FINAL REPORT

Project Title: "Evaluation of Composite Post-Tensioning Systems on Bridge COS 79-0955, Coshocton County"

Authors: Dr. William E. Wolfe
Dr. Henry R. Busby
Dr. Tarunjit S. Butalia
Research Agency: The Ohio State University Research Foundation

Date: March, 2003

Sponsoring Agencies: Ohio Department of Transportation
Federal Highway Administration

"Prepared in cooperation with the Ohio Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration"

1. Report No.

FHWA/OH-2003/004

2. Government Accession No.

3. Recipient's Catalog No.

4. Title and subtitle

Evaluation of Composite Post-Tensioning System on Bridge COS-79-0955, Coshocton County

5. Report Date

March 2003

6. Performing Organization Code

8. Performing Organization Report No.

7. Author(s)

Dr. William E. Wolfe
Dr. Henry R. Busby
Dr. Tarunjit S. Butalia

10. Work Unit No. (TRAIS)

9. Performing Organization Name and Address

Ohio State University
Civil and Environmental Engineering & Geodetic Science
College of Engineering
2070 Neil Avenue
Columbus, OH 43212-1275

11. Contract or Grant No.

State Job No. 14722(0)

13. Type of Report and Period Covered

Final Report

12. Sponsoring Agency Name and Address

Ohio Department of Transportation
1980 W Broad Street
Columbus, OH 43223

14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract

Aging highway infrastructure, increasing traffic loads and the high cost of rehabilitation have combined to make novel repair methodologies increasingly attractive. Several new products now receiving attention in bridge rehabilitation are made from high strength, durable composite materials. In the current project, fiberglass rods were affixed to a 14-meter long single span concrete T-beam bridge. The project was divided into three phases. The first phase consisted of the installation of strain and displacement sensors on the existing structure and the measurement of deflections during controlled traffic loads. Composite rods were attached in the second phase. During the third phase the response of the newly reinforced bridge to vehicular and environmental loads was monitored. No actual improvement in bridge performance was detected after the fiberglass rods were installed. The slight difference between the before and after deflections and strains was less than the variability that should be expected in the readings.

17. Key Words

Concrete bridge, composite, retrofit

18. Distribution Statement

No Restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Page	22. Price
Unclassified	Unclassified		

Form DOT F 1700.7 (8-72) Reproduction of competed page authorized

ACKNOWLEDGEMENTS

This report was prepared as part of a research project titled “Evaluation of Composite Post-Tensioning System on Bridge COS-79-0955, Coshocton County,” and was performed at The Ohio State University. The principal sponsor of the research was the Ohio Department of Transportation. Substantial additional support was provided by The Ohio State University, College of Engineering. The cooperation of the staff at ODOT and the Federal Highway Administration is appreciated. The efforts of a number of students and staff of the Department of Civil and Environmental Engineering and Geodetic Science were required to bring this monitoring program to a successful conclusion. In particular, the authors would like to thank the following individuals: H. Jason Hughes, Ross Baldwin, Stephen Taliaferro, Richard D. Fortner, David R. Davis, A. Ray Hunter, Todd G. Guttman and Floyd Rister.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

INTRODUCTION

According to the Federal Highway Administration's 1999 annual report titled "Status of the Nation's Highways, Bridges, and Transit" an estimated 29% of the nation's bridges need to be either rehabilitated or replaced. Aging highway infrastructure, increasing traffic loads and the high cost of rehabilitation have combined to make novel repair methodologies increasingly attractive to transportation officials. Several new products now receiving attention in bridge rehabilitation are comprised of composite components, which, because of their high strength and stiffness to weight ratios and durability, are logical substitutes for conventional highway materials. The rehabilitation method being studied in the current project consists of attaching composite rods to an existing concrete bridge. After the rods are affixed to the bridge, each rod is loaded to a design tension. The design tension is intended to improve the capacity of the bridge by changing the mean stress in its principal structural members. There are several advantages to this rehabilitation technique; the stiffening members can be applied with little or no traffic interruption, scaffolding and site preparation may be minimized, and tendons and anchors can be prefabricated to reduce field work.

The Ohio Department of Transportation (ODOT) contracted with The Ohio State University (OSU) to observe the installation of external post-tensioned fiber reinforced polymer rods on bridge COS-79-0955, perform sufficient testing and analysis to evaluate the effectiveness of the reinforcement. Fiber Reinforced Systems, Ltd. of Columbus, Ohio (FRS) was selected by ODOT to design and install the reinforcing rods. Typical instrumentation installed by OSU consisted of strain gauges placed in locations where high stresses were anticipated, displacement devices at the center of the span, and load cells on selected rods.

The study bridge, ODOT designation COS-79-0955, is a 14 meter long single span reinforced concrete T-beam bridge with six longitudinal beams. It is located on State Route 79 in Coshocton County. A map of the area is given in

Appendix B. The purpose of the post-tensioning was to increase the load capacity by reducing the maximum positive moment over the piers.

The project was divided into three phases. The first phase consisted of the installation of strain sensors on the existing structure and the measurement of deflections prior to the installation of the structural composite rods. In the second phase FRS attached to the bridge the composite rods, brackets and, on select rods, load cells supplied by OSU. The design of the rods, their attachments and locations was performed by FRS in direct consultation with ODOT bridge engineers before OSU was approached to monitor the performance of the bridge. During the third phase of the project the response of the reinforced bridge to vehicular and environmental loads was monitored. The loads in the instrumented rods were monitored for the remainder of the project duration. Although the proposal considered only a short term project, some extended monitoring of the installed instrumentation was carried out to adequately evaluate the performance of the composite post-tensioning system.

RESEARCH OBJECTIVES

This study was designed to focus on measuring the effects of the attachment of carbon fiber tension rods to an existing reinforced concrete bridge subjected to various types of static and dynamic traffic loads and environmental conditions. In the proposal, the following research objectives were identified:

1. Evaluate the material and structural properties of the carbon fiber reinforcing rods in laboratory tests by performing tests on representative small-scale specimens;
2. Instrument the bridge with monitoring devices to evaluate structural response to traffic loads and environmental conditions;
3. Begin taking measurements that should lead to an evaluation of the effects of environmental factors on the performance of the reinforcing rods;

4. Use the data collected to assist ODOT in the development of standard guidelines for the use of composites in bridge repair in Ohio.
5. Evaluate the material and structural properties of the carbon fiber rods in the laboratory by performing tests on representative specimens;

Shortly before installation FRS, with the approval of ODOT engineers, substituted glass fiber reinforced rods with a lower design tension for the carbon fiber rods originally specified. Therefore the performance evaluation identified in the first research objective was necessarily changed to one of simply monitoring the behavior of the bridge as stiffened with the glass fiber composite rods. Further, the contractor expressed concern that the proprietary nature of the rods and their attachments would not be protected if representative specimens were provided to OSU researchers. Since the laboratory test program proposed in research objective No. 5 required specimens to test, ODOT engineers were informed that, unless the contractor provided specimens for laboratory testing, no independent evaluation of the properties of the reinforcements was possible.

GENERAL DESCRIPTION OF RESEARCH

In the last two decades, fiber reinforced composites have become important components of many engineered structures. Although the majority of these applications have been in the area of high performance structures, several demonstration and full-scale composite bridge structures to support pedestrian, bike, and vehicular traffic have been proposed.

This study focused on measuring the effects of the attachment of glass fiber tension rods to an existing reinforced concrete bridge subjected to various types of static and dynamic traffic loads and environmental conditions. Table 1 gives an overview of the reinforcement and bridge properties. Four tension rods were placed on each of the outside beams, two rods on the inner face and two on the outer face (8 rods total). The rods were 9.5mm in diameter and 12 meters

long as measured from bracket to bracket. Design tension was 35.6 kN (8,000 lbs) per rod. Figure 1 is a photograph of the bridge taken during testing. Figure 2 shows the position of two of the installed rods. The rods were attached to steel brackets as shown in Figure 3. The brackets were bolted to the beams of the bridge and the rods tensioned by tightening the nut with a torque wrench (Figure 4).

Bridge	COS-79-0955
Location (County)	Coshocton
Spans	1
Span Length (m)	14
Deck	Reinforced Concrete
Beams	Concrete T-beams
Number Longitudinal Girders	6
Transverse Girders	0
Number of Lanes on bridge	2
Post-Tensioning Rods	
Material	Fiberglass
Length (m)	12
Diameter (mm)	9.5
Design Tension (kN)	35.6
Bracket Connection	Bolted
Load Cells	2
Strain Gauges	6
LVDTs	2

Table 1 Bridge and Reinforcement Properties

Figure 1 Bridge COS-79-0955. (Looking East)

Figure 2 Typical Reinforcement Placement

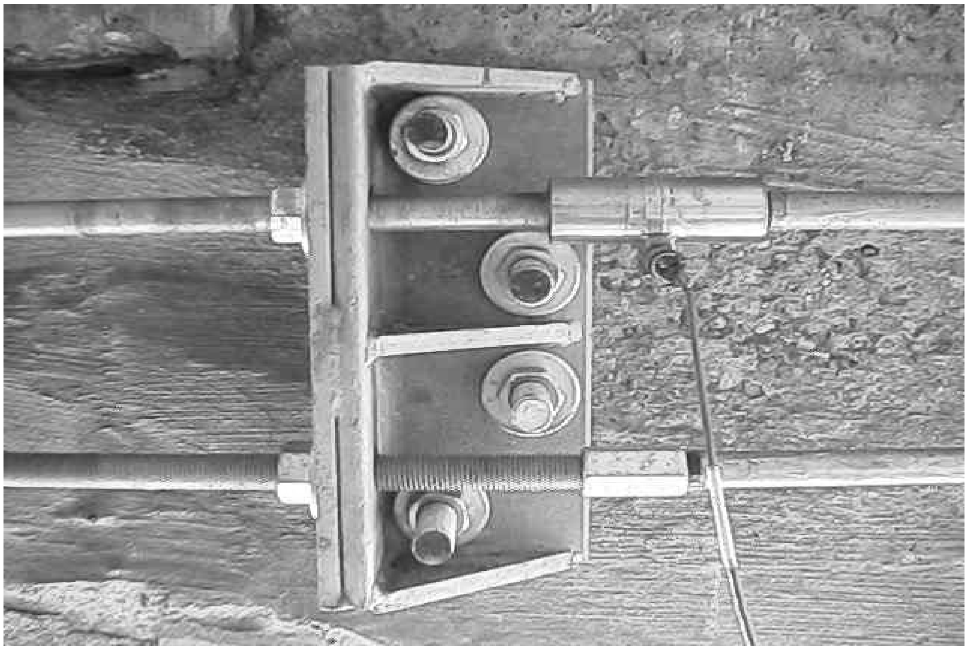

Figure 3 Standard Bracket Assembly

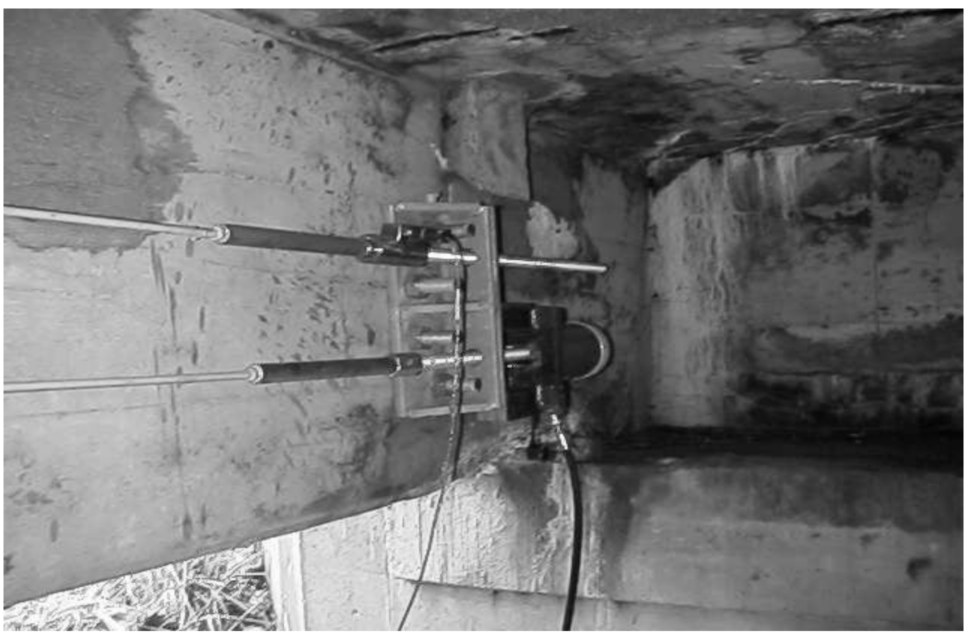

Figure 4 Tensioning the Rods.

