment ratio versus tensile strain on FDBG sensor embedded in a small scale concrete beam

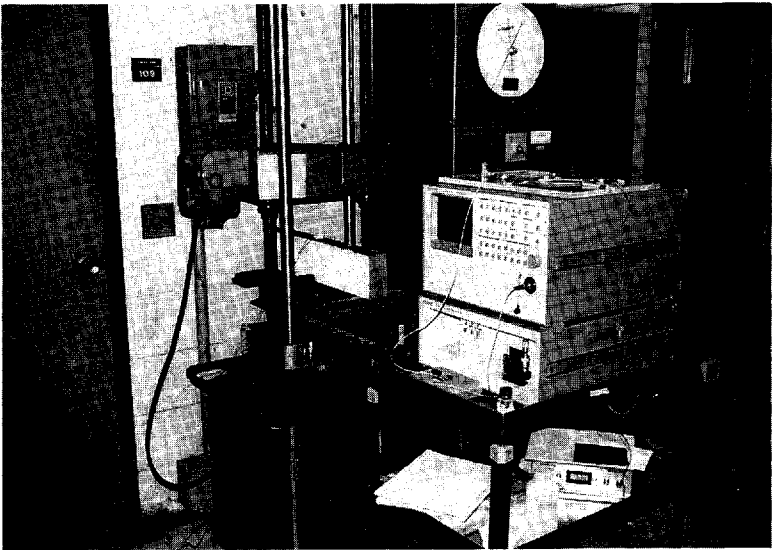


Fig. 10—Photograph of the third point flexural test set-up for the small scale concrete beam model

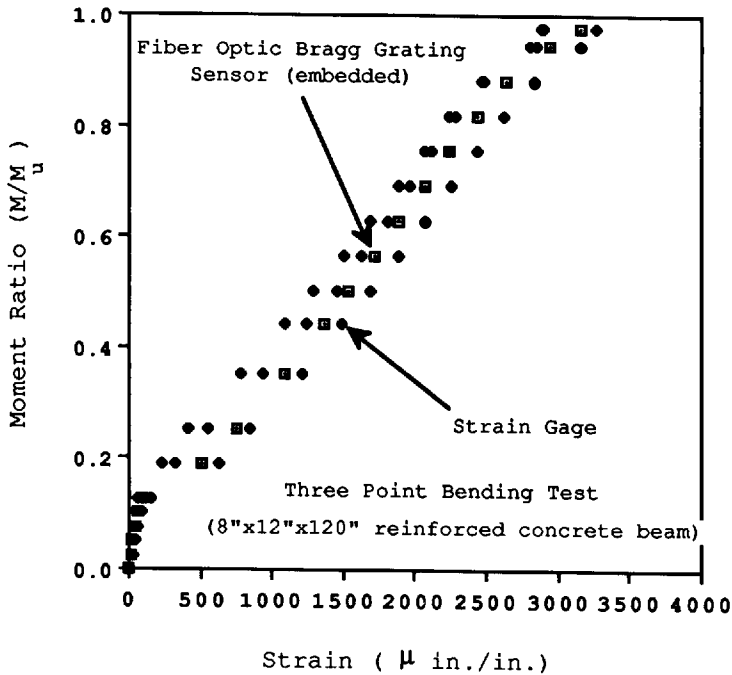


Fig. 11—Moment ratio versus tensile strain on a FOBG sensor embedded in a large scale concrete beam

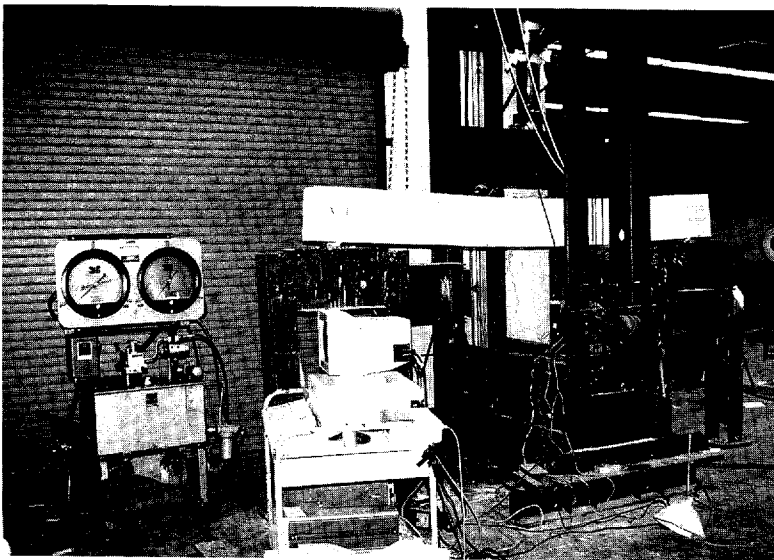


Fig. 12—Photograph of the third point flexural test set-up for the large scale concrete beam model

