DCT 157

Evaluating the Load Carrying Capacity of Bridges Without Plans Using Field Test Results

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September 2004

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This work was sponsored by the Delaware Center for Transportation and was prepared in cooperation with the Delaware Department of Transportation. The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views of the Delaware Center for Transportation or the Delaware Department of Transportation at the time of publication. This report does not constitute a standard, specification, or regulation.

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Executive Summary

Bridge load rating has become an integral part of bridge management in the United States. Ratings are used as a means to characterize the load carrying capacity of bridges, to allocate funding for the repair and rehabilitation of bridges, and to approve permit vehicles and superload crossings. Most load ratings are calculated using simple analytical models that are based on information obtained from the structural plans for the bridge; however, for some bridges, particularly for many smaller, older bridges, structural plans may no longer be available. Determining load rating factors for these types of structures is particularly difficult. The resulting ratings are usually based on numerous, conservative, asssumptions. In many cases these structures end up with ratings much lower than they would have if plans were available for the bridge. There is a need for methods to help in determing realistic load ratings factors for bridges for which plans are not available. This report summarizes the results of a study into methods for load rating bridges for which structural plans are not available, based on the results of diagnostic load tests.

As a first step in this investgation, a survey was conducted of the 1997 Delaware bridge inventory to quantify the number of bridges in the state that are without plans and to determine the characteristic features of these bridges. Of the approximately 1300 bridges in the state at that time, 189 were without structural plans. The majority of these bridges have unknown dates of construction or were built before 1940, are constructed of steel or concrete, and are open without restrictions. Of the bridges without plans, 30 were load posted as of 1997. The majority of the posted bridges without plans have unkown dates of construction or were built before 1930, are constructed of concrete, have unknown design live loads, have an operating rating of between 30 and 40 tons and an inventory rating of less than 20 tons. The survey showed that a method for load rating concrete slab bridges without plans would be most beneficial to DelDOT, accordingly, research was focused specifically on this type of bridge.

A brief review of analytical load rating procedures, as outlined by AASHTO is presented. This includes the allowable stress rating procedure and the load factor rating procedure. Load rating using the computer program BRASS is also discussed.

Two methods for load rating concrete slab bridges without plans were developed. The methods are referred to as the "Steel Area Method" and the "Simplified Method." Both are based on using the results of a controlled diagnostic load test of a bridge. In a typical test, the slab is instrumented with strain transducers and also perhaps displacement transducers. A loaded truck of known weight then crosses the bridge on a designated longitudinal path while the strains and displacements are continuously recorded.

The primary unknown in a slab bridge without plans is the size and amount of reinforcing steel. Once an estimate of the amount of reinforcing steel is obtained, a load rating can be calculated using the usual analytical procedures or BRASS. The objective of the steel area method is to estimate the area of steel in the slab using the results of the load test. Using this approach, strains are measured at two or more load levels in the test. A plot of load versus strain under the load is then created, the slope of which is referred to as the "strain stiffness." Using elementary beam theory, the slope of the strain-stiffness plot was derived in terms of the span length, depth of slab, distance to the neutral axis, Young's modulus of the concrete and the area of steel in the slab. Equating the measured strain-stiffness to the theoretical stiffness yields one equation for two unknown parameters, the area of steel and the distance to the neutral axis. Assuming strains are linearly distributed through the depth of the beam (a standard assumption in the analysis of concrete beams) yields a second equation for the two unknown parameters. Using these two equations the area of steel and distance to the neutral axis can be estimated. A similar procedure can be followed using the measured deflection of the slab.

For concrete bridges that are designed to be under-reinforced, the controlling design criterion in terms of bending is yielding of the steel reinforcement. The simplified method uses this fact, the load test data, cross-sectional dimensions and material properties to directly rate bridges without plans. The rating is based entirely on the observed live load strain as compared to the allowable strain for the steel. The rating equation is first expressed in terms of the allowable strain in the steel, the dead load strain and the live load strain. The allowable strain is calculated using accepted formulas and the material properties of the steel. Next, the live load strain in the steel is conservatively estimated to be equal to the concrete strain measured on the bottom surface of the deck in the test: again, assuming strains are linearly distributed through depth of the section and there is nominally 1.5 to 2 in of cover, the strain measured on the bottom of the slab will be only slightly larger than the strain in the reinforcement (i.e., a conservative estimate). Finally, an estimate of the dead load strain is obtained assuming the section is not cracked, using a gross moment of inertia. This strain is compared to the strain required to cause cracking in the slab. If the calculated dead load strain is greater than the cracking strain, the dead load strain is recomputed assuming a cracked section. The dead load strain assuming a cracked section is estimated from the ratio of the measured live load strain to the live load moment. Once the final dead load strain is obtained the load rating is computed.

A series of small scale experiments was conducted in the laboratory to test the steel area method. Three concrete beams, measuring 42 in by 4 in by 5 in, were fabricated with different amounts of reinforcing steel. The beams were loaded in three point bending; strains and deflections on the bottom of the beam were measured as the specimens were loaded to failure. Unfortunately, the observed trends in the experimental data were not consistent with elementary beam theory, as one would expect, and indicated a possible problem with the experimental test setup. Some factors that may have affected the test results were the difficulty in accuractely measuring low strains and displacements of small scale beams, improperly bonded strain transducers and placement or location of the reinforcing steel in the specimens. Nevertheless, estimates of the area of steel in each beam were made using the steel area method. Estimates obtained using measured strains were more accurate than those obtained using displacement data. Using strain data, two of the three estimated areas were within 25% of the actual area of steel, while the third differed by a factor of three. The obvious problems with the experimental data precluded making a reasonable assessment of the validity of the method based on the laboratory data.

The two methods for load rating bridges without plans were tested using the results of an actual diagnostic load test of a concrete slab bridge for which plans were available (allowing the success of these methods to be evaluated). Bridge 1-450N, located on U.S. Route 13 north of Smyrna, Delaware, was tested in Novermber, 1998, as part of another projected funded by DelDOT. The bridge is a concrete box culvert with a span length of 8 feet and a slab depth of 10 in. The culvert has very little negative moment steel reinforcement at the corners, therefore, it behaves much like a simply supported slab bridge. The slab is buried under more than two feet of overburden; cores taken at the site show that the overburden consists of an old concrete roadway and fill. The current roadway rests on top of the overburden.

The bridge was instrumented with twelve strain transducers. A three axle dump truck with a total weight of 29.6 tons was used as the load for the test. The maximum strain recorded during the test at any gage location was 15 microstrain.