FIELD TESTING

Static and dynamic load tests were performed on the bridge before and after tensioning the composite rods. The loading consisted of two ODOT trucks with measured axle spacings and estimated weight distributions. Both static tests (the trucks were placed a specific locations on the bridge) and dynamic tests (the trucks were driven across the bridge at a specified speed) were conducted. The location of the applied load was recorded for the static tests and the vehicle speed and truck locations relative to each other were monitored and recorded for each dynamic test. Table 2 lists the protocol details. Figure 5 shows the tandem dump trucks used to load the bridge. Figure 6 is a photograph of a side-by-side dynamic test while Figure 7 is an example of a front-to-back static test and Figure 8 shows the trucks during a front-to-back dynamic test.

COSHOCTON					
Test	Lane Location		Relative Position of The Two Trucks	Speed (km/hr)	Figure Showing Response Time History
	Truck 1	Truck 2			
1100-02	N	N	FB	8	C.1
1100-03	S	S	FB	80	C.2
1100-04	N	N	FB	50	C.3
1100-05	S	S	FB	50	C.4
1100-06	N	S	SS	50	C.5
1100-07	S	N	SS	80	C.6
1100-08	S	S	FB	static	C.7
1100-09	N	S	SS	static	C.8
1100-11	N	N	FB	8	D.1
1100-13	S	S	FB	80	D.2
1100-14	N	S	SS	50	D.3
1100-15	S	S	FB	50	D.4
1100-17	N	-	N	static	D.5

Note: N is northbound. S is Southbound. FB is front-to-back. SS is side-by-side.

Table 2 Test Program

Figure 5 Side-By-Side Static Load Test

Figure 6 Side-By-Side Dynamic Test. Trucks Are Southbound

Figure 7 Front-To-Back Static Load Test. Trucks Are In Northbound Lane.

Figure 8 Front-To-Back Dynamic Load Test. Trucks Are Southbound.

INSTRUMENTATION

The loads in selected rods, deflections at mid-span, and strain at selected locations were measured. Load cells, linear variable differential transformers (LVDTs), and strain gauges were the transducers used to measure these quantities. The data collection rate for each test was 100 samples per second per channel. Figure 9 gives the bridge instrumentation layout.

Load Cells

Figure 10 shows one of the load cells in place on the top rod inside each of the outer beams of the bridge. Notice the end of the composite rod is screwed directly into the load cell. Load was recorded over a period of several months to determine whether the rods maintained the design tension.

Strain Gauges

Typical strain gauges were installed in the locations identified in Figure 9. Six gauges, two at the mid-span and four at the quarter-spans were attached to the bottom of the concrete beams.

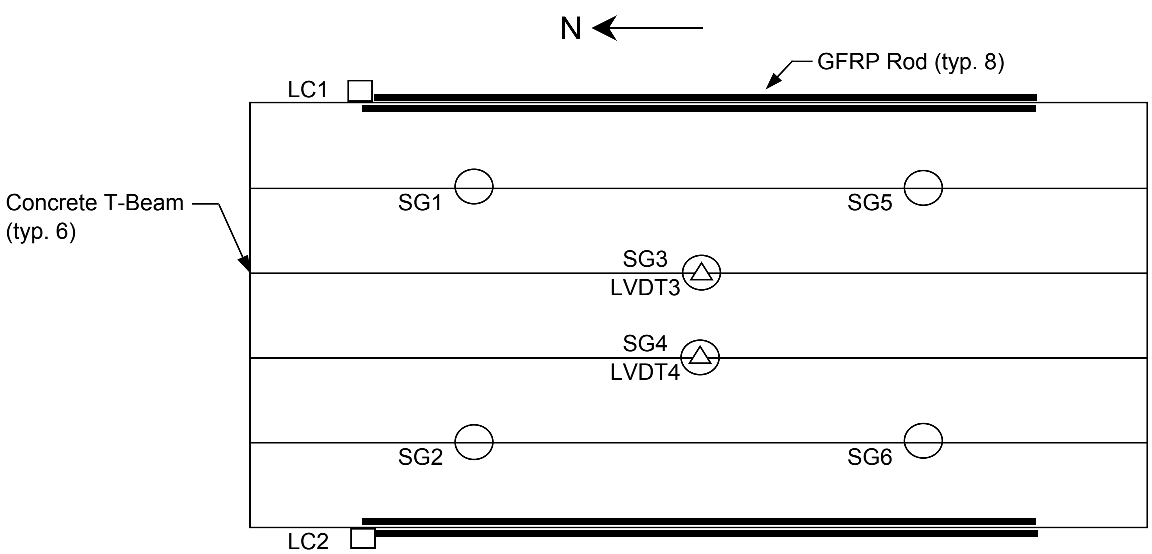

Figure 9 Instrumentation Plan.

Figure 10 Load Cell Attached to Composite Rod.

Linear Variable Differential Transformers (LVDT)

The LVDTs were mounted to a steel frame beneath the two innermost beams at the mid-span (adjacent to the strain gauges) to measure the deflection of the bridge deck. As shown in Figure 11, the spring-loaded tip of the LVDT was placed in contact with the bottom of the concrete bridge beam.

Figure 11 Typical LVDT Installation.

TEST RESULTS

Plots of the measured response time histories for each test are compiled in Appendices C and D. Summaries of those tests are presented in this section. Table 3 is a listing of the maximum deflections recorded during each load test. Where two values are given they represent the peak responses of each LVDT in the tests where trucks crossed the bridge one behind the other. The first value is the maximum deflection due to the first truck and the second value corresponds to the second, heavier truck. Figure 12 is an example of one of these deflection time histories. The mean decrease in deflection for all speeds and LVDT locations was about 0.04 mm or 4%. However, the standard deviation for the deflections was 0.05 mm, so changes in the deflection of the deck after the composite rods were tensioned, were essentially unchanged. What was observed was the repeatability of the measurements. With no effect attributed to

the rods, the instrumentation was capable of measuring the displacements of the bridge deck to within 0.05 mm.

Test	Deflection LVDT 3 (cm)	Deflection LVDT 4 (cm)
Before Tensioning		
1100-02	0.0612	0.0495
	0.0663	0.0556
1100-03	0.0538	0.0698
	0.0587	0.0795
1100-04	0.0602	0.0467
	0.0724	0.0577
1100-05	0.0457	0.0632
	0.0510	0.0744
1100-06	0.1151	0.1265
1100-07	0.1148	0.1369
1100-08	0.0434	0.0658
1100-09	0.1067	0.0998
Post-tensioned		
1100-11	0.0668	0.0513
	0.0630	0.0475
1100-13	0.0546	0.0747
	0.0523	0.0668
1100-14	0.1056	0.1156
1100-15	0.0444	0.0602
	0.0493	0.0686
1100-17	0.0493	0.0363

Table 3 Maximum Displacements

Table 4 lists the maximum strains for each test. As in Table 3, two values are given for each strain gauge in those tests where trucks crossed the bridge one after the other. The strains varied widely but the recorded average peak strain actually appears to have increased by 2% after the rods were tensioned. As was the case with the displacements, the measured strain increase is within the range of scatter in the data. With small amplitudes such as those measured during the load tests, a change from one test to the next of the position of one truck relative to the other, or the truck speed could account for the strain differences observed.

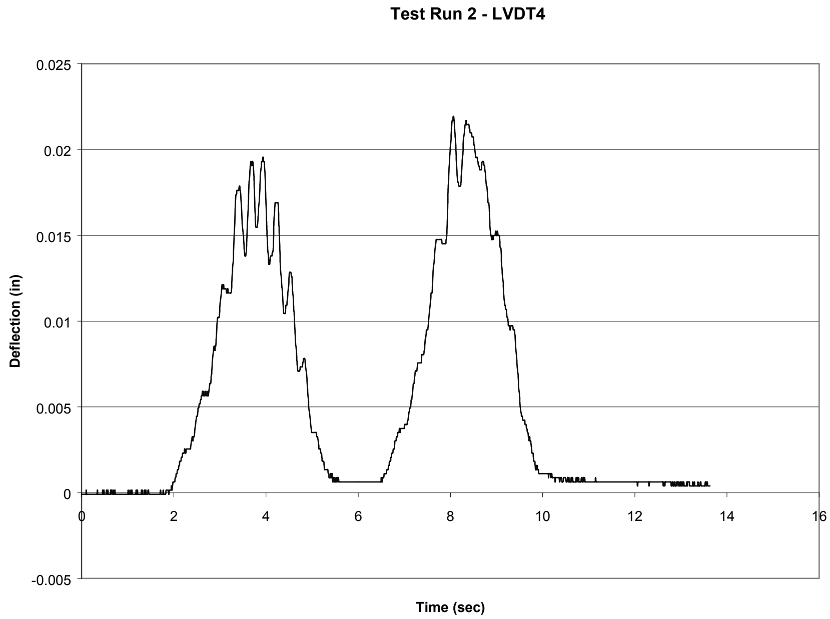

Figure 12   Typical Field Deflection Data

The load on the rods did not change as the trucks crossed the bridge. This can be seen in the plots of the load data (after tensioning) shown in Appendix D. Figure 13 is a summary of the monthly load readings through July 2001. As seen in the figure, the rods have maintained the initial load. The rod with load cell 1.1 attached has shown a slight increase in load over the monitoring period. The temperature was just above freezing when the rods were initially tightened in November, therefore any deviations above or below freezing could result in different expansion or contraction of the rods relative to the concrete beams. These results suggest that relaxation of the fiberglass rods should not be a concern.