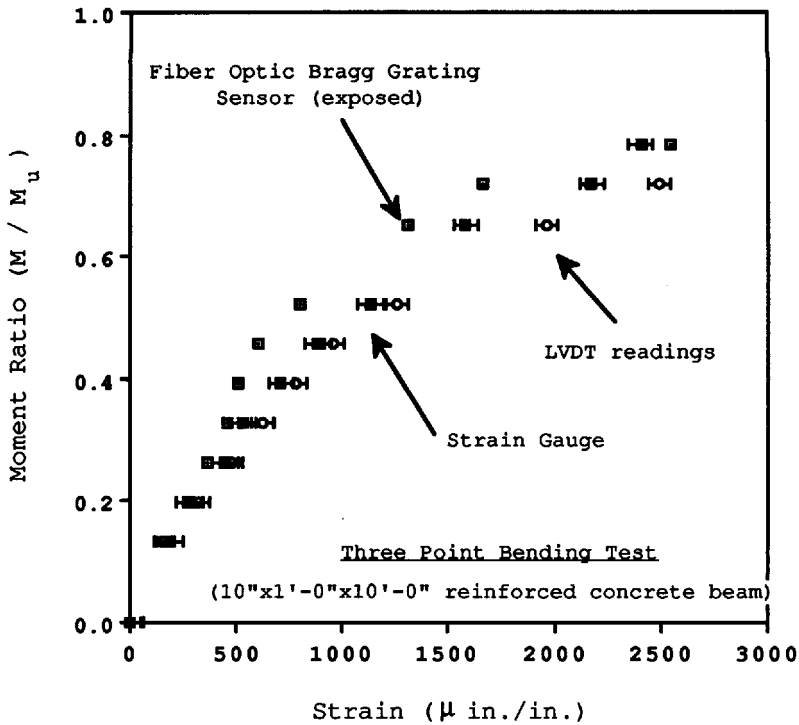


Fig. 13—Moment ratio versus tensile strength on FOBG sensor exposed on the underside of the large concrete beam

SP 143-2

Moving Load Tests on Three Precast Pretensioned Concrete Railway Bridges

by V. Sharma, W.L. Gamble, J. Choros, and A.J. Reinschmidt

Synopsis:

Load tests were conducted on three recently constructed precast pretensioned concrete railroad bridges. The three bridges tested were instrumented at their center line of span with concrete strain gages applied to appropriate locations to quantify the bridge behavior and to obtain impact factors. A dedicated test train was used to apply the loading for all the measurements. This train consisted of two locomotives followed by ten freight cars representative of most common types of cars used in modern freight trains. The test consist had a total gross weight of 1250 tons and a length of 1400 feet and included 5-platform type intermodal cars, 89 ft. flat cars (TOFC/COFC) and also hopper and box cars. The test train was operated at speeds of up to 70 mph. This paper describes the bridges tested, with special emphasis on field testing techniques and the personal computer based data acquisition system.

Keywords: Bridges (structures); girders; impact strength; load tests (structural); precast concrete; prestressed concrete; pretensioning; railroad bridges; slabs; tests

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INTRODUCTION

The Association of American Railroads is currently conducting a research program to evaluate the behavior and structural integrity of railway bridges under today's operating environment. Many of the existing steel railway bridges were built before 1950 and therefore range from 40 to 100 or more years old. These older bridges were designed for steam locomotive power and comparatively lighter freight car loads of 50 tons or less. Most of the prestressed concrete railway bridges are not older than 30 years, and were designed for diesel locomotives and 70 or 100 ton freight cars.

The modern freight environment consists of unit coal/grain trains with heavier load capacity freight cars: 100 ton capacity cars (wheel loads of 33 Kips) and double stack service unit trains utilizing 125 tons capacity trucks (wheel loads of 39 Kips). All the trains are powered by diesel locomotives.

The overall objectives of the bridge research program at the AAR are to:

1. Develop loading spectra of bridges in today's freight car environment, by field measurement of loads on the bridges due to various types of trains (unit, intermodal, mixed freight, etc).
2. Develop clear understanding of the bridge behavior (behavior of primary and secondary structural members) for various types of bridges (concrete, prestressed concrete, steel plate girder, steel truss) under today's freight environment, by detailed studies to measure bridge response (strains, deflections, etc).
3. Conduct detailed studies as in (2) above, under controlled load environments (using specific train consists at various speeds) to obtain impact loading spectra for various bridge types.
4. Develop an understanding of the fatigue load environment of steel bridges under today's freight trains. This includes measurements of cyclic loading experienced by primary load carrying structural members during passage of different types of freight trains.

The AAR's bridge research program was initiated in 1988 and is being conducted jointly with researchers from the University of Illinois at Urbana-Champaign. Funding to the University of Illinois is provided by the AAR and the National Science Foundation. Tests on the following bridges have been completed:

1. Milledgeville, Illinois : Open deck plate-girder with mixed traffic [1],
2. Chana, Illinois : Ballasted plate-girder with mixed traffic [2],
3. Blacksburg, Virginia : Open deck girder with unit coal traffic [3],
4. Del Rio, Tennessee : Open-deck through-truss bridge [4],
5. Madisonville, Tennessee : Open-deck through plate girder bridge, and
6. Lorenzo, Illinois : Three precast pretensioned concrete bridges with spans ranging from 18 to 60 feet [5].