The experimental data from the test was used to load rate the bridge using the two methods developed, assuming plans were not available for the bridge. Two additional factors needed to be considered in applying the methods to the actual bridge: transverse load distribution and the effective wheel load. A transverse load distribution factor of 0.11 was calculated for the bridge based on the test data. This compared to the AASHTO value of 0.22 for the bridge.

The affect of the overburden on the bridge response was seen to be significant, just from the very low strain measurements recorded during the test. With the very short span and relatively deep overburden, the wheel load is spead out and away from the slab, thereby reducing the effective load on the slab. This had to be taken into account in the load rating analyses. To investigate the effect of the overburden, the effective wheel load on the slab was calculated three different ways, (1) assuming no overburden (i.e., the full wheel load is applied directly to the slab), (2) using the AASHTO formulas for distributing the load assuming 2 feet of fill on top of the slab and (3) using a more precise Boussineq procedure for distributing the load assuming 2 feet of fill on top of the slab. The effective rear wheel load, assuming no overburden was 1233 lb; using the AASHTO formulas the wheel load was 459 lb and using the Boussineq procedure the effective wheel load on the slab was 289 lb. The results show that the overburden reduces the effective load on the slab by 60 to 80%.

The area of steel in the slab was estimated several different ways, for different assumptions regarding the overburden. The actual area of steel in the slab is 1.05 square inches per foot of width. Assuming no overburden (full wheel load applied directly to the slab), the estimated area of steel was 6.28 square inches per foot. Using the AASHTO formula and assuming 2 feet of fill, the estimated area of steel was 2.42 square inches per foot; using the Boussineq procedure and assuming 2 feet of fill the estimated area of steel was 1.53 square inches per foot. The existing concrete roadway will also tend to distribute the wheel load; therefore, the area of steel was also estimated taking the 12 in roadway into acount. Using the AASHTO formula and assuming 2 feet of fill, and that the present 12 in concrete roadway adds to the effective depth of the slab (i.e., the slab depth is 22 in), the estimated area of steel was 1.09 square inches per foot; using the Boussineq procedure with 2 feet of fill and an effective slab depth of 22 in, the estimated area of steel was 0.69 square inches per foot. The results show that the effect of the overburden and roadway can be significant in estimating the area of steel, however, when these factors are taken into consideration, the area of steel is estimated with reasonable accuracy. Load ratings based on these areas of steel would accordingly be similar to the current load rating for the bridge as though plans were available. Once the area of steel has been determined, results of the diagnostic load test could also be used to improve estimates of the distribution factor and the effect of fill on the load rating.

The simplified method was also tested using the data from bridge 1-450. The allowable strain for the reinforcing steel was 626 microstrain. The live load strain, propotioned to be equivalent for an HS20-44 truck was 10.8 microstrain and the impact factor was 0.2. The dead load strain was estimated to be equal to 43.3 microstrain. This yields a rating for the bridge of 45. The existing rating (< 1) for bridge 1-450N is much lower since it does not include the effect of fill, and better than expected load distribution. If the effect of fill is included and the measured distribution factor is used, the analytically determined rating increases to 67.

Three key conclusions and recommendation can be drawn from this investigation:

1. The steel area and the simplified methods have been developed as tools to assist in load rating of bridges for which there are no structural plans. Each is based on the results of a controlled diagnostic load test; once the test data is obtained the methods are easy to apply and are akein to design type calculations. Each is founded in the mechanics of the behavior of a reinforced concrete beam/slab. Each leads to a load rating factor for the bridge.

2. There are advantages and disadvantages to both methods. The steel area method requires information about the concrete strength and more detailed analysis, but does not require an estimate of the dead load strain in the slab. The simplified method requires very little analysis and is based almost entirely on the measured strains; however, it requires an estimate of the dead load strain in the slab. At this stage in the development of the techniques, neither is seen to be decidedly better than the other.

3. In the single test conducted, both methods provided similar results, showing that the tested structure has a very high rating, similar to the rating that would be obtained from a purely analytical procedure when the effect of the overburden is included. The effect of overburden and the concrete roadway was found to be significant and should be considered in the rating procedure; whether there are plans for the bridge or not.

4. Additional field tests need to be conducted to fully validate the methods. The ideal

candidate bridge for validating these approaches would be a recently constructed, simply supported concrete slab bridge with no overburden. Once validated, the methods should be useful tools for rating concrete slab bridges without plans.

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Chapter 1

Introduction

1.1 Overview

Currently, most bridges are evaluated using simplified models that rely on structural dimensions and properties determined from original design plans and observations made during on-site inspection [1]. However, original design plans do not exist for many older bridges in the United States. Structural properties for bridges without original design plans are not easily obtained. This is especially a problem for older concrete bridges for which the amount of reinforcing steel may be unknown. As a result, the bridge capacity or rating can not be evaluated easily using traditional methods. This can result in the bridge being posted at a very low load level. While these load limits are meant to ensure public safety, the time consumed in detouring around a posted bridge creates an inconvenience to the traveling public and a significant financial cost to the state in which the bridge is located. In addition, posted bridges are among those most likely to receive costly repairs or be replaced. Therefore, the load carrying capacity of a posted bridge should be determined accurately.

Load testing is one method for estimating the strength capacity of existing bridges without original design plans. It also helps engineers better understand the behavior of existing bridges. Load tests can help public agencies decide which bridges to post, rehabilitate, or replace, and as a result can help prevent public money from being unwisely spent on unnecessary repair. In addition, the public does not have to face the inconvenience of unnecessary construction.

1.2 Nondestructive Static Load Testing Methods

Static load tests performed on bridges can be divided into two categories: proof load tests and diagnostic load tests. These methods are nondestructive and allow the bridge to remain in service after the test is performed. The basic procedures for both methods are discussed in the following two sections. However, only the diagnostic load test was performed on the bridges discussed in this paper and therefore will be discussed in greater detail in later chapters.

1.2.1 Proof Load Test

Proof load tests are designed to directly determine the maximum live load that a bridge can safely support [2]. First, a starting proof load and a target proof load are calculated. Then, sensors are installed on the bridge in appropriate locations to monitor the strain and deformations of the structure. The bridge is then slowly and incrementally loaded (so no dynamic effects are introduced) beginning with the starting proof load. Testing personnel must carefully watch the bridge for excessive cracking, deflection or settlement. They must also check for to see that nonlinear response exceeding 10% does not occur. The test must be concluded if any of the aforementioned behaviors occur. The load should be removed immediately if the test is concluded due to excessive cracking, deflection, or settlement. The load should be removed in increments no larger than the loading increments when the test is concluded due to nonlinearity or reaching the target load. The bridge should then be inspected to check for a damaged, distressed, or displaced component [3].