Test	SG1	SG2	SG3	SG4	SG5	SG6
Before Tensioning						
1100-02	13	9	6	14	12	6
	17	9	6	14	17	11
1100-03	10	18	5	18	4	18
	5	18	5	23	4	18
1100-04	14	9	5	14	10	5
	19	9	5	14	15	10
1100-05	6	19	5	19	6	24
	6	19	5	19	6	29
1100-06	19	32	9	23	19	32
1100-07	27	27	11	26	14	27
1100-08	8	22	2	17	4	28
1100-09	27	27	7	22	19	23
After Tensioning						
1100-11	13	6	9	17	14	5
	18	6	9	12	14	5
1100-13	6	19	6	20	6	23
	6	19	6	20	6	23
1100-14	22	34	7	25	17	28
1100-15	5	14	4	19	10	24
	5	14	4	19	10	28
1100-17	10	4	4	9	19	9

Note: All values given in microstrain ($\mu\varepsilon$).

Table 4 Peak Strain Values.

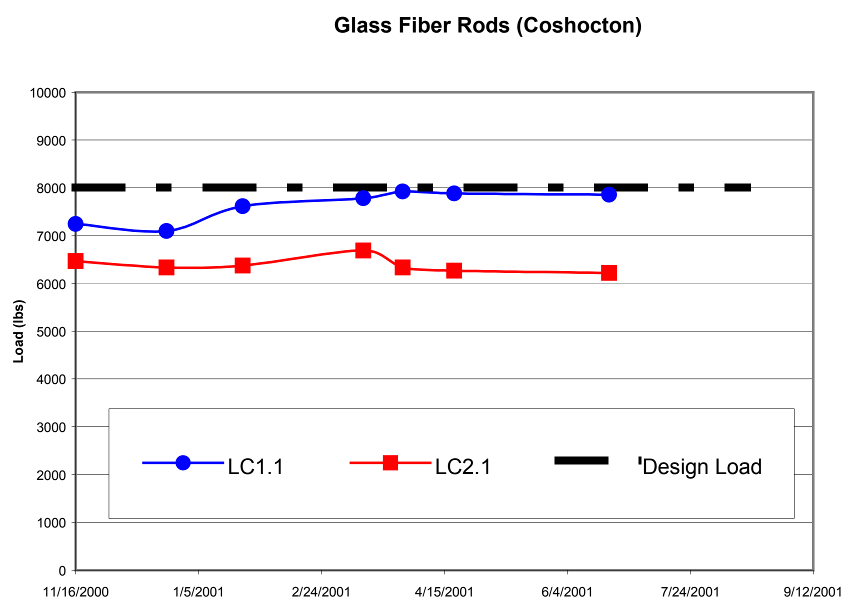

Figure 13 Loads in Rods as a Function of Time

MODEL DESCRIPTION

In order to perform an independent check of the field measurements and to suggest an approach to predicting the effect of external reinforcements on similar bridges, a finite element model using the commercial software package, ANSYS, was developed. To compare the measurements recorded during the load tests with a computed dynamic deflection, the bridge was modeled as a twenty element, two-dimensional beam using standard elements included in the ANSYS library (Figure 14). Each node had three degrees of freedom, translation along the length of the bridge, vertical translation, and rotation in the plane of the allowed translations. End conditions were modeled by allowing limited rotation of the bridge deck by adding vertical beam elements to the node at each end of the bridge to simulate the abutments. The two end elements were fixed at their bases resulting in bending being allowed at the node connected to the bridge deck but restricted by the beam stiffness. Figure 15 is a detail of one end of the bridge showing one of the vertical elements. The response of the structure to load was assumed to be linear and elastic.

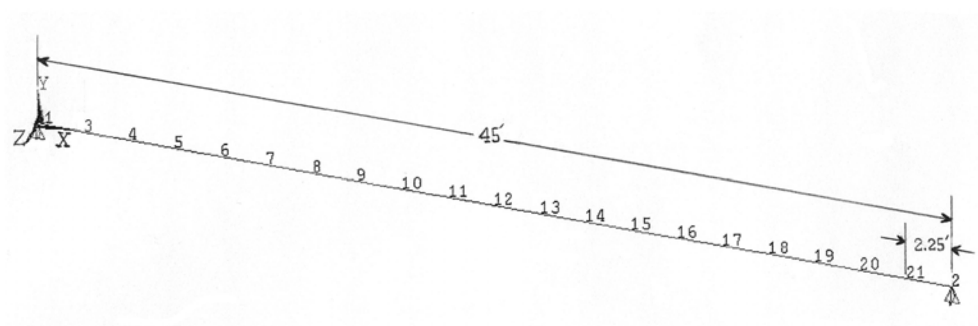

Figure 14 20-element Simply Supported Model.

For a linear elastic beam element, the stiffness (K) is a function of the elastic modulus (E), the moment of inertia (I), and the beam length (L). Since the length of the bridge is known, stiffness was evaluated in terms of the effective modulus and moment of inertia (I_e).

Modeling the bridge as one material required the use of one modulus for both concrete and the reinforcing steel. This should result in a slightly higher value than is appropriate for concrete alone. Nevertheless the elastic modulus was assumed to be equivalent to a standard value for normal strength concrete (22,000 MPa (4.608 x 10^8 psf) Hibbeler, 1994).

The moment of inertia is a function of the height, width, reinforcement, cracking and distribution of area with respect to the neutral axis. The moment of inertia, $0.31m^4$, and the cross-sectional area, $3.6m^2$, were calculated from the plans for the bridge, which were provided by ODOT.

Energy dissipation is modeled by the inclusion of Rayleigh damping. The damping ratio was assigned a value of 0.10 for all modes as suggested by Weaver and Johnson (1987). As a check on this assumption, the effect on the calculated deflection of damping values ranging from 0.0 to 0.15 was evaluated. For truck speeds up to 50 km per hour the calculated deflections differed by no more than 10%.

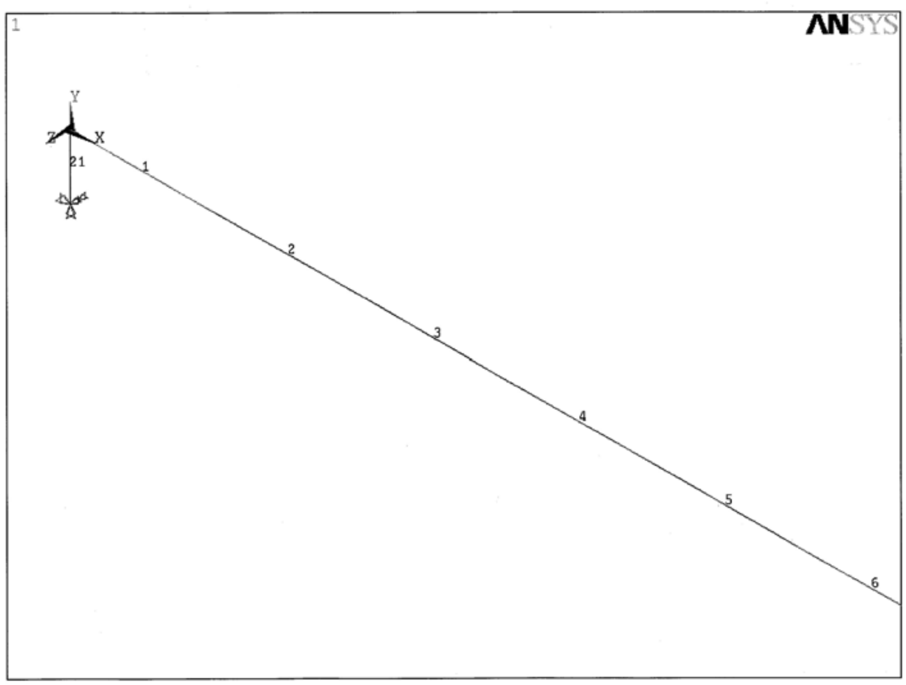

Figure 15 Model Abutment Detail

Since the actual truck loads were not available for the COS-79-0955 tests, the loads to simulate the test trucks as they moved across the bridge were taken from the axle weights obtained for the trucks used to load SCI-23-0096. For computation purposes, the front axle weight was taken to be 75 kN and for the rear axle the load was 180 kN. The load tests before and after tensioning were conducted on the same day with the same trucks, so any effects caused by discrepancies between assumed and actual weights should be negligible. The loading time step (the increment of time required to reach the peak value) was taken to be equal to the time required for the truck to travel the distance of one element. Therefore different time steps were used for the load function in order to represent different truck speeds. The load ramped up at the first node to simulate the truck approaching that point and then decreased to simulate unloading. While the load ramped down at the first node it simultaneously ramped up at the next node so that the full force of each axle was continuously applied to the bridge. Figure 16 is a graphical representation of the loading.

The amount of end constraint had the greatest effect on the calculated response to loading so the vertical beam stiffness was adjusted until the predicted deflections of the pre-tensioned bridge agreed with the measured values. With the properties of the bridge thus determined, the effects of the tensioned composite rods on the response of bridge to loads could be evaluated. The effect of the rods was primarily due to the compressive load and the moment about the neutral axis. The modulus of the composite rods would not contribute significantly to the effective modulus of the bridge (the fiberglass rods provided a negligible increase in either flexural or extensional rigidity). Because the rods were placed only on the outer beams, the post-tensioning effect is not even over the cross-section of the bridge. However, Klaiber et al. (1981, 1983) demonstrated the feasibility of strengthening a 4-beam simple span bridge on steel beams by post-tensioning only the exterior beams. The results of that study showed that when only the exterior beams were post-tensioned, about two-thirds

of the post-tensioning remained in the exterior beams while the remainder was distributed among the interior members. Since the subject bridge is relatively small, the effects of the post-tensioning should contribute some added strength at the center, although less than at the outer beams. The LVDTs were placed below the center beams because this was where deflection would be greatest.

The composite rods were modeled as an applied compressive load and moment. The load was applied as a 285 kN force (8 rods x 36 kN design load) at the nodes representing the bracket locations. A moment of 46.5 kN·m, (285 kN x 0.163m) was applied to the same nodes. Both the moment and the force remained constant for the duration of the simulated truck loading.

At a speed of 50 km/hr the model predicted a decrease in maximum deflection of 2.4% in the reinforced bridge. Since the bridge is modeled as a 2-D beam, the compressive stress is applied evenly over the cross-section and the variation in stress across the beam is not properly accounted for. The actual bridge will have a higher concentration of stress near the brackets and less added compressive stress at the top and bottom of the beam and correspondingly less benefit. The loads applied by the composite rods did not increase the stiffness enough to document a verifiable change in the deflection.

SUMMARY AND CONCLUSIONS

The deflection results suggest no actual decrease in the maximum mid-span deflection occurred, after the rods were installed and tensioned. The slight difference between the before and after deflections is less than the variability that should be expected in the readings. Strain results also suggest no improvement. Monthly load readings for the composite rods show the rods have maintained their initial tension.

The improvement suggested by the model is small enough that slight variations in the conditions between field tests (such as truck speed or position) could nullify the change. Using the model to evaluate the effect of varying rod tension, a relationship between the decrease in deflection and rod tension was

developed. For example a 10% reduction in deflection would require each rod to carry 153 kN (34,500 lbs). Incorporating loads five times, or greater, the original design load would require reanalyzing the design of all parts to determine if such loads are feasible. It is possible that the gripping system used to attach the rod ends to the bridge may be able to withstand loads of this magnitude but no testing was performed (see above explanation). A greater total load could be applied by increasing the number of rods. Increasing the number of rods (without increasing the load on each rod) would spread the load more evenly across the width of the bridge by placing the additional rods on the center beams. Increasing the loads on individual rods might be best accomplished by using a higher strength material such as the carbon fiber rods originally specified for this project.