This paper deals with the instrumentation, data acquisition and testing of these concrete bridges.

General Project Description

Load tests were conducted on three recently constructed precast, pretensioned concrete railroad bridges as part of an on-going cooperative research program. A test train was operated across the bridges at various speeds ranging from 5 to 70 mph (8 to 113 km/hr). Access to the bridges was provided by the Atchison, Topeka, and Santa Fe Railway (AT & SF), and they also provided the test train through an agreement with AAR.

The main purpose of the test was to provide information on the dynamic, or "impact," effects which the bridges might experience. Present design provisions in the AREA Manual of Railway Engineering, Chapter 8, include clauses relating to "impact" effects. The provisions relating to prestressed concrete bridges are largely based on load tests done in the 1960's, and there were several concerns leading to the current tests:

1. The existing data base was quite small, as only a few bridges were tested.
2. The data base includes bridges having only a limited range of spans.
3. The strain measurements from the earlier tests were made with equipment which is much inferior to currently available equipment, and this becomes especially important when it is realized that the strains measured were generally quite small and therefore high resolution is needed.
4. Train loadings are somewhat heavier, on the average, than in the 1960's, and the basic nature of the trains has changed somewhat with the introduction of various intermodal cars including 5-platform container and trailer cars.

The earlier tests were reported in a series of six Engineering Reports ("ER") from AAR, and they are listed in the references. The AT & SF Bridges described in ER-18 are more extensively described in a Portland Cement



Association (1960A) report [12]. Table 1 lists the reports along with the spans and a limited amount of other information about the structures.

Secondary objectives were simply to gain an understanding of the response of multi-beam bridges to the train loading, whether static or dynamic. The designer perhaps typically assumes that the live load is evenly divided between the four box girders used in some bridges, for example. Strain measurements give a basis for judging the validity of this assumption.

Other Related Research

Three papers or groups of papers related to dynamic stresses in prestressed concrete railroad bridges can be cited. Muller and Dux (1988, 1989, 1992) report the effects of major rail defects on measured stresses and strains in bridges with 15 and 25 m (49 and 82 ft) spans. They milled flat-bottomed depressions into the head of one rail in three patterns:

1. 1 depression, 3 mm deep x 250 mm long,
2. 1 depression, 6 mm deep x 300 mm long, and
3. 2 depressions, 6 mm deep x 300 mm long, spaced at 1680 mm, which was the axle spacing on the coal train cars in use in Queensland, Australia.

Major impact effects were recorded for all three conditions, with the two-defect case being worse. The wheel loadings were relatively light by current US standards.

Venuti and Heusch (1979) provide a summary of earlier work and make some recommendations about design. They calculated theoretical dynamic load factors and found that they agreed reasonably well with measured values for prestressed bridges. The AREA impact factors for reinforced concrete bridges were judged to be very conservative, and to have little correlation with theoretical values. Their study did not include effects of flat wheels, and they assumed high quality jointless track. The effects of track modulus were included.

A group of inter-related papers by Chu, et al (1986), Garg, et al (1985), Wang (1990), and Wang, et al (1991) present theoretical studies of railroad bridge dynamics in which the interactions between the bridge and the rolling stock are accounted for. Dynamic modeling of the cars is given considerable emphasis.

A 1960 issue of "Concrete for Railways" (Portland Cement Association (1960B)) contains a review of then-current construction which now has historical value. Projects utilizing solid pretensioned slabs, various box sections, and various I-girder sections are described. Most projects used precast, pretensioned members while a few used precast, post-tensioned members in which the members were precast at or near the job site but not in final positions on the piers.

DESCRIPTION OF THE TEST BRIDGES

The three bridges are located on the AT & SF East-bound mainline track south-west of Joliet, IL, between 45 and 52 miles SW of Chicago. (Although the track is referred to as the East-bound track, its orientation is more nearly SSW-NNE in the region containing the bridges. As a consequence of this, the rail force measurements refer to the East and West rails.) Bridges X45.4 and X49.4 were constructed in 1987, the third bridge X52.0 was constructed in 1989 using precast, pretensioned components which were placed on existing substructures. All three are ballast-deck structures, with wood ties and continuous welded rail. The concrete deck at Bridge X52.0 was coated with an asphaltic material known as "Semi-Mastic" which was hot-mopped on. This was topped by two layers of 3/8 in. thick asphaltic panels, in the form of 4 x 8 ft sheets, which covered the center 8 ft of the bridge width. Ballast and trackwork were then installed.

The bridges were designed for 115% of the Cooper E-80 loading, with diesel impact factors. The impact equation was (for spans under 60 ft):

$$I = 35 - l^2/500 \text{ Percent} \dots\dots\dots (1)$$

where l was in feet, center to center of bearings.

The extra 15% was to compensate for centrifugal force effects and the possibility of having the track center six inches off of the bridge center. All three bridges are on tangent track.