1.2.2 Diagnostic Load Test

Diagnostic load tests are performed to determine the effect of a known load on various components on the structure [2]. A pre-weighed test truck is used to conduct the test. Numerous strain gages are set up at predetermined locations on the bridge superstructure. Strain measurements are recorded as the test vehicle is driven across the bridge at a crawling speed of about 3 to 5 mph to minimize any dynamic effects. Data can also be recorded during high-speed passes to examine impact effects [4].

1.3 Objective of Study

The purpose of this thesis is to develop methods for determining the load carrying capacity of bridges without original design plans, specifically concrete slab bridges, using diagnostic load testing. Two methods are developed. The first method presented uses strain measurements from the field test, combined with basic mechanics principles to estimate the unknown area of reinforcing steel. Traditional load rating methods can be used once the reinforcing steel area has been estimated. The second method presented uses strain measurements from the field test combined with known material properties to directly determine the bridge load rating.

In developing methods for evaluating the load carrying capacity of bridges without plans, the first step taken was to evaluate the inventory of planless bridges in Delaware. Chapter 2 presents the status and characteristics Delaware's planless bridges. The second step was to review how bridge load carrying capacities are traditionally computed. In Chapter 3, traditional bridge load rating categories as well as current load rating methods are presented. The third step was to develop methods for determining the load carrying capacity for planless concrete bridges based on field test results. The two methods that were developed are presented in Chapter 4. The fourth step was to validate one of the methods for determining the load carrying capacity of planless bridges with an experimental lab study. This study is detailed in Chapter 5. The final step was to validate both methods by conducting an actual bridge load test and using the test results to evaluate the bridge's load carrying capacity. Chapter 6 presents the details of an actual bridge load test and how the methods developed can be used to evaluate the bridge load carrying capacity. Chapter 7 contains the conclusions of the research.

Chapter 2

Description of Bridge Types without Original Design Plans in Delaware

2.1 Introduction

This chapter describes the status and characteristics of planless bridges in Delaware. The 1997 Delaware Bridge Inventory was used to determine both the bridge design and material type of Delaware's planless bridges. The Bridge Inventory was then used to determine the characteristics and operational status of the most common Delaware design types. Delaware Department of Transportation (DelDOT) inspection reports were also used as a source for additional information. All of this information was used to identify the most common bridge types for which original design plans do not exist, and to guide the development of appropriate methods for load rating.

2.2 Description

Most bridges are currently evaluated and load rated using simplified models that rely on structural dimensions and properties determined from original design plans and observations made during on-site inspections. However, original design plans do not exist for many bridges in the United States. The lack of original design plans makes it difficult to determine the structural properties of these bridges. This is especially true for concrete slab bridges and concrete culverts since, after construction, the area of reinforcing steel becomes unknown.

The 1997 Delaware Bridge Inventory was used to determine how many bridges in the state of Delaware have no original design plans, and to identify the design type, date of construction and materials of construction of these bridges. The information is presented in Tables 2.1 and 2.2. The search uncovered 189 bridges that have no original design plans. The bridges identified were of the following types: stringer/multi-girder, girder and floorbeam system, tee beam, box beam or girder, frame, truss-thru, arch-deck, culvert, arch-thru, slab and movable-swing. The bridges without plans were constructed of the following materials: concrete, steel, prestressed concrete, timber, masonry, aluminum and wrought or cast iron. Concrete and steel were the most common bridge materials. The most common design types were slabs, culverts, arch-decks and stringer/multi-beams. In addition, many of Delaware's planless bridges have unknown dates of construction.

The inventory was then searched for more detailed information on the most common types of bridges without plans: slabs, culverts, arch-decks and stringer/multi-beams. The following information was collected: year built, design live load, operational status, material type, number of spans in the main unit, maximum span length, structure length, operational rating gross load, inventory rating gross load and whether or not secondary plans existed. The information collected was divided into sub-topics. The percentage of bridges for a given design type was computed for each sub-topic and is presented in Tables 2.3(a) and 2.3(b).

The following conclusions can be made about Delaware's planless bridges. The majority of Delaware's planless bridges:

- Have unknown dates of construction or were built before 1940,

- have an unknown design live load or a design live load of HS20,
- are open with no restriction,
- are constructed of steel or concrete,
- are simple spans with a maximum span length of less than 50 feet and a structure length of less than 80 feet,
- have an operating rating gross load of less than 40 tons and an inventory rating gross load of less than 30 tons for all types except culverts which are less than 40 tons,
- and, half of the planless slab and arch-deck bridges have secondary plans while a majority of the culverts and multi-beam/stringer bridges have no secondary plans.

The subset of bridges without plans that are posted were then examined to determine of these bridges had any unique characteristics. The following information was obtained on the most common design types: year built, design live load, material type, number of spans in the main unit, maximum span length, structure length, operational rating gross load, inventory rating gross load and whether or not secondary plans existed. The information collected was divided into sub-topics. The percentage of bridges for a given design type was computed for each sub-topic and summarized in Tables 2.4(a) and 2.4(b).

The following conclusions can be made about Delaware's posted planless bridges. The majority of Delaware's posted planless bridges:

- have unknown dates of construction or were built before 1930,
- have unknown design live loads,
- are constructed of concrete,
- have a simple span with a maximum span length of less than 50 feet and a structure length of less than 90 feet,
- and, an operating load rating of between 30 and 40 tons and inventory load rating of less than 20 tons,

The results of this search indicate that Delaware has a significant numbers of bridges posted at a very low load level. In addition, Delaware has a significant number of bridges with no original design plans. This means that there is a critical need for field testing methods to load rate these bridges. The rest of this thesis focuses on concrete bridges without original design plans. This was done because concrete bridges are especially difficult to rate due to the unknown reinforcing steel area. In addition, the Delaware Bridge Inventory indicates that concrete slab bridges and concrete culverts are the most common planless bridge types in Delaware

2.3 Tables

Table 2.1: Breakdown of Delaware's Bridges without Original Design Plans by year built and material

Year Built	No. of bridges of a given type of material							
	Concrete	Steel	Prestressed Concrete	Timber	Masonry	Aluminum Wrought Iron or Cast Iron	Concrete Continuous	Steel Continuous
Unknown	59	15	0	2	6	1	1	0
1901-1910	1	1	0	0	0	0	0	1
1911-1920	6	4	0	1	1	0	0	1
1921-1930	7	3	0	5	0	0	0	0
1931-1940	5	3	0	2	0	0	0	0
1941-1950	1	0	0	0	0	0	0	0
1951-1960	1	2	0	0	0	0	0	0
1961-1970	3	12	0	0	0	1	0	0
1971-1980	0	18	0	0	0	7	0	0
1981-1990	0	9	1	1	0	0	0	0
1991-1998	4	3	1	0	0	0	0	0
Total No. of bridges	87	70	2	11	7	9	1	2