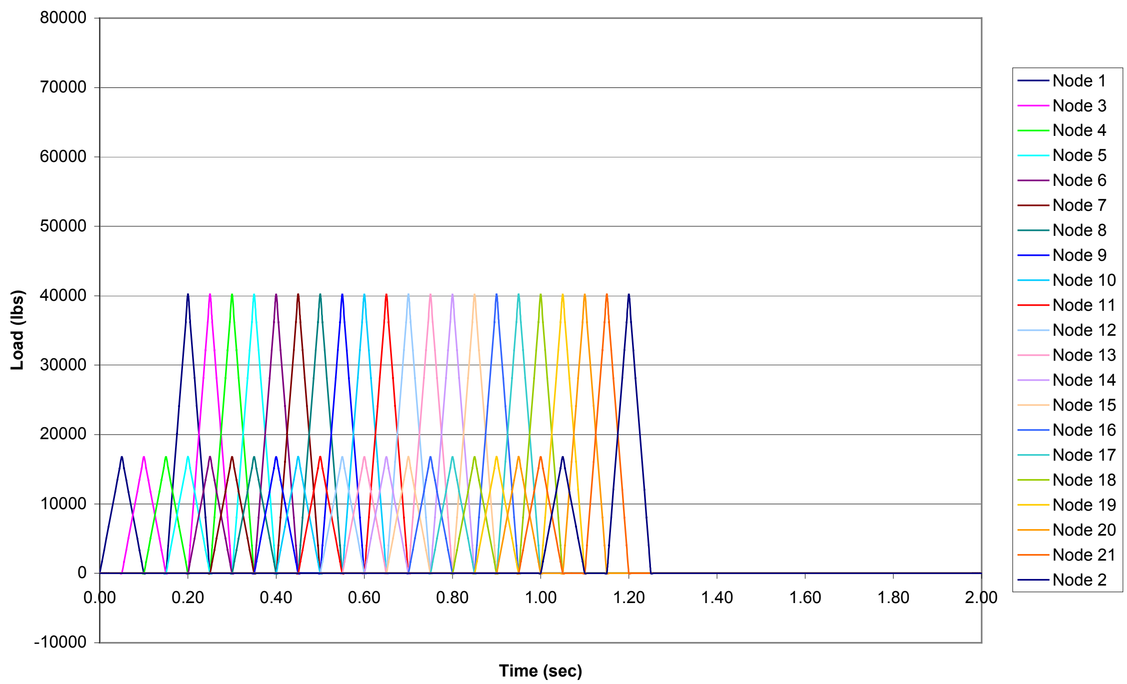

Figure 16 Load Function.

APPENDIX A

BIBLIOGRAPHY

Hibbeler R.C., *Mechanics of Materials*, 2nd ed. Englewood Cliffs, New Jersey: Prentice-Hall, 1994.

Klaiber F.W., Dunker K.F., and Sanders W.W. 1981. "Feasibility Study of Strengthening Existing Single Span Steel Beam Concrete Deck Bridges". Final Report, Engineering Research Institute Project 1460, ISU-ERI-Ames-81251, Iowa State University.

Klaiber F.W., Dedic D.J., Dunker K.F., and Sanders W.W. 1983. "Strengthening of Existing Single Span Steel Beam and Concrete Deck Bridges". Final Report – Part I, Engineering Research Institute Project 1536, ISU-ERI-Ames-83185, Iowa State University.

Ohio Department of Transportation. Map of Coshocton County. May 2001. From website: http://www.dot.state.oh.us/map1/ohiomap/images/county/coshoc.gif

U.S. Federal Highway Administration, Washington, D.C., *Status of the Nation's Highways, Bridges, and Transit: Conditions and Performance,* January 1999.

Weaver W., Johnston P.R., *Structural Dynamics by Finite Elements,* Prentice-Hall, Englewood Cliffs, New Jersey, 1987.

APPENDIX B

Location of Bridge COS-79-0955

(Ohio Department of Transportation, 2001)

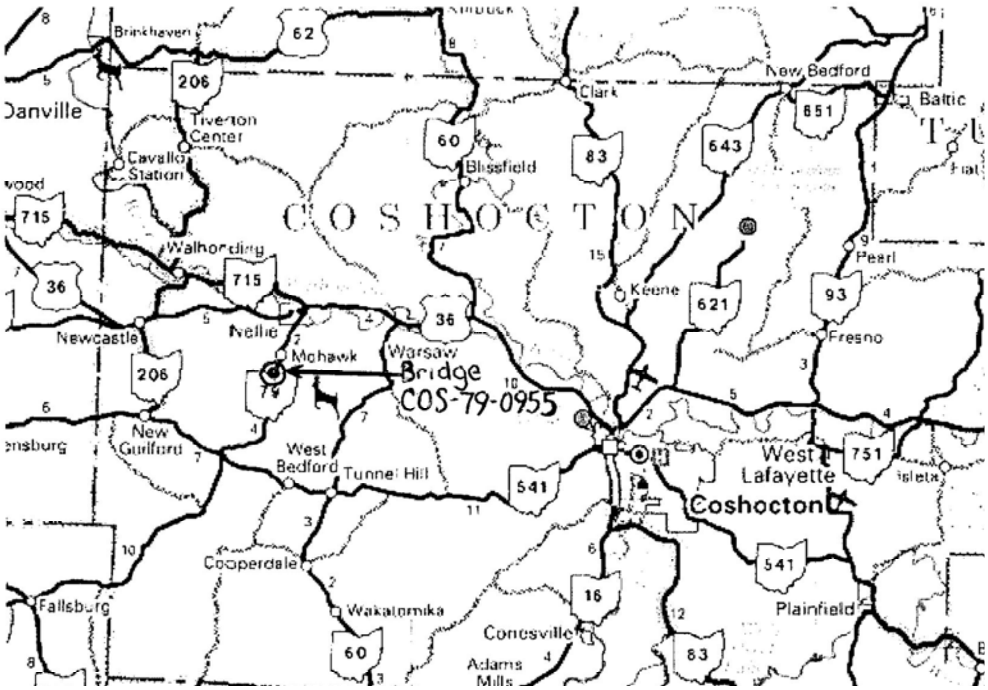

Figure B-1 Location of Bridge COS-79-0955

(Ohio Department of Transportation, 2001)

APPENDIX C

Load Test Results Before Post-Tensioning

The following are plots of test data collected during the load tests performed on November 16, 2000. The fiberglass reinforcing rods were tensioned later the same day.

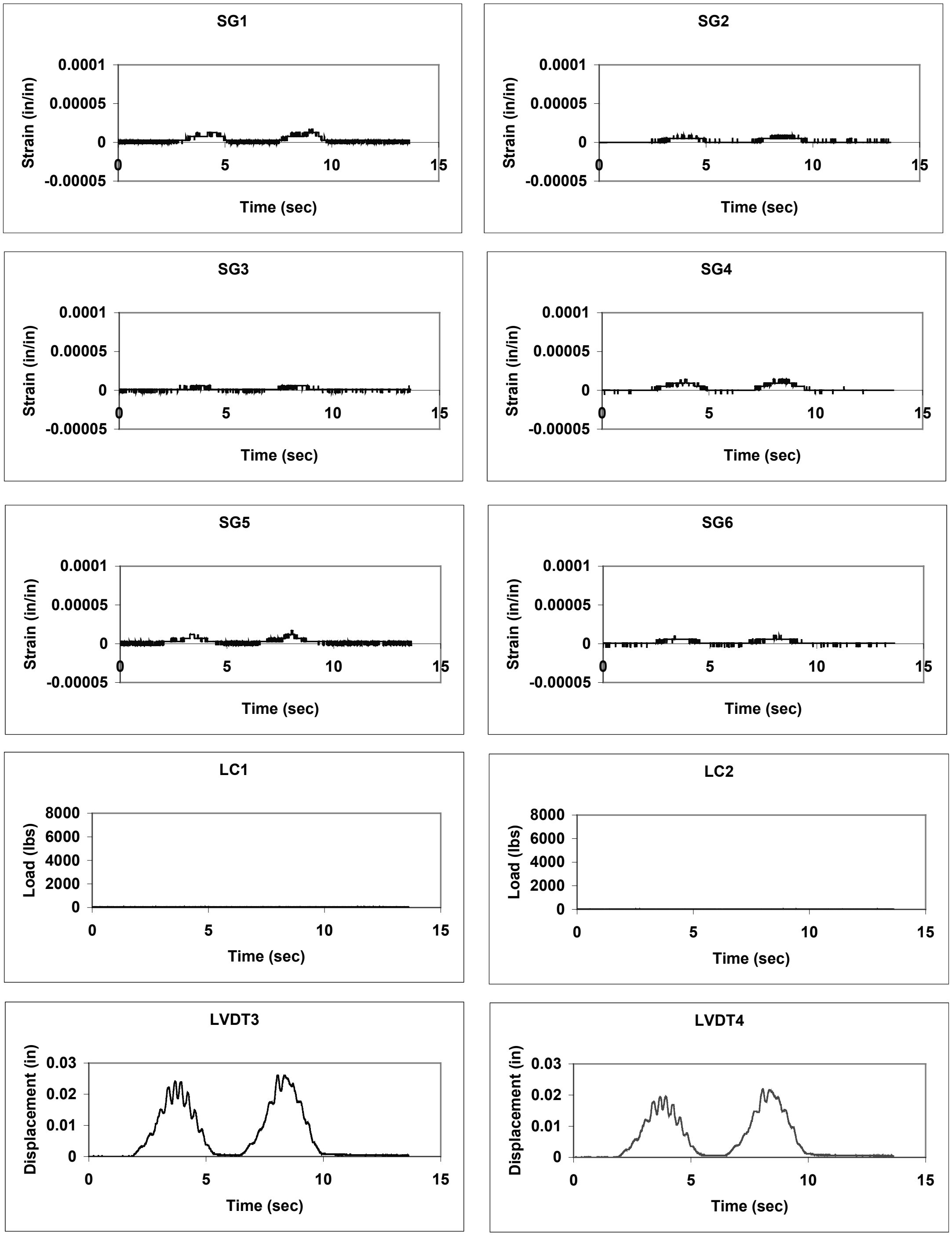

Figure C.1 Bridge COS-79-0955 Test 1100-02.