Revenue service includes mixed freight, unit freight, container cars, piggy-back with various platforms, and AMTRAK. Traffic amounts to about 19 million gross tons per year. Gradients and curvatures in the region are small, and trains operate at high speeds.

The largest bridge, at Mile Post 52.0 (Figure 1), has ten spans of 60 ft (18.3 m) nominal length each. Each span was made from four precast



pretensioned single-cell box girders which were placed side-by-side and connected with shear keys and transverse bolts. The X in the designation means that it is in the South (Eastbound) track of a widely-spaced double-track line.

The smallest bridge, at Mile Post 49.4, has a single 18 ft (5.5 m) span. The span was made from two solid precast, pretensioned concrete slabs.

The intermediate sized bridge, at Mile Post 45.4, has two spans of 28 ft 5 in. (8.66 m) each. Each span was made from two two-cell precast, pretensioned concrete box girders.

The specified concrete strengths were $f'_{ci} = 4500 \text{ lb/in.}^2$ at transfer and $f'_c = 5500 \text{ lb/in.}^2$ at 28 days. The strength of the concrete at the time of the tests is not known, but it is expected to be in excess of 7500 lb/in.^2 . All three bridges used the same concrete quality. The effective span of the three bridges are 58'4", 17'3" and 27'4" respectively.

Grade 270 Low-Relaxation pretensioned strands, 0.5 in. diam, were used in all bridges. All strands are straight, and they were pretensioned to $0.789 f'_s = 213 \text{ k/in.}^2 = 32.6 \text{ kips/strand}$.

DESCRIPTION OF INSTRUMENTATION AND DATA COLLECTION SYSTEM

General

Two types of data were collected at each bridge. Vertical forces were measured in each rail at a section within a few inches of midspan. Longitudinal concrete strains were measured at 12 to 14 points distributed around the cross section where the rail force gages were measured. The measuring sections were offset slightly from midspan in bridges X45.4 and X52.0 because of joints in the ballast retainers.

The rail forces were measured by means of strain gages attached to the webs of the rails. The gages measured shear strain in the rail webs, and strain was calibrated against a known load applied by hydraulic jacks at each location. There were four two-element chevron-type gages at each force measurement location, with sets on both sides of the webs separated longitudinally by 10.0 in. The strain gages were connected such that one strain reading was produced

from the entire group of gages, and the reading represented an average of the four gages, or eight measuring elements.

The exact locations of the rail force measurements were influenced by the tie locations, because the measurements had to be made between ties, with at least 2 in. clear length between the gages and the adjacent tie plate. Spikes were pulled and tie plates moved small distances in order to accommodate this requirement. Figure 2 shows the rail gages in place.

Output from the strain gage circuits for each bridge was fed to a digitizing and switching unit and then to a microcomputer, where it was stored awaiting processing. The strain sampling rate was 512 samples/second on all channels.

Two separate records were made of each strain trace. In one record, the strain was recorded directly. In the second, the electrical signal was passed through a filter before being recorded. The intent of the filter was to remove some of the high-frequency electrical "noise" which seems to be inherent in dynamic strain recording systems. Three complete systems were required because of the separation between the three bridges.

Locations of Strain Gages

Figures 3, 4 and 5 show a schematic of the three bridges where the strains were measured. At each bridge CH01 and CH02 were for rail force measurements. Three locations each on the east and west faces of the bridge were chosen to measure response of the faces and also get an idea of the location of the neutral axis. Each slab or box girder response was measured by measuring strains at the bottom face in at least 2 locations.

Selection Of Strain Gages

A laboratory study was conducted to quantify the response of various types of strain gages to select the gage which would suit the needs of our testing. One-quarter inch weldable gages were compared to two-inch strain gages with either acetate or mylar backing. These gages were applied to a concrete tie and a known incremental load was applied to the tie, and the response of the three gages was obtained for various loads. It was found that all of the three gages demonstrated linear responses and that the strains

measured by the quarter inch gage were only about one-third of the theoretically expected strain. Therefore, these quarter inch gages were not used in further laboratory testing. The choice then was between acetate backing or mylar backing 2" gages. The output from the acetate gage was generally found to be lower than the mylar backed gages and after several test repetitions, it was found that the mylar backed gages provided the most reliable performance, and the measured strains were consistently higher and closer to the expected values, and the response was linear for increasing loads. After several discussions with the gage manufacturers a procedure for field application of these gages was picked upon. This procedure was repeated in the laboratory a few times to determine the consistency of the technique and to provide some level of experience to the technicians who would apply these gages in the field. The following procedure was strictly adhered to during the field application of these gages.