Table 2.2: Breakdown of Delaware's bridges without Original Design Plans by year built and type

Year Built	No. of bridges of a given type of design and/or construction										
	Slab	Stringer/Multi-beam or Girder	Girder and Floorbeam System	Tee Beam	Box beam or Girders - Multiple	Frame	Truss-Thru	Arch-Deck	Culvert	Arch-Thru	Movable-swing
Unknown	17	7	0	0	0	0	3	12	44	1	0
1901-1910	0	0	2	0	0	0	0	1	0	0	0
1911-1920	1	2	3	1	0	0	0	4	1	0	1
1921-1930	1	6	0	0	0	0	1	2	5	0	0
1931-1940	1	5	0	0	0	1	0	0	3	0	0
1941-1950	0	0	0	0	0	0	0	0	1	0	0
1951-1960	0	2	0	0	0	0	0	0	1	0	0
1961-1970	0	0	0	0	0	0	0	0	16	0	0
1971-1980	0	1	0	0	0	0	0	0	24	0	0
1981-1990	1	0	0	0	1	0	0	0	9	0	0
1991-1998	0	0	0	0	1	0	0	1	6	0	0
Total No. of bridges	21	23	5	1	2	1	4	20	110	1	1

Table 2.3a: Characteristics of most common bridges without plans (by percentage of the total number of each type)

	Slab	Culvert	Arch-deck	Multi-beam/Stringer
Total # of Bridges	21	110	20	23
Year Built				
unknown	81.8	40	60	27.3
1901-1910	0	0	5	0
1911-1920	4.55	0.9	20	9.1
1921-1930	4.55	4.6	10	27.3
1931-1940	4.55	2.7	0	22.7
1941-1950	0	0.9	0	0
1951-1960	0	0.9	0	9.1
1961-1970	0	14.5	0	0
1971-1980	0	21.8	0	4.5
1981-1990	4.55	8.2	0	0
1991-2000	0	5.5	5	0
Design Live Load				
unknown	81.8	51.8	95	72.7
HS20	13.6	43.6	0	13.6
HS15	4.5	0.9	0	0
H20	0	3.6	0	0
HS25	0	0	0	0
H15	0	0	5	13.6
Operational Status				
Open, no restriction	68.2	89.1	85	63.6
Posted for load-carrying capacity	31.8	9.1	15	36.4
Posted for other reasons	0	1.8	0	0
Material Type				
Concrete	90.9	45.5	75	0
Steel	0	43.6	0	63.6
Concrete Continuous	0	0.9	0	0
Timber	9.1	0	0	36.4
Masonry	0	1.8	25	0
Aluminum/cast iron/wrought iron	0	8.2	0	0
Number of spans in the main unit				
1	100	49.1	90	90.9
2	0	16.4	5	0
3	0	28.2	5	9.1
4	0	6.4	0	0
Maximum span length (ft)				
0-10	4.5	70	30	27.3
11-20	45.5	29.1	35	13.6
21-30	18.2	0.9	10	22.7
31-40	13.6	0	5	9.1
41-50	4.5	0	5	18.2
51-60	4.5	0	5	0
61-70	4.5	0	0	4.5
71-80	4.5	0	5	0
81-90	0	0	5	4.5
91-100	0	0	0	0

Table 2.3b: Characteristics of most common bridges without plans (by percentage of the total number of each type)

	Slab	Culvert	Arch-deck	Multi-beam/Stringer
Total # of Bridges	21	110	20	23
Structure length (ft)				
1-10	9.1	25.5	15	31.8
11-20	77.3	34.5	30	18.2
21-30	13.6	31.8	20	9.1
31-40	0	3.6	0	9.1
41-50	0	3.6	10	9.1
51-60	0	0.9	10	4.5
61-70	0	0	5	4.5
71-80	0	0	5	4.5
81-90	0	0	5	0
91-100	0	0	0	0
101-110	0	0	0	0
111-120	0	0	0	0
121-130	0	0	0	0
131-140	0	0	0	4.5
141-150	0	0	0	4.5
Operating rating gross load (lb)				
10-20	4.5	2.7	0	9.1
21-30	22.7	4.5	5	13.6
31-40	45.5	82.7	75	31.8
41-50	4.5	1.8	0	18.2
51-60	4.5	1.8	0	4.5
61-70	9.1	0	0	9.1
71-80	0	1.8	5	4.5
81-90	4.5	1.8	0	4.5
91-100	4.5	4.5	10	4.5
Inventory rating gross load (lb)				
10-20	72.7	37.2	70	50
21-30	4.5	0.9	0	4.5
31-40	9.1	55.5	15	36.4
41-50	4.5	0.9	5	4.5
51-60	0	0.9	0	0
61-70	9.1	0	0	4.5
71-80	0	2.7	0	0
81-90	0	0.9	0	0
91-100	0	0.9	10	0
Do secondary plans exist?				
yes	50	20.9	50	36.4
no	50	79.1	50	63.6

Table 2.4a: Characteristics of most common posted bridges without plans (by percentage of the total number of each type)

	Slab	Culvert	Arch-deck	Multi-beam/Stringer
Total # of Bridges	7	12	3	8
Year Built				
unknown	100	91.7	33.3	25
1901-1910	0	0	33.3	0
1911-1920	0	0	0	12.5
1921-1930	0	8.3	33.3	25
1931-1940	0	0	0	25
1941-1950	0	0	0	0
1951-1960	0	0	0	12.5
1961-1970	0	0	0	0
1971-1980	0	0	0	0
1981-1990	0	0	0	0
1991-2000	0	0	0	0
Design Live Load				
unknown	85.7	91.7	100	62.5
HS20	0	0	0	0
HS15	14.3	0	0	0
H20	0	8.3	0	0
HS25	0	0	0	0
H15	0	0	0	37.5
Material Type				
Concrete	85.7	91.7	66.7	0
Steel	0	8.3	0	37.5
Concrete Continuous	0	0	0	0
Timber	14.3	0	0	62.5
Masonry	0	0	33.3	0
Aluminum/cast iron/wrought iron	0	0	0	0
Number of spans in the main unit				
1	100	100	100	87.5
2	0	0	0	0
3	0	0	0	12.5
4	0	0	0	0
Maximum span length (ft)				
0-10	42.9	83.3	33.3	37.5
11-20	57.1	16.7	33.3	12.5
21-30	0	0	0	25
31-40	0	0	0	0
41-50	0	0	0	25
51-60	0	0	0	0
61-70	0	0	0	0
71-80	0	0	0	0
81-90	0	0	33.3	0
91-100	0	0	0	0