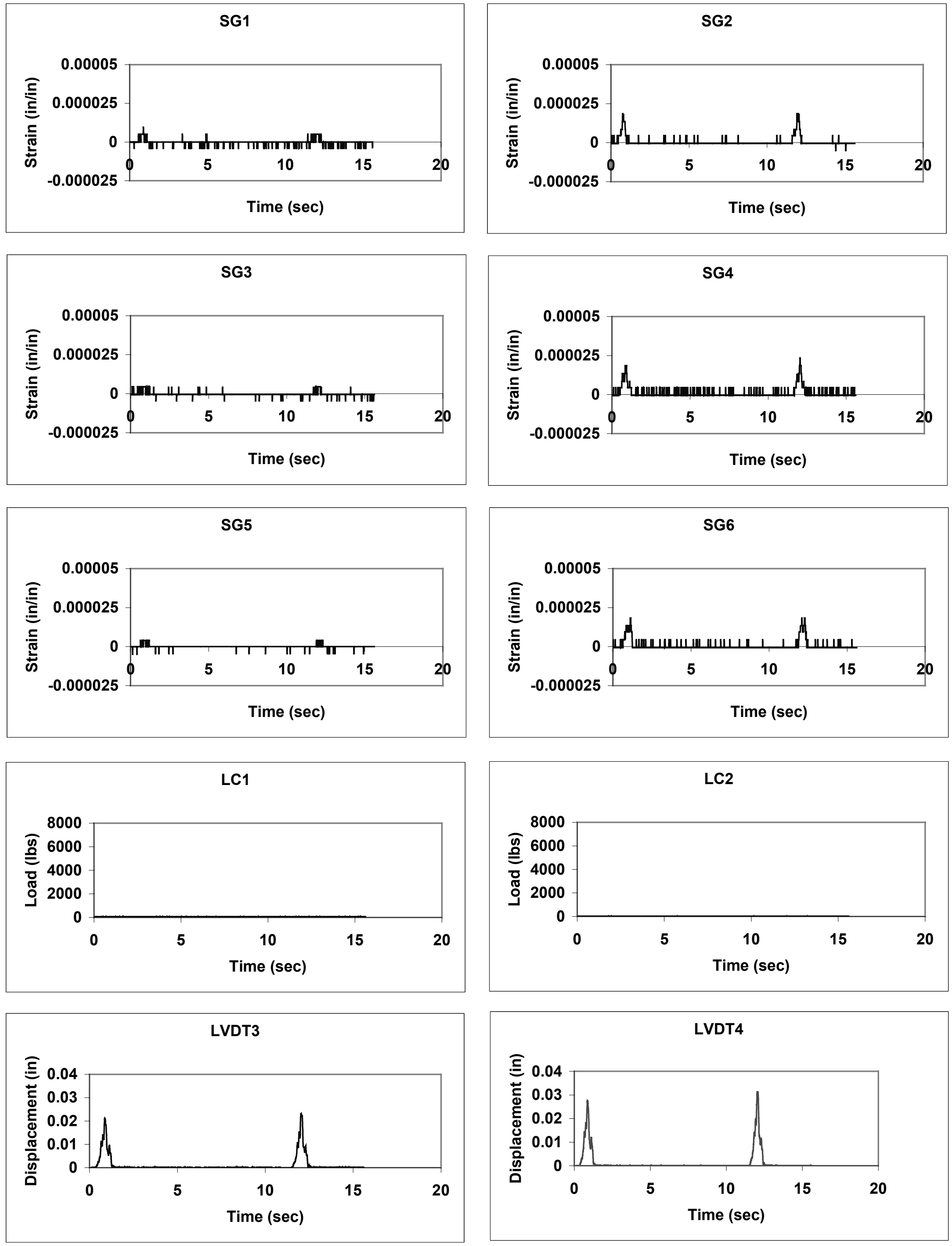

Figure C.2 Bridge COS-79-0955 Test 1100-03.

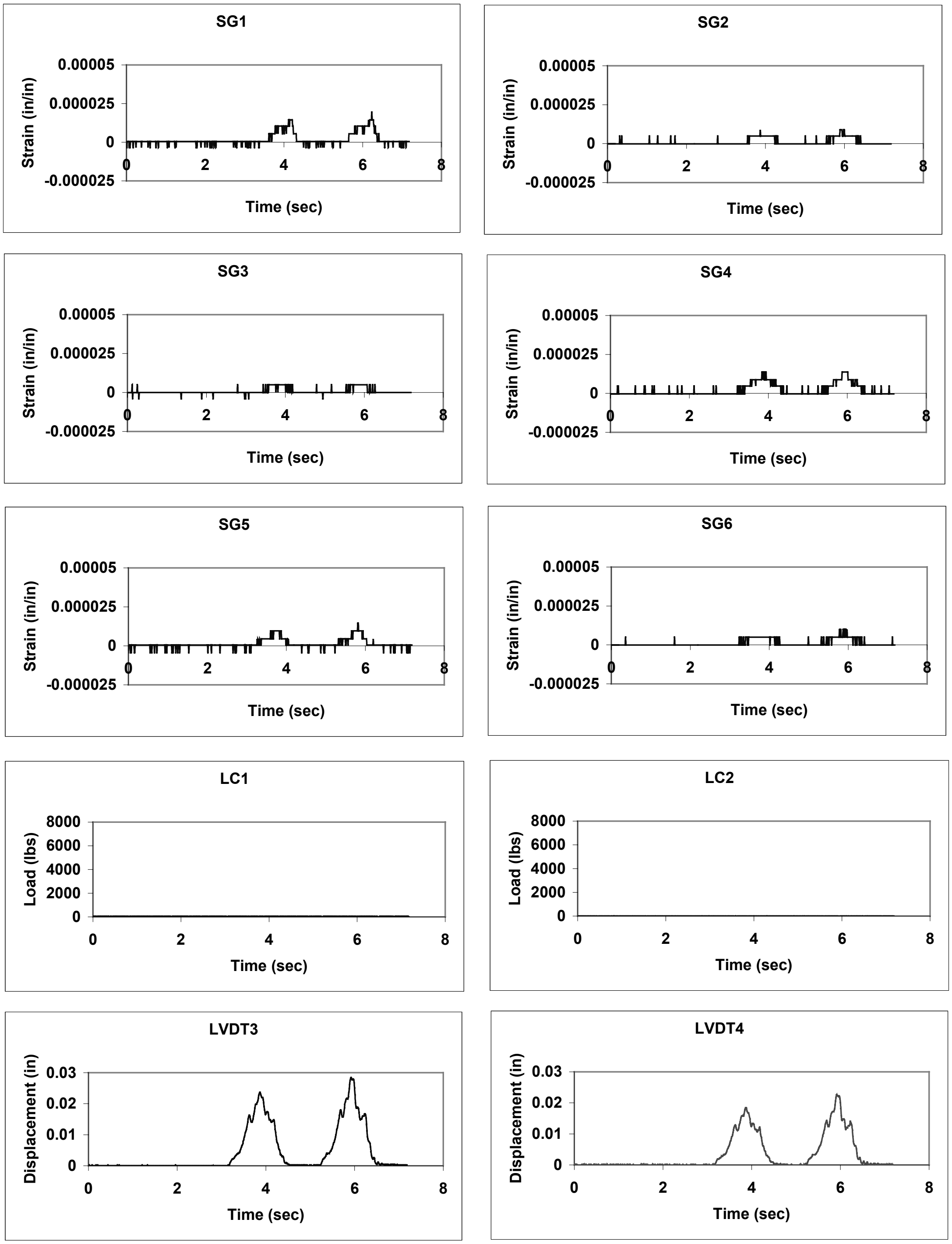

Figure C.3 Bridge COS-79-0955 Test 1100-04.

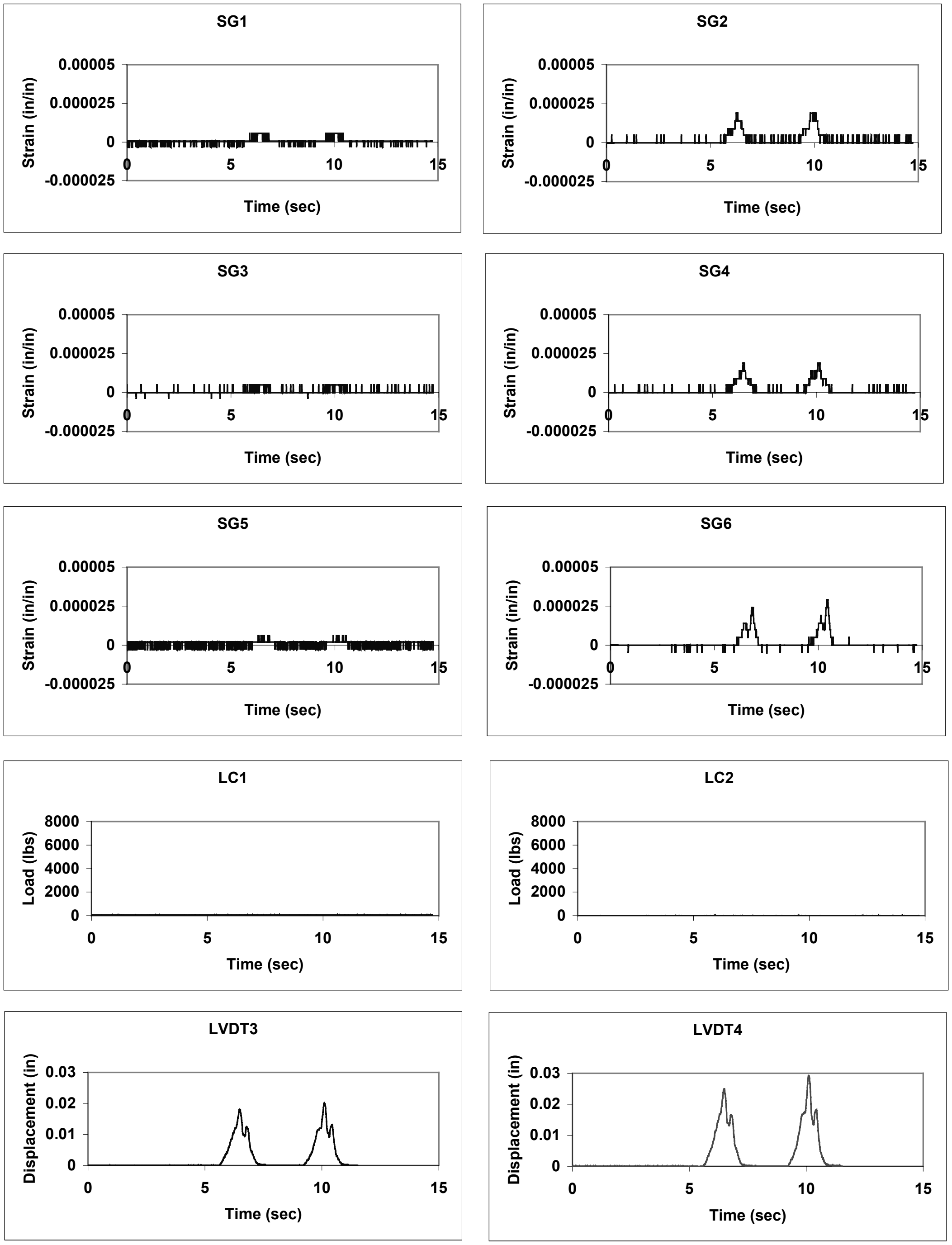

Figure C.4 Bridge COS-79-0955 Test 1100-05.