Procedure for Strain Gage Installation

1. The strain gage location should be such that there are no large size (0.25" or more) voids in the concrete in the vicinity. This turned out to be an easily met requirement because the faces of the structures were quite smooth.
2. The locations where the strain gages are to be installed are carefully marked, and a small area, say 4" to 6" diameter is ground with a fine grinder to obtain a smooth surface.
3. This surface is washed with acetone (or Conditioner A) to remove any loose powdery material and then rinsed with distilled water.
4. This area is then dried by using heat lamps (800 Watts). Hooks to hold the heat lamps in place are cemented a foot or so away (dental cement works just fine). Generally an hour or so of heating is required to completely dry the surface.
5. Then apply a layer of AE10 epoxy to the cleaned and dried area. If there are any fine voids they should be completely filled, this layer of AE10 epoxy is then cured by using the heating lamps.
6. The cured surface is made perfectly smooth by using a very fine sandpaper. The surface is now ready to receive the mylar backed strain gages.
7. Apply the gage with M-Bond-200 epoxy. Care should be exercised in pressing the gage in place. Based on past experience, best results are obtained if the gage is pressed from the middle outward with uniform pressure with a finger.

8. Let the area dry and cure for about 30 minutes. Then the integrity of the gage application is checked using a strain box. The gage is pressed by a finger at about 4 or 5 different places along the length of the gage. The strain output should be negligible. We had arbitrarily chosen a value of 4 micro inch/inch. If there are any air pockets or incomplete bonding the gage output would be 10 micro inch/inch or more. Of all the gages installed in this test program only three gages had to be removed and reinstalled.

9. After satisfactory completion of Step 8, the gage is covered with an approved polyurethane-waterproofing material.

All the strain gages (except those for rail force measurements) were wired as single-acting gages (full bridge) with appropriate completion resistors. The completion resistors are housed in an environmentally sealed box. Cables of appropriate length connect this box to the data acquisition box described below.

DATA ACQUISITION

DOS-based data acquisition packages were chosen to record data from each of the three concrete bridges. Three separate systems were required because of the distances between the bridges and the fact that all the testing was to be accomplished in one day. The data acquisition system consists of three main components

1. Signal Conditioning

The signal conditioning for all the strain gage bridges was provided by the Acurex MDAS-II (Modular Data Acquisition System). This unit provides the excitation voltage and balancing of the strain gage bridge. The unit can hold 16 cards with two channels on each card, giving a total of 32 channels. The system is programmable and flexible. Each channel is configured independently as follows:

- i. The type of bridge circuit (full, half or quarter.) In our case the rail force measurements were full bridges and all concrete strain measurements were quarter bridges.
- ii. Excitation voltage (4 or 8 V.) We had selected 4V as the excitation because 120 Ω gages were used.
- iii. The amplifier gain (from 1 to 1000.) We used a gain of 1000, since we were dealing with small strains.
- iv. The calibration resistor (we selected a precision resistor of 174.65 K Ω with a 0.02% tolerance), which can be internal or external. The calibration resistor simulates a known value of

strain. If any signal gain errors are present, the shunt calibration will show this so that a post-processing correction can be made.

2. The Filters

When collecting and digitizing analog data in a railroad environment our experience has shown that it is good practice to use appropriate filters. The filtered data would always have less noise and inexplicable spikes in the data and provide guard against aliasing. We chose 6-pole Bessel low-pass filters with a cut-off frequency of 30 Hz. The cut-off frequency for filtering should be selected taking into consideration the frequency response of the structure. For concrete bridges, all the vibration modes of interest are below 20 Hz frequency. The characteristics of the type of filter chosen, e.g. roll off properties, are also important. For the filters chosen, a 30 Hz cut-off guaranteed full capture of frequencies below 20 Hz. Sampling rate selection is based on data acquisition storage limitations as well as the need to capture the peaks of the structural response. For a 20 Hz phenomenon, the chosen sampling rate of 512 will give 25 samples in each cycle. Maximum error in peak value obtained with this sample rate is obtained by $1 - \sin(90 - 90/25) = 0.2\%$, if the response waveform is assumed to be sinusoidal.

3. The 386-Computer and the A-D Card

The filtered analog signals are sent to the analog to digital (A/D) board in the computer. This A to D card (Analog devices RTI-800 A/D is a 12-bit card affording a resolution of 1 in 4096 counts with a maximum voltage range of $\pm 10.24\text{V}$) is compatible with the software selected for interfacing it with the computer. The programs 'SNAPSHOT' and 'SNAPSTREAM' (developed by HEM Data Corporation) were selected for their ability to sample large amounts of data in a continuous data stream. The data is streamed directly (in binary form) to the CPU memory of the computer. Each of the systems had a memory of 6 Megabytes, which is sufficient to record very long trains. The longest data file collected during the concrete bridge tests was only 1.4 Megabytes. After completion of a test run the CPU memory is downloaded to the hard disk (100 Megabytes) of the computer. An accompanying software called 'STM2PLOT' can convert the data to ASCII format, if needed. It has been our practice to examine the data by plotting each and every channel after the first test run to ensure that data of acceptable quality is being gathered on all the channels. This is very important, especially for 'one-shot' type tests where there is only one opportunity to get it right.