Table 2.4 b: Characteristics of most common posted bridges without plans (by percentage of the total number of each type)

	Slab	Culvert	Arch-deck	Multi-beam/Stringer
Total # of Bridges	7	12	3	8
Structure length (ft)				
1-10	28.6	50	33.3	25
11-20	57.1	50	33.3	25
21-30	14.3	0	0	0
31-40	0	0	0	12.5
41-50	0	0	0	0
51-60	0	0	0	12.5
61-70	0	0	0	12.5
71-80	0	0	0	0
81-90	0	0	33.3	0
91-100	0	0	0	0
101-110	0	0	0	0
111-120	0	0	0	0
121-130	0	0	0	0
131-140	0	0	0	0
141-150	0	0	0	12.5
Operating rating gross load (tons)				
10-20	14.3	25	0	25
21-30	71.4	25	33.3	37.5
31-40	0	25	66.7	0
41-50	0	8.3	0	25
51-60	14.3	0	0	0
61-70	0	0	0	12.5
71-80	0	0	0	0
81-90	0	0	0	0
91-100	0	16.7	0	0
Inventory rating gross load (tons)				
0-10	28.6	25	33.3	25
11-20	42.9	58.3	66.7	62.5
21-30	14.3	0	0	0
31-40	14.3	0	0	12.5
41-50	0	0	0	0
51-60	0	0	0	0
61-70	0	0	0	0
71-80	0	16.7	0	0
81-90	0	0	0	0
91-100	0	0	0	0
Do secondary plans exist?				
yes	57.1	58.3	33.3	50
no	42.9	41.7	66.7	50

Chapter 3

Bridge Load Rating

3.1 Introduction

This chapter discusses how a bridge's load carrying capacity is traditionally calculated assuming detailed structural drawings are available. Reviewing these procedures will help in the understanding of how the different load rating methods work, as well as the meaning of each load rating category. This chapter first includes a discussion of three traditional load rating categories: Inventory Rating, Operating Rating, and Posting Rating. Next, the two traditional load rating methods: the Allowable Stress Rating method, and the Load Factor Rating method are presented. The last topic discussed in Chapter 3 is a general overview of the BRASS computer program, which is used by DelDOT to rate bridges.

3.2 Bridge Load Rating Categories

Bridge load rating calculations provide a basis for determining a bridge's safe load capacity. Three different load ratings are assigned to a bridge based on three different types of load intensities. They are the inventory, operating and posting rating.

Bridges are rated according to the weight of standard trucks. Each standard truck has a different axle loading and configuration. The American Association of State Highway and Transportation Officials (AASHTO) uses a standard truck called an HS20-44. This vehicle is a three-axle tractor-trailer with the first two axles spaced at 14 feet. The spacing between the trailer axle and the rear axle is varied from 14 feet to 30 feet in order to determine which spacing gives the worst loading case in the analysis. The total vehicle weight is 72 kips with 8 kips on the front axle and 32 kips each on the trailer and rear truck axle [5]. Delaware uses seven standard trucks, including the HS 20-44. The ratings will vary for each standard truck since each truck loads the bridge differently. Delaware uses the lowest rating to determine the load-carrying capacity of a given bridge.

Any bridge component can be load rated. However, it is generally assumed that the bending moment or shear in the girders will give the critical rating. As a result, the other bridge components are not typically rated [2].

3.2.1 Inventory Rating

The inventory rating captures the lower range of bridge performance. It represents the load level that can safely utilize an existing bridge for an infinite period of time [5]. In simple terms, this rating indicates the bridge's performance under the loading of a large quantity of everyday traffic.

3.2.2 Operating Rating

The operating rating captures the upper range of bridge performance. Occasionally, a bridge may need to handle an abnormally large live load. The bridge's life span would be shortened if that load were to repeatedly pass over the bridge. However, one needs to determine if the bridge can carry an abnormally large load infrequently. Therefore, the operating rating represents the absolute maximum permissible load level to which the structure may be subjected [5].

3.2.3 Posting Rating

The posting rating is used for legal purposes in each state. Each state has a legal load limit for each axle configuration for the predominant trucks that operate in that state. If a truck is overloaded for its axle configuration when it pulls into a weigh station, it is ticketed and impounded until it is unloaded [4].

Each state determines how it wants to compute its posting ratings. For example, Delaware computes its posting rating by adding two-thirds of the inventory rating to one-third of the operating rating for the allowable stress design rating procedure. If a bridge has a posting rating lower than one, the allowable weight of that truck is posted on the bridge so no trucks will overstress the bridge. A rating of less than one means that the bridge condition has deteriorated to the point that it cannot handle even the legal limit [4].

3.3 Bridge Load Rating Methods

There are currently two methods for load rating a bridge. They are the Allowable Stress Rating method and the Load Factor Rating method.

3.3.1 Allowable Stress Rating Method

The allowable stress rating utilizes 55% of the yield stress for the critical member to determine the inventory rating. The operating rating is obtained by utilizing 75% of the yield stress for the critical member. To calculate the rating factor (RF), the maximum moment, or capacity, is determined from the allowable stress for the inventory or operating limits. The dead load moment is then determined and subtracted from the calculated capacity. What remains is the live load capacity. The live load capacity is then divided by the maximum live load moment with the effect of impact included. The impact is based upon the bridge span length. The allowable stress rating equation is given by:

$$RF = \frac{Capacity - Deadload}{LiveLoad(1+I)} \tag{3.1}$$

A rating factor greater than one implies that the capacity of the structure is sufficient to carry the prescribed dead and live loads, with the appropriate factors of safety. A rating less than one implies that it cannot and therefore, loads on the bridge must be restricted. Once the two rating factors are determined, one multiplies the loading vehicle by the rating factor to obtain the inventory and operating ratings [4].