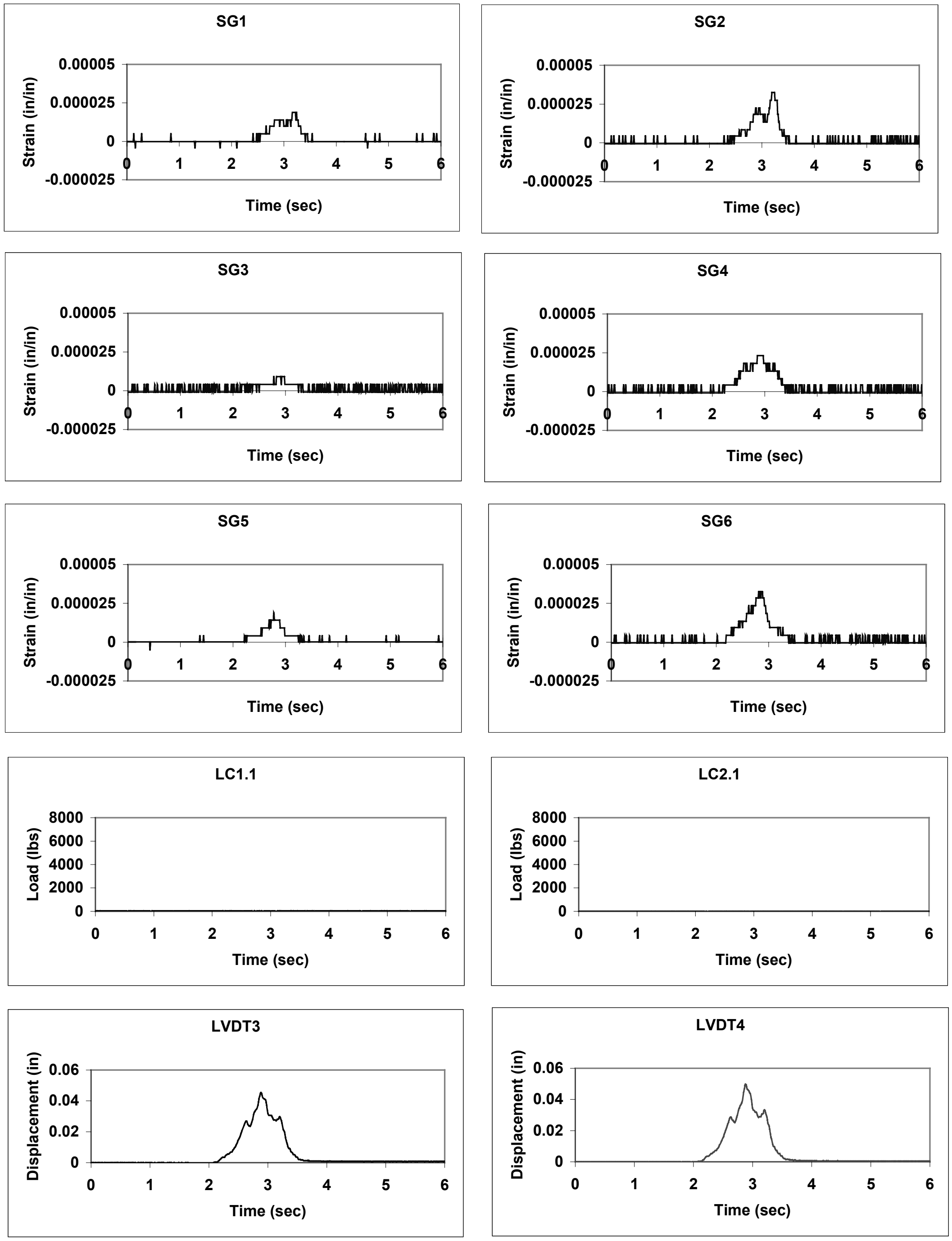

Figure C.5 Bridge COS-79-0955 Test 1100-06.

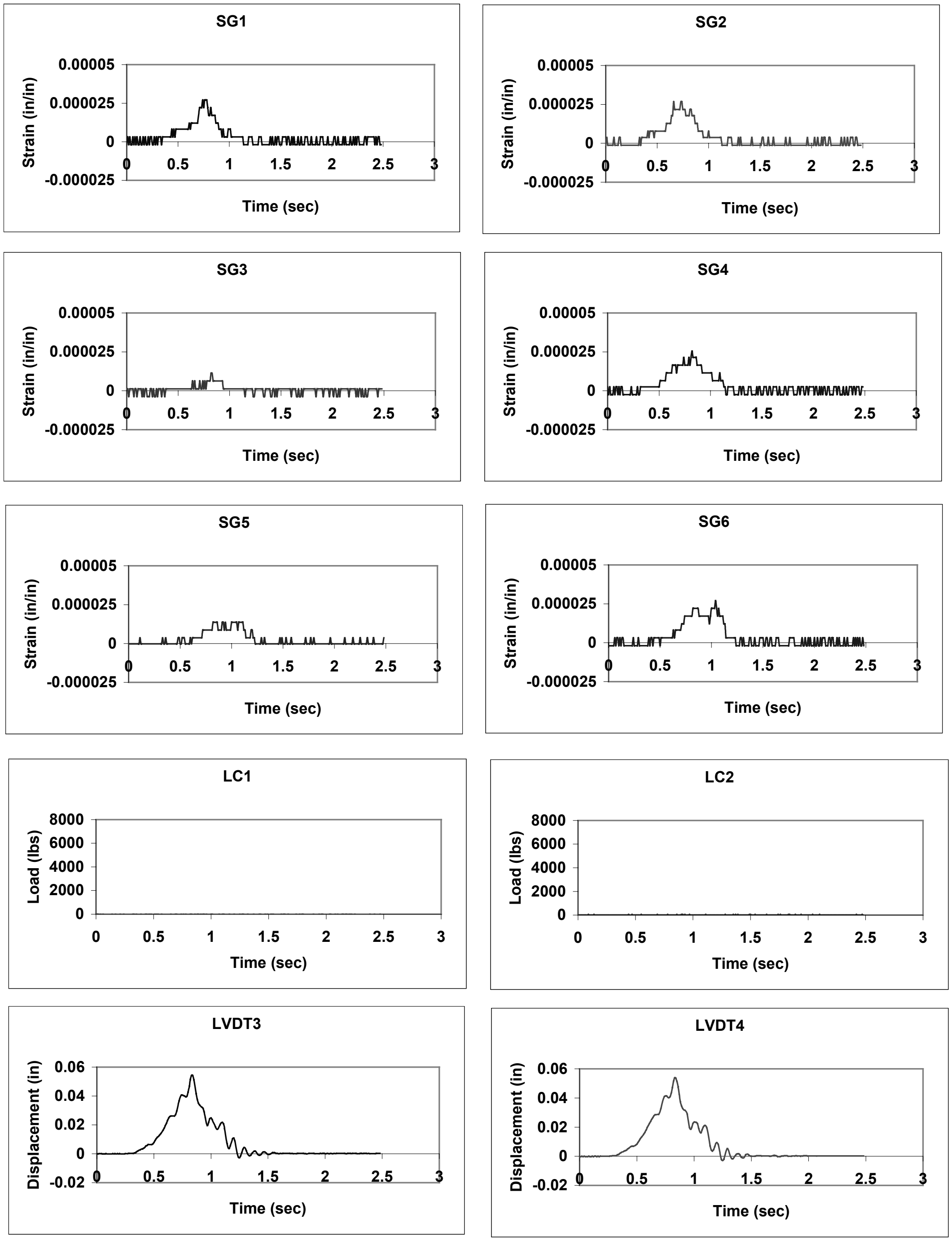

Figure C.6 Bridge COS-79-0955 Test 1100-07.

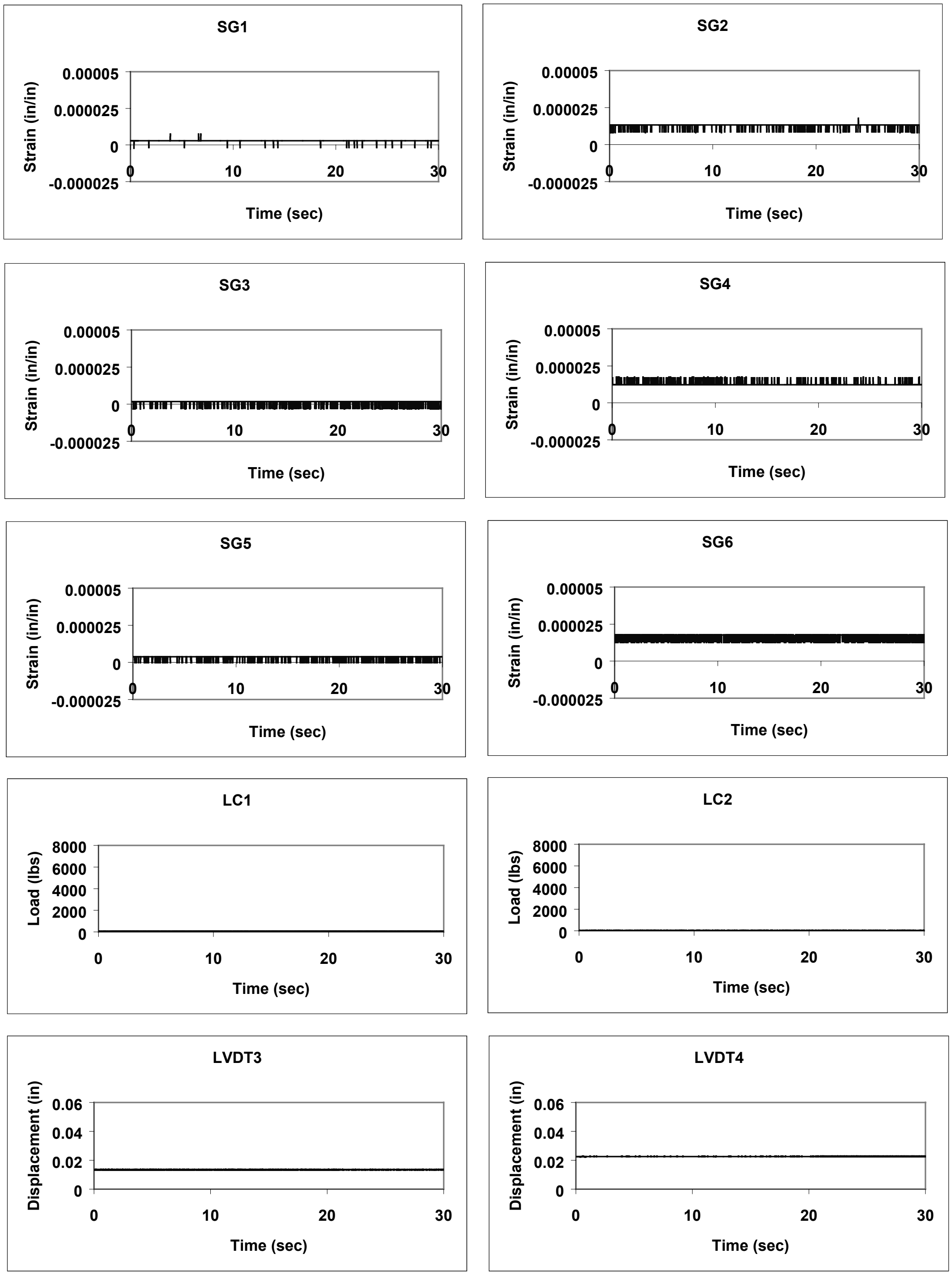

Figure C.7 Bridge COS-79-0955 Test 1100-08.