A schematic of various components is shown in Figure 6. All the components of the data acquisition package are housed in a NEMA-12 enclosure (Figure 7). This enclosure has been modified to receive air circulation and air conditioning (heating or cooling) from a portable AC/Heat unit which can sit on or near this enclosure. The thermostats in the enclosure control the operation and provide safety shut-off of the data acquisition system in case of malfunction of the environmental control units. All the 110V electrical power needed for operation of the system was voltage/surge regulated. During the concrete bridge tests, the power was supplied by portable gasoline generators.

REPRESENTATIVE STRAIN MEASUREMENTS

An example of measured wheel loads (or rail forces) on east and west rail for bridge X45.4 during a 30 mph crossing of the bridge is shown in Figure 8. Also shown in the figure are, from left to right, the various cars of the test consist and their respective positions in the consist. The sum of the wheel loads on the east and west rail gives axle loads which are shown on the top part of Figure 8. The first locomotive has two trucks each having three axles, followed by the second locomotive which also has two three-axle trucks. The first freight car is a 5-platform spine car which has five platforms and six two-axle trucks. The intermediate platforms of such a freight car share one truck with the adjacent platform.

The measured strain response of the east double box girder of bridge X45.4 is shown in Figure 9. Channel 7 is near the east edge of the box, channel 8 is in the middle and channel 9 is near the west edge of the east box girder. The response is quite similar for the three locations with some minor variations. Also it can be seen in Figure 9 that there is a distinct peak response caused by the passage of each of the locomotive trucks. These first four peaks are approximately of the same magnitude because each truck of the two locomotives weigh about the same, though actually the first locomotive weighs approximately 2% more than the second locomotive. Also, it is possible to identify peak response from each truck of the freight cars in the test consist. The last car of the consist was a loaded box car preceded by a loaded hopper car. The last three peaks in the response are due to the passage of these two cars. The middle peak of these last three peaks is due to the trailing truck of the hopper car and the leading truck of the box car. The existence of discrete identifiable peak response depends on the truck spacings and their relationship to the bridge span.

The strain response of one of the box girder bottom face gage of the bridge X52.0 is shown in Figure 10 for 30 mph speed. Since this bridge span is longer (60 ft. compared to 28'5" for bridge X45.4), the response is of a



slightly different nature. Only three peaks in the beginning are discrete, being caused respectively by

- i. The leading truck of the leading locomotive
- ii. The trailing truck of the leading locomotive and the leading truck of the trailing locomotive
- iii. The trailing truck of the trailing locomotive and the leading truck of the first car. Therefore, in this response the second of the first three peaks is always substantially larger than the other two peaks. Also, the passage of the trailing end of the consist (namely, the loaded hopper car and the loaded box car) yields only one major peak (due to the adjacent trucks of the two cars). This major peak is preceded and followed by minor peaks due to the passage of the leading truck of the hopper car and the trailing truck of the box car.

Theoretically calculated values of the strains at the bottom face of the beam were obtained by using:

1. Axle loads and axle spacing information of the test train.
2. Simple structural analysis concepts such as influence lines, stress-strain relationships and an assumed value of the modulus of elasticity of 4420 ksi.

Figure 11 shows the comparison of the calculated strains versus measured strains of bridge X52.0 for the 70 mph test run. Notice that the waveform of the calculated response is almost identical to that of the measured response (barring amplitude). An increased value of the modulus of elasticity of concrete would yield a closer match of the peaks.

CONCLUSIONS AND RECOMMENDATIONS

● Maximum live load tensile strains were found to be in the 100-125 $\mu\epsilon$ range. These strains correspond to 500-700 psi stress. The average precompression in the concrete being 1000 psi, these live load strains have adequate margins of safety.

● It was found that for continuous welded rail (CWR) on track class 5 or better, impact factors were quite low. Maximum values recorded were in the 10-15% range, provided that there were no wheel defects. However the presence of non-condemnable out-of-round or flat wheels can significantly

increase the impact factor. These increased values can be up to 2-3 times the values obtained for average quality wheels.

- The procedure used for concrete gage application in these field tests was found to be very successful and the performance of the gages was consistent and reliable.
- The data acquisition system is capable of collecting data at high sampling rates for up to 32 channels simultaneously. The data sampling rate of 512 samples/second was found to be very satisfactory for capturing the shape and peaks of the strain waveform during the passage of the test train at speeds of up to 70 mph. It is recommended that the data sampling rate be selected with proper regard to the maximum frequency of interest.
- The use of appropriate analog filters while collecting and digitizing analog data in railroad environment is recommended. Filtered data has less noise and no inexplicable spikes in it. Also, filtering eliminates aliasing.
- It is recommended that the data acquisition system should have "quick-look" capabilities. The data should be examined by plotting each and every channel after the first test run (and a few more times, if possible) to ensure that data of acceptable quality is being gathered in all the channels. It helps to do some theoretical calculations of expected values before doing the field testing. This is very important, especially for "one-shot" type tests, where there is only one opportunity to get it right.