3.3.2 Load Factor Rating Method

The load factor rating method applies a factor to the dead load and the live load in the rating equation. It is assumed that the structure can reach the ultimate capacity given in the design guidelines for the material in question. The load factor method is the same as the allowable stress method after this step [4]. The load factor inventory rating factor equation is:

$$RF = \frac{Capacity - 1.3(Deadload)}{1.3(5/3)(Liveload)(1+I)} \qquad (3.2)$$

The load factor operating rating factor equation is:

$$RF = \frac{Capacity - 1.3(Deadload)}{1.3(Liveload)(1+I)} \qquad (3.3)$$

3.4 BRASS

BRASS stands for Bridge Rating and Analysis of Structural Systems. The Wyoming Department of Transportation created the BRASS program with sponsorship provided by the Federal Highway Administration [7]. The main purpose of BRASS is to compute inventory, operating and, posting ratings according to the

Manual for Maintenance Inspection of Bridges [5]. This bridge rating program is used by DelDOT to rate their bridges.

BRASS allows a wide variety of structures and materials to be analyzed. BRASS can rate the following bridge structures: continuous beam bridges, rigid-frame bridges, slant leg bridges and rigid-frame box culverts. BRASS can rate bridges constructed out of the following materials: reinforced concrete, prestressed concrete, structural steel or timber. BRASS can also design bridge decks and girder sections as well as perform a structural bridge analysis. The methods of analysis used vary according to the type of structure. A column analogy is used for beam type structures. Cell structures and slant leg frames are analyzed by the slope deflection method. Deflections are calculated using virtual work. BRASS uses a Gauss-Jordan method to invert the resulting matrices used to solve the governing equation [4]. Details on how to use BRASS for bridge rating are given in Appendix C.

Chapter 4

Two Methods for Evaluating the Load Carrying Capacity of Concrete Bridges without Plans using Load Test Data

4.1 Introduction

The previous chapter discussed traditional methods for evaluating bridge load carrying capacity where structural drawings are available. This chapter discusses the development of two methods for evaluating the load carrying capacity of planless concrete bridges. The first method, referred to as the steel area method, uses strain measurements from field testing combined with basic mechanics principles to estimate the unknown area of reinforcing steel in a concrete slab bridge. The estimated reinforcing steel area can then be used with the traditional rating techniques described in Chapter 3. For example, if BRASS is used to rate the bridge, the estimated area can be included in the BRASS command file, and the program will then have sufficient information to estimate the load carrying capacity of the planless bridge. The second method, referred to as the simplified method, uses strain measurements from field testing combined with known material properties to directly determine the load carrying capacity of a planless bridge.

The two methods developed assume that the bridge has not been loaded above the allowable stress limit during the test. Furthermore, the methods assume that the slab can be analyzed as a beam of unit width. In addition, the following fundamental assumptions were used to develop both methods [8]:

1. Plane sections remain plane
2. No slip between the longitudinal reinforcing steel and the concrete
3. Materials are linear elastic

In the process of determining load carrying capacity, the following additional information can also obtained from the strain data: the lateral distribution factor, effective wheel load (reduced due to the effect of load distribution including the effect of fill) and slab stiffness.

The steel area method is developed in Section 4.2.1 and application of the steel area method is presented in Section 4.2.2. The development and application of the simplified method is presented in Section 4.2.3.

4.2 Bridge Capacity Evaluation Methods

4.2.1 Development of the Steel Area Method

The steel area method uses load test data combined with basic mechanics principles to estimate the unknown reinforcing steel area of a planless concrete bridge. Then, a program such as BRASS can be used to rate the bridge. The method assumes that the bridge was not loaded above the allowable stress limit during field testing, and that the fundamental assumptions presented in Section 4.1 are true. The steel area method can be used to load rate both concrete beam bridges and concrete slab bridges. Procedures have been developed to estimate the amount of reinforcing steel from measured strains or measured displacements. The derivation of the equations used to estimate the unknown reinforcing steel area using strain data is presented in Section 4.2.1.1. The derivation of the equations used to estimate the unknown reinforcing steel area using displacement data is presented in Section 4.2.1.2.

4.2.1.1 Estimating Steel Area using Strain Data

We begin by considering a simply supported rectangular beam of width b and depth h that is loaded at midspan with a concentrated force P. Furthermore, we assume that as the beam is loaded the strain on the bottom surface of the beam (ε_{ct}), directly beneath the load is measured. The assumed stress and strain distributions across the cross-section for strains in the allowable load region are shown in Figure 4.1 [8]. In addition, the beam cross-sectional geometry is shown in Figure 4.1 [8]. The variables needed to derive the equations used to estimate the reinforcing steel area are defined below:

d = The distance between the top of the beam and the reinforcing steel

b = The beam width

h = The beam depth

x = The distance from the top fiber of the beam and the neutral axis

L = The beam span length

P = The concentrated load

E_c = The Young's modulus of concrete

E_s = The Young's modulus of the reinforcing steel

A_s = The reinforcing steel area

$$n = \frac{E_s}{E_c}$$

M_n = The beam bending moment

f_s = The steel rebar stress

f_c = The concrete compression stress at the outer fiber

ε_s = The steel rebar strain

ε_c = The concrete compression strain at the outer fiber

ε_{ct} = The tensile strain at the outer fiber of the beam

The equations needed to estimate the reinforcing steel area using strain data are derived in the following manner. In this derivation, it is assumed that the tensile concrete has cracked. First, the resultant internal compressive force is obtained by the integration of stress times the area upon which it acts. Thus, the internal compressive force within the concrete is the volume of a triangular wedge:

$$c = 0.5 f_c bx = 0.5 E_c \varepsilon_c bx \tag{4.1}$$

The resultant tensile force in the reinforcing steel is:

$$T = f_s A_s = E_s \varepsilon_s A_s \tag{4.2}$$

Equating C to T gives:

$$\frac{f_s}{f_c} = \frac{bx}{2A_s} \tag{4.3}$$

The ratio of f_s to f_c may also be obtained from the linear strain relationship and assuming that stress is proportional to strain.

$$\frac{\varepsilon_s}{\varepsilon_c} = \frac{d-x}{x} \tag{4.4}$$

$$\frac{f_s}{f_c} = \frac{E_s \varepsilon_s}{E_C \varepsilon_C} = n\frac{d-x}{x} \tag{4.5}$$

Equating (4.3) and (4.5) yields the following expression for the distance to the neutral axis:

$$\frac{bx^2}{2} = nA_s(d-x) \tag{4.6}$$

The steel rebar strain (ε_s) can be related to the strain at the bottom fiber of the beam (ε_{ct}) using the linear strain relationship.

$$\varepsilon_s = \varepsilon_{ct}\left[\frac{h-x}{d-x}\right] \tag{4.7}$$

The internal moment for the concrete beam is given by.