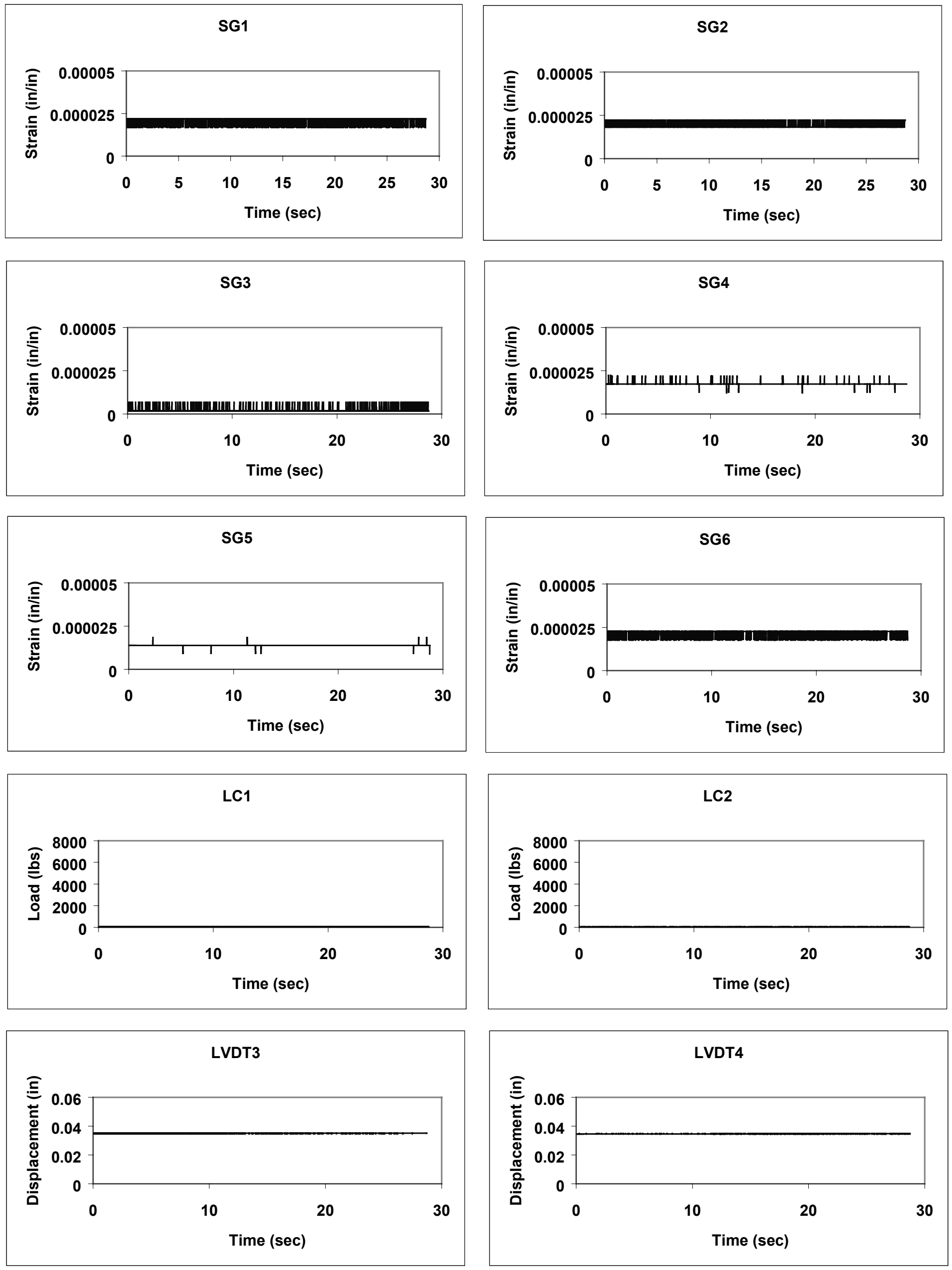

Figure C.8 Bridge COS-79-0955 Test 1100-09.

APPENDIX D

Load Test Results After Post-Tensioning

The following are plots of test data collected during the load tests performed on November 16, 2000 after post-tensioning the GFRP rods the same day.

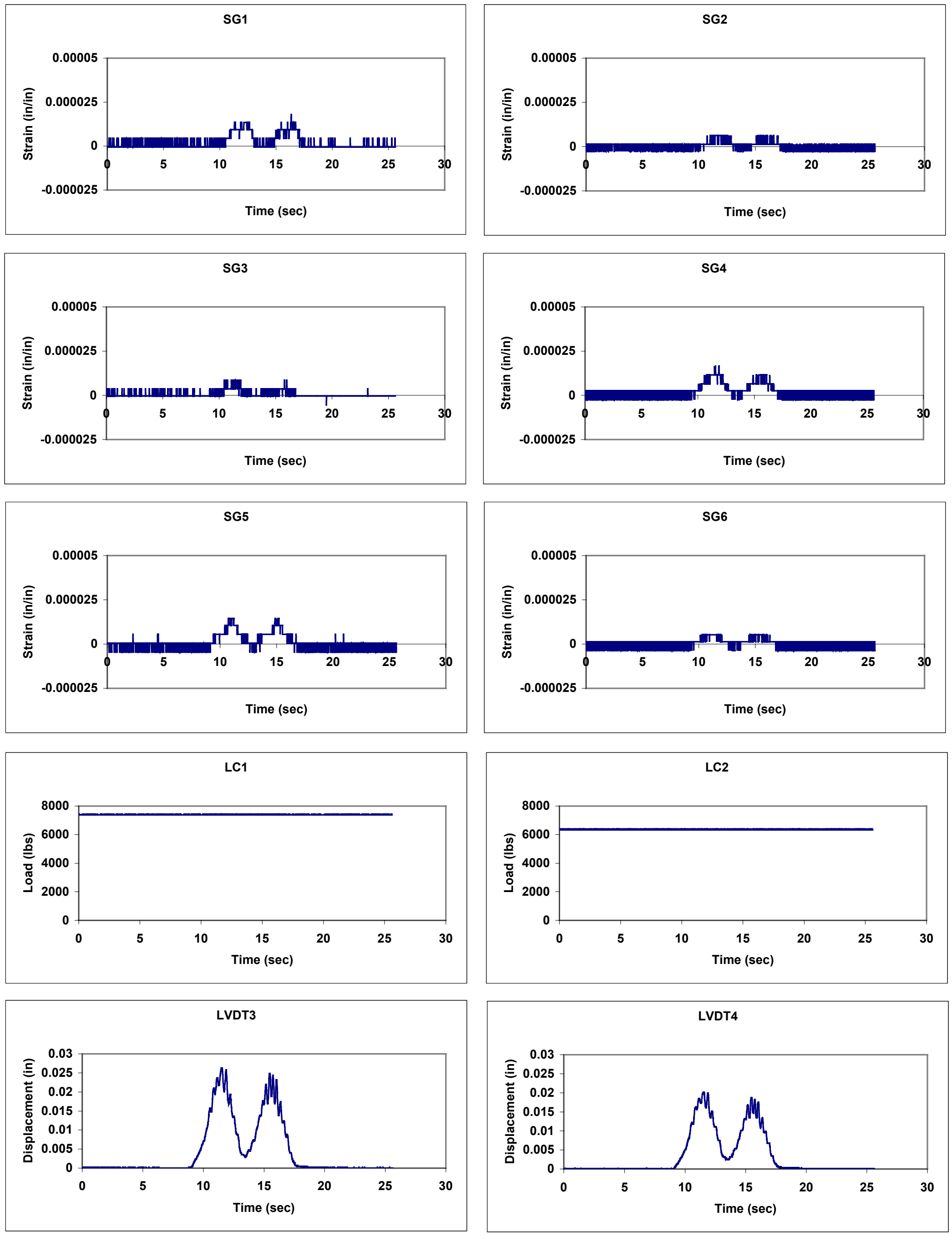

Figure D.1 Bridge COS-79-0955 Test 1100-11.

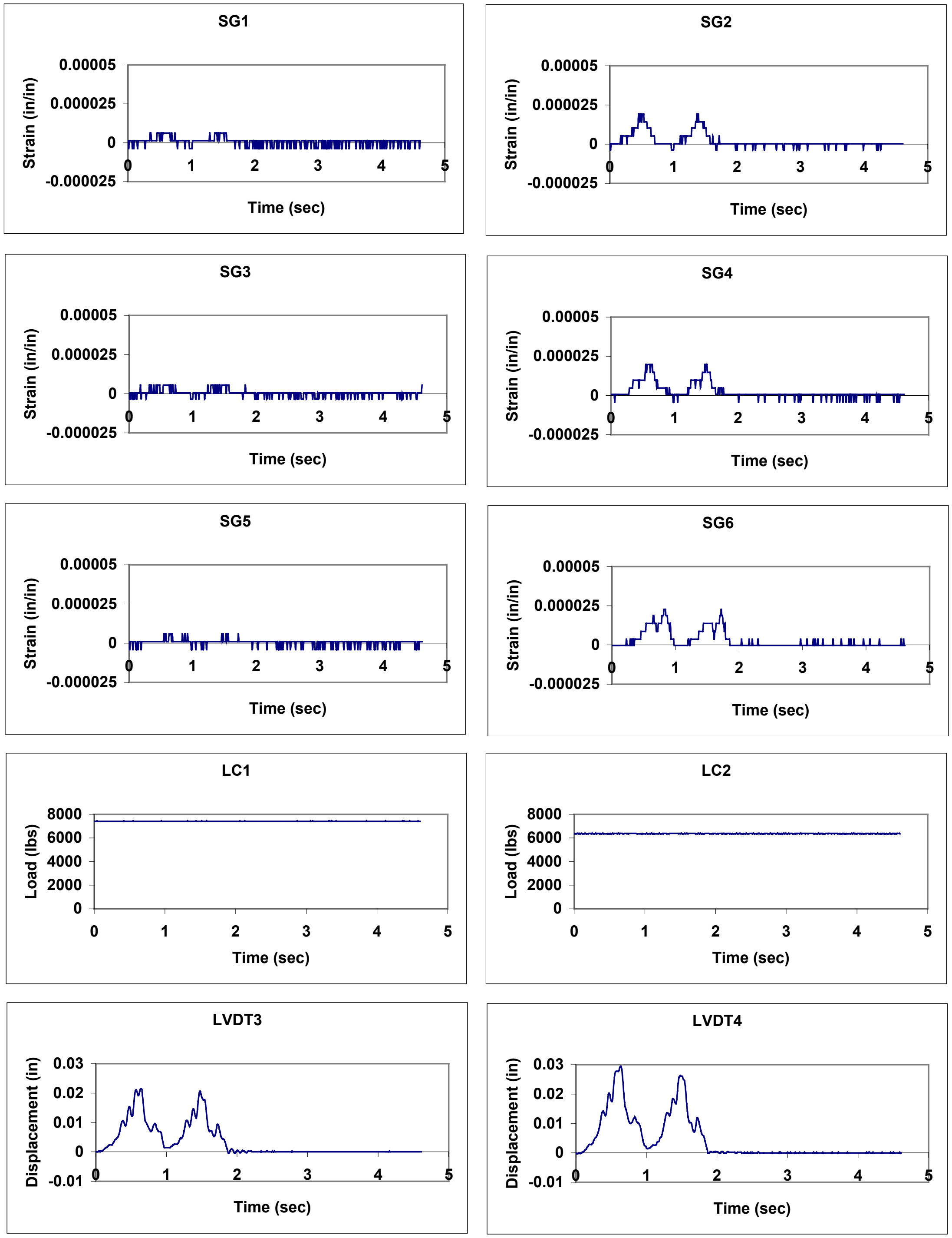

Figure D.2 Bridge COS-79-0955 Test 1100-13.