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TABLE 1 — LISTING OF REPORTS AND DETAILS OF THE ANALYZED STRUCTURES

Report	Spans, ft.	Cross-section	Notes
ER-18	71.5	4-71" deep, PT Modified I-Sections	Single Track, 4 beams wide, 2 simple spans
ER-21	30.5,30.5	30" deep x 26.75", 14" diam. void	Double Track, 12 beams wide
ER-48	24.0	2-cell box, 75" wide, 28.5" deep	Single Track, 2 boxes
ER-61	24.92, 37.58	28" and 36" deep, 28" wide, 16" diam. void	Single Track, 5 boxes
ER-64	34.18, 42.35	42" deep x 36" 27" x 26" diam void	Single Track, 6 boxes
ER-67	33.0	27" deep x 42" 16" X 32" void	Single Track, 4 boxes

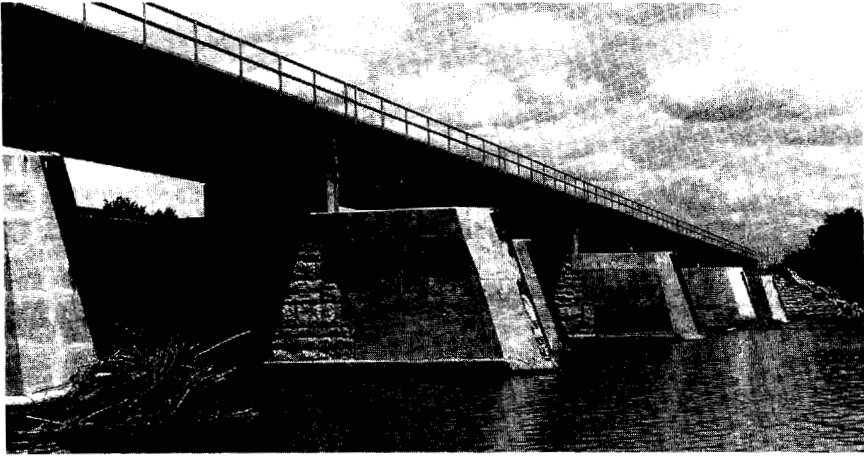


Fig. 1—Photograph of bridge X52.0

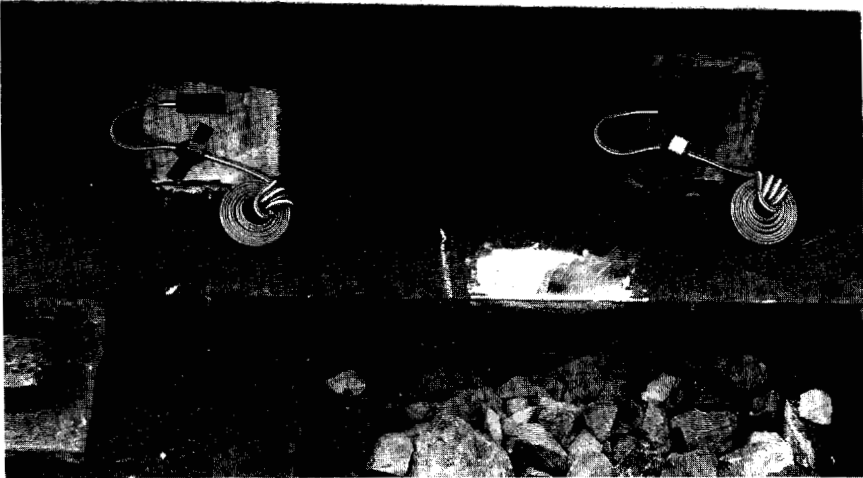
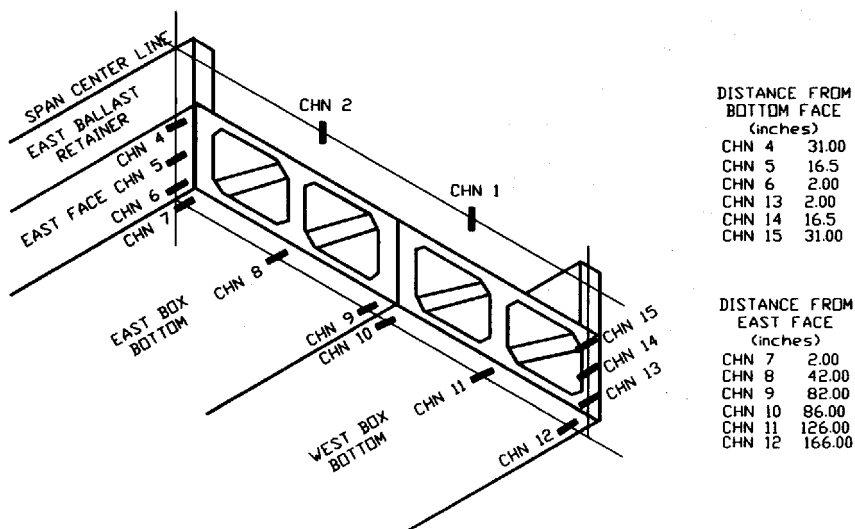
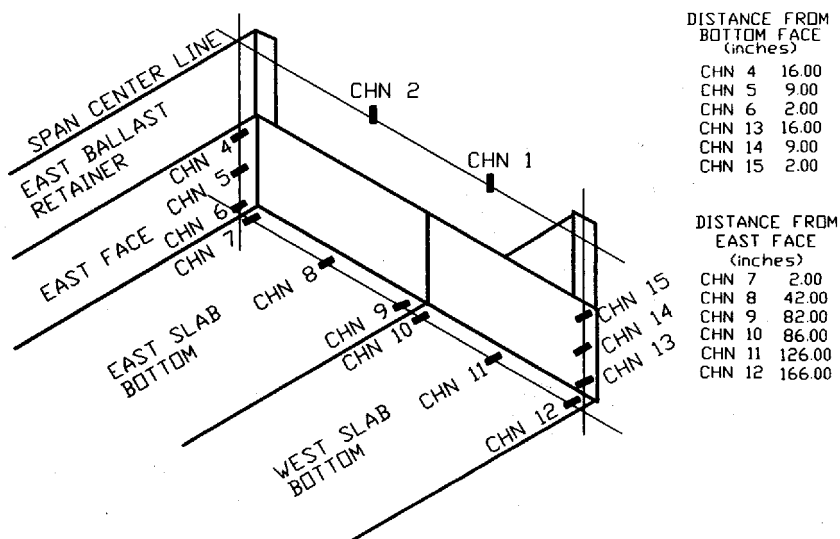