$$M_n = A_s E_s \varepsilon_s (d - \frac{x}{3}) = A_s E_s \varepsilon_{ct}\left[\frac{h-x}{d-x}\right](d-\frac{x}{3}) \tag{4.8}$$

The moment at midspan for a simply supported beam loaded at the midspan is given by

$$M_n = \frac{PL}{4} \tag{4.9}$$

Substituting (4.9) into (4.8) yields

$$\frac{PL}{4} = A_s E_s \varepsilon_{ct} \left[\frac{h-x}{d-x}\right]\left(d - \frac{x}{3}\right) \qquad (4.10\text{ a})$$

Rearranging equation (4.10) a yields

$$k_{\varepsilon_t} = \frac{P}{\varepsilon_{ct}} = \frac{4A_s E_s \left[\frac{h-x}{d-x}\right](d - \frac{x}{3})}{L} \qquad (4.10\text{ b})$$

Equation (4.10) defines the slope of the load versus strain curve for the beam and will be referred to as the "strain stiffness" of the beam. Note that it is a function of L, h, d, x, E_s and the area of reinforcing steel, A_s. When the load versus strain plot is generated from experimental data, the strain stiffness can be determined from the curve. With the strain stiffness known, Equations (4.6) and (4.10 b) can be solved simultaneously for the unknowns x and A_s. Note that if the depth to the reinforcing steel (d) is unknown, there are actually three unknowns in the two equations. In that case, one can assume a typical cover depth, for example 1.5 inches (d = h – 1.5).

4.2.1.2 Estimating Steel Area using Displacement Data

The same beam and load conditions used in Section 4.2.1.1 will be used again here. Now it is assumed that the deflection of the beam directly underneath the load is measured. The equations used to estimate the reinforcing steel area using displacement data are derived below.

The expression for the distance to the neutral axis was derived in the last section. Assuming the section is cracked under its own weight, the moment of inertia for the cracked beam is

$$I_{cr} = \frac{1}{3}bx^3 + nA_s(d-x)^2 \tag{4.11}$$

The deflection equation for a concrete beam simply supported with a concentrated load at midspan is given by

$$\delta = \frac{PL^3}{48E_cI_{cr}} \tag{4.12}$$

Rearranging (4.12), the stiffness of the beam can be expressed as

$$k_{E_\delta} = \frac{P}{\delta} = \frac{48E_cI_{cr}}{L^3} \tag{4.13}$$

Equation (4.13) defines the slope of the load verses displacement curve for the beam and will be referred to as the " displacement stiffness" of the beam. Note that it is a function of L, I_{cr}, E_c, x, n, d, b and the reinforcing steel area A_s. With the displacement stiffness known, one can solve Equations (4.6) and (4.13) simultaneously for the unknowns x and A_s.

4.2.2 Application of the Steel Area Method for Rating Concrete Bridges without Plans

A reinforced concrete slab bridge behaves much like a two-dimensional flat plate structure. However, under many circumstances, the slab can be designed as a beam with unit width. This is, in fact, the procedure routinely used by bridge engineers to design flat slab bridges. To make the simplification from a two-dimensional plate to a one-dimensional beam, the transverse load distribution factor is required. The distribution factor represents the portion of a wheel load carried by a unit width of the slab. Another factor that must be considered in the design is the effect of overburden or fill that might be present on top of the slab. Fill tends to distribute a load applied at the roadway level in all directions, thereby reducing the magnitude of the load that is actually applied to the slab. In fact, according to the AASHTO Specifications [11], the effect of live load can be ignored under certain circumstances for structures with substantial fill. Since the bridges tested and used later in this thesis had significant amounts of fill, in applying the steel area method to the concrete slab bridge, the effect of load distribution and fill must be considered in the procedure. In this section, the effect of these factors is presented, and the procedure for estimating the area of reinforcing steel in a slab bridge from test data is discussed.

4.2.2.1 Transverse Load Distribution Factor

When a concentrated load is applied to a reinforced concrete slab, the load is distributed over an area larger than the actual contact area. The load is distributed in the spanwise direction, and in the orthogonal direction, i.e., transverse or normal to the roadway direction. The transverse load distribution factor (DF) is a measure of load distribution in the slab in the transverse direction. It can also be thought of as the fraction of the wheel load that is carried by a one-foot width of slab. The distribution factor can be estimated using equations defined by AASHTO, or using measured strains from a field test. Using measured strains, the distribution factor for concrete slab bridges can be calculated from a series of longitudinal strains measured along a transverse line of the slab (preferably at mid-span) using the following procedure:

1. Determine the time at which the absolute maximum strain value occurs in the slab for the truck pass. This can be done in the slab for a truck pass using the strain data computer program given in Appendix B.
2. Create a plot of the longitudinal strain along a transverse line at the time of maximum strain.
3. Calculate the area under the curve created in step 3.
4. Calculate the effective width of the slab by dividing the area calculated in step four by the maximum strain value along the line.
5. The transverse load distribution factor is calculated as the inverse of the beams' effective width:

$$DF = \frac{1}{effective \quad beam \quad width} \tag{4.14}$$

4.2.2.2 Effective Wheel Load

The effective wheel load is required for both the steel area and simplified methods. The effective wheel load can be determined by multiplying the actual wheel load by the transverse distribution factor, when no fill exists between the roadway and the bridge; however, when fill is present the load will be distributed longitudinally and transversely over the entire span, thus reducing the magnitude of the effective wheel load on a beam of unit width. The resulting stress distribution on the slab or beam, when fill is present is shown in Figure 4.2. To account for the effect of fill, two methods were used to calculate the effective wheel load when fill exists, the Boussinesq Method and the AASHTO method.