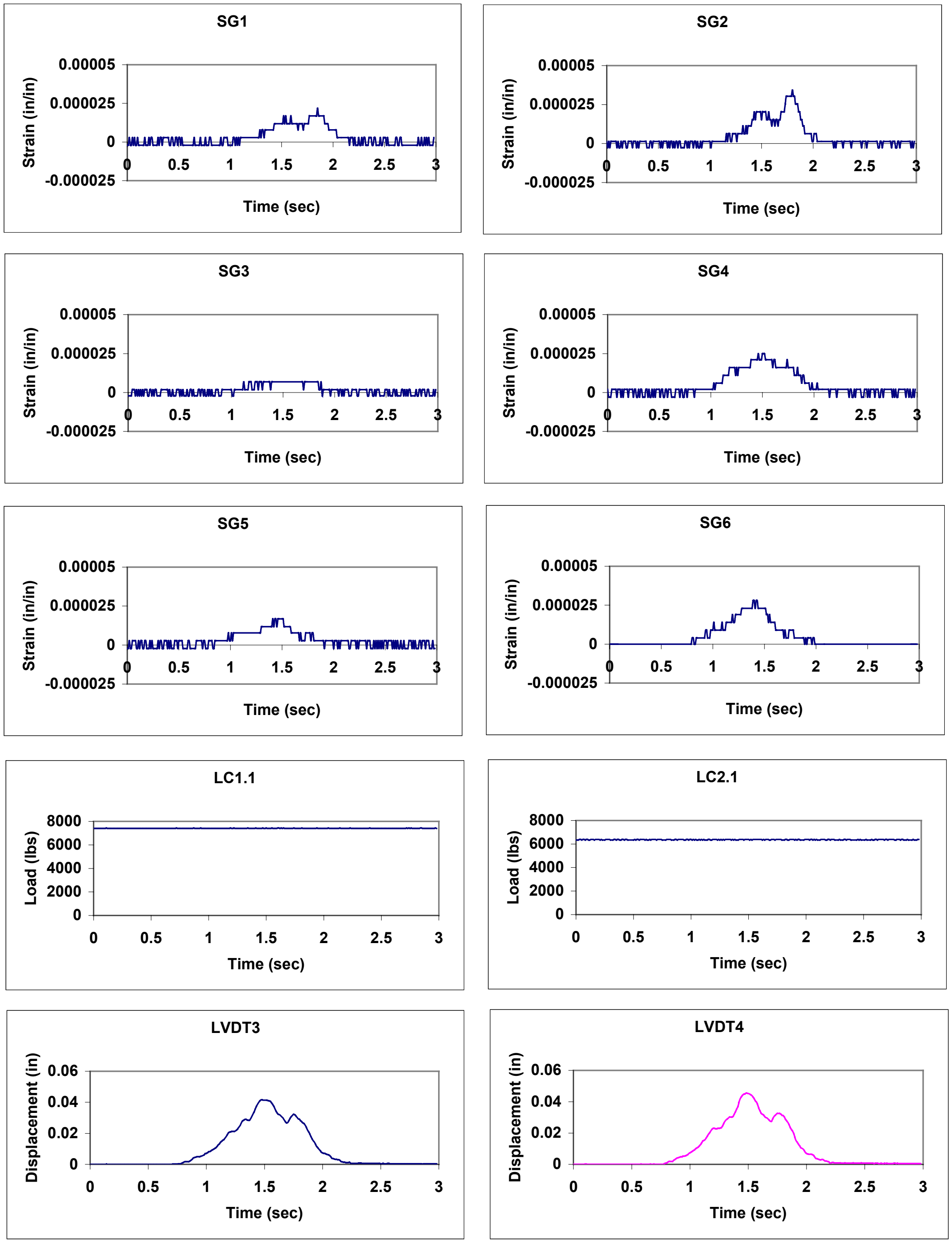

Figure D.3 Bridge COS-79-0955 Test 1100-14.

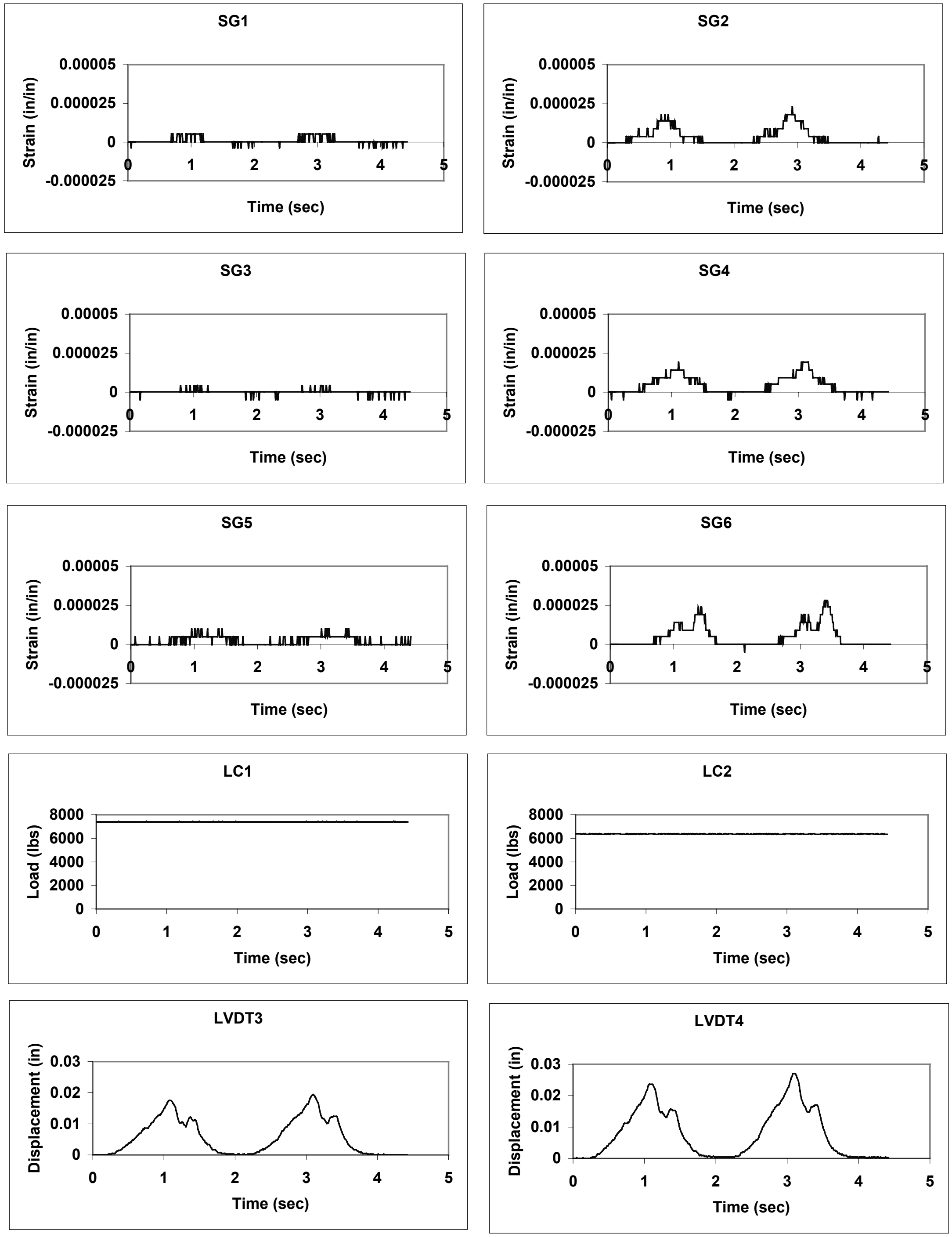

Figure D.4 Bridge COS-79-0955 Test 1100-15.

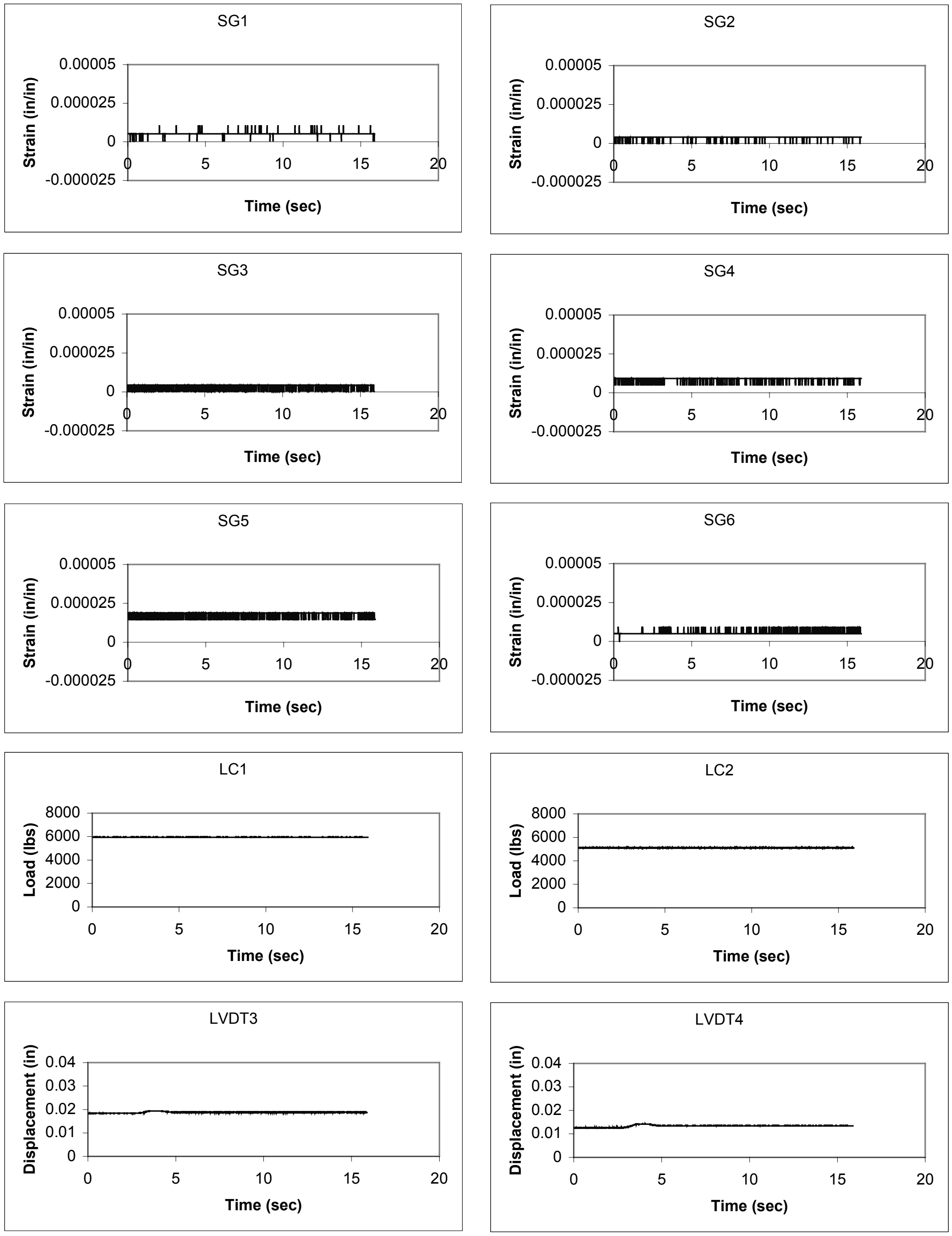

Figure D.5 Bridge COS-79-0955 Test 1100-17.