Fig. 2—Photograph of strain gages for rail force measurements



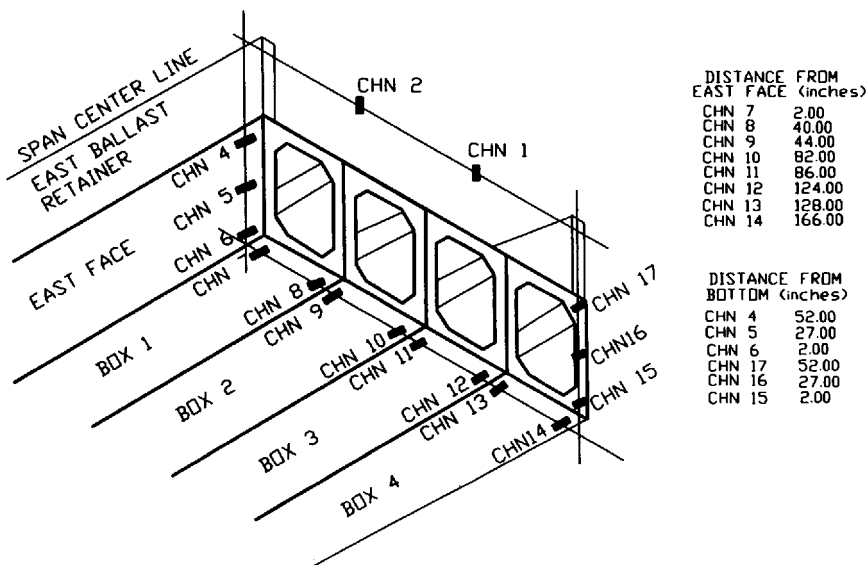
GAGE LOCATIONS FOR BRIDGE X45.4

Fig. 3—Strain gage locations for bridge X45.4



GAGE LOCATIONS FOR BRIDGE X49.4

Fig. 4—Strain gage locations for bridge X49.4



GAGE LOCATION FOR BRIDGE No. X52.0

Fig. 5—Strain gage locations for bridge X52.0

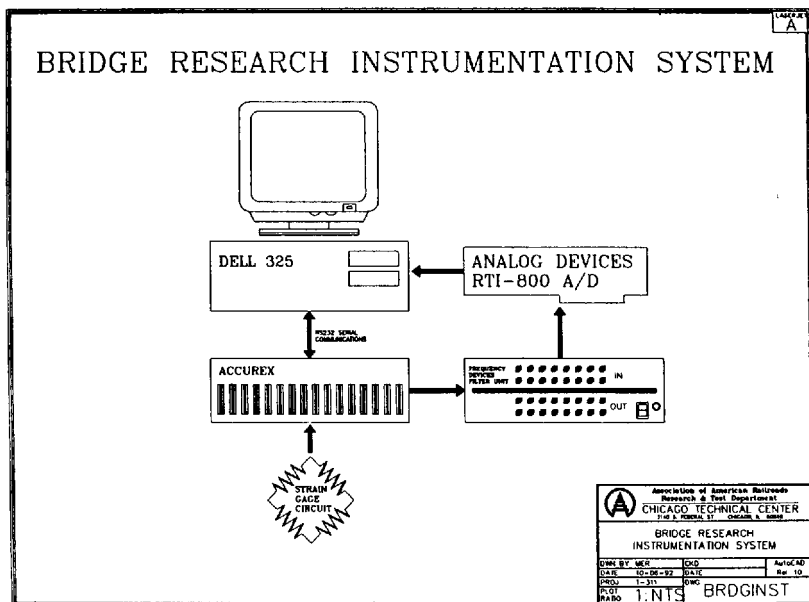


Fig. 6—Schematic of the various components of the data acquisition system