4.2.2.2.1 The Boussinesq Method

This method uses the Boussinesq equation [9] to calculate the pressure distribution on the slab due to the wheel load. The moment at mid-span due to this pressure distribution is then determined using statics. The effective wheel load that would produce the same moment is then calculated from the distributed load moment. The procedure is as follows:

1. Multiply the actual wheel load by the transverse distribution factor (DF).
2. Calculate the pressure due to the wheel load determined in step one, at several points along the span, using Boussinesq's equation for a point load [9]:

$$\sigma_z = \frac{Q}{z^2} N_B \tag{4.15}$$

where: Q = The wheel load determined in step one

z = The fill depth

$$N_B = \frac{(3/2\pi)}{[(r/z)^2 + 1]^{5/2}}$$

$$r = \sqrt{x^2 + y^2}$$

x = Distance from the wheel load

y = 0.00

3. Determine the moment at mid-span of the bridge due to the distributed load calculated in step 2.
4. Calculate the effective wheel load that would produce the same moment as calculated in step three, using the relation

$$P_{eq} = \frac{4M}{L} \tag{4.16}$$

where: M = Moment at mid-span caused by the stress distribution

L = The bridge span

4.2.2.2.2 AASHTO Method

This method uses AASHTO Specification 3.6.1.2.6 to determine the effective wheel load when fill is present between the roadway and the bridge [11]. The procedure is as follows:

1. Multiply the actual wheel load by the transverse distribution factor (DF).
2. Determine the equivalent distributed load acting on the span in accordance with the AASHTO LRFD Bridge Design Specifications and Commentary Manual [11]:
    a. The effect of the fill on the live load distribution can be neglected when fill depth is less than 2 feet.
    b. When the fill depth exceeds 2 feet, the wheel load is uniformly distributed over a square area with sides equal to 1.5 times the fill depth in select granular fill and the fill depth in all other cases.
    c. The effect of the live load may be neglected for single-span culverts when the fill depth is more than 8 feet and exceeds the span length. The effect of the live load may be neglected for multiple span culverts when the fill depth exceeds the distance between the faces of the end walls.

3. Calculate the moment at mid-span due to the distributed load calculated in step two.
4. Calculate the effective wheel load that would produce the same moment as calculated in step three using equation (4.16).

4.2.2.4 Reinforcing Steel Area

The steel area method uses the following procedure to rate concrete beam bridges and concrete slab bridges without original design plans:

1. Calculate the transverse load distribution factor as described in Section 4.2.2.1.
2. Calculate the effective wheel load, including the effect of fill if necessary, as described in Section 4.2.2.2.
3. Plot the effective wheel load versus strain or effective wheel load versus deflection.
4. Calculate the beam/slab strain stiffness or the beam/slab defection stiffness from the plot created in Step 3.
5. Estimate the reinforcing steel area (per unit width of slab) using Equations (4.6) and (4.10 b) or Equations (4.6) and (4.13).
6. Enter the following information into the BRASS command file: the transverse distribution factor, bridge type, reinforcing steel area, impact factor, dead load, truck load, the span description, cross-sectional dimensions, end conditions, material properties, traffic direction, the type of bridge load rating, the location where the bridge is to be rated, the factors used for the load rating, and whether or not shear is being considered.

7. Run the BRASS command file and load rate either the concrete beam bridge or concrete slab bridge.

4.2.3 The Simplified Method

The simplified method uses load test data, cross-sectional dimensions and material properties to directly rate bridges without design plans. The advantage of this method is that one does not have to estimate the reinforcing steel area or the live load distribution factor. Instead, the rating is based on observed live load strain as compared to total allowable strain values. The allowable stress formula used by the simplified method for load rating bridges is both simple and easy to use. It is important to note that this method is only an approximate technique. It is not intended to result in a precise rating, but rather, to give a bridge engineer a simple method for determining valuable information that can be used to re-evaluate low posting levels. Since the method is based on field test data, the computed ratings can be quite informative.

In terms of strain, one can evaluate the load rating of a bridge component using the following allowable stress equation

$$RF = \frac{\varepsilon_{all} - \varepsilon_{DL}}{\varepsilon_{LL}(1+I)} \tag{4.18}$$

where RF = Rating Factor

ε_{all} = Allowable Strain

ε_{DL} = Dead Load Strain

ε_{LL} = Live Load Strain

I = Impact Factor

and I is given by

$$I = \frac{50}{L+125} \leq 0.30 \tag{4.19}$$

To use this formula, values for ε_{all}, ε_{DL} and ε_{LL} are needed. For concrete bridges that are designed to be under-reinforced, the controlling design criterion in terms of bending is yielding of the steel reinforcement. Therefore, in terms of allowable values, the maximum allowable service load strain, ε_{all} is taken to be:

$$\varepsilon_{all} = \frac{0.55 f_y}{E_s} \tag{4.20}$$

where f_y = Assumed Reinforcing Steel Yield Stress

E_s = Young's Modulus of the Reinforcing Steel

This value of ε_{all} will generally be known based on the age of the bridge and the grade of reinforcing steel common for that era. One can see that the only additional information needed for the bridge rating are the values of ε_{DL} and ε_{LL} (I is known from Equation 4.19). From the load test, one can conservatively estimate the value of ε_{LL} in the reinforcing steel for the tested load level. The value of ε_{LL} can conservatively be taken as the maximum strain measured on the bottom face of the concrete during the load test. In actuality, the strain at the level of the steel will be somewhat less, as indicated in Figure 4.3. However, since the distance from the steel to the bottom fiber of the beam is relatively small, this should give a good estimate for the live load strain in the steel. It should be noted that the ε_{LL} will not necessarily match theoretical calculations since it is effected by many factors that are not typically included in analysis. For example, the effect of complex boundary conditions, better than assumed load distribution, and the effect of fill. In general, the measured ε_{LL} will be less than theoretically predicted, but will in fact be a more accurate measure of the in-situ behavior of the bridge to live loads.

At this point, only the dead load strain ε_{DL} in the steel needs to be estimated in order to compute the load rating factor. Unfortunately, the dead load strain in the reinforcing steel can not be measured during the load test. There is no precise method for estimating the dead load steel strain. This is further complicated by the fact that under dead load, and even live load, the concrete in tension may or may not have cracked. Therefore, some common sense and reasoning must be applied to get a conservative value. We start by considering the following general equation for linearly distributed bending stresses in the beam such as those shown in Figure 4.1

$$\varepsilon_{DL} = \frac{M_{DL}y}{EI} \tag{4.21}$$

where y = Distance to location of desired strain

M_{DL} = Dead load moment at section where strain is being calculated

E = Young's Modulus

I = Beam moment of inertia

Since we do not know whether or not the concrete cracks in tension under the dead load, we begin by assuming that it does not crack. We will seek to find the tensile strain ε_{DL} in the beam outer fiber, and as before, use this as a conservative value for the nearby strain in the steel ε_{DL}. Having assumed that the concrete has not cracked, we take $I = I_g$, $E = E_c$ and $y = h/2$. We can do this since the unknown steel area has little effect on the values of I and y until after cracking. Furthermore, both $I = I_g$ and $y = h/2$ will lead towards a conservative estimate of ε_{DL}. We can now rewrite Equation 4.21 as

$$\varepsilon_{DL} = \frac{M_{DL}(h/2)}{E_c I_g} \tag{4.22}$$

which simplifies to

$$\varepsilon_{DL} = \frac{6M_{DL}}{E_c b h^2} \tag{4.23}$$

where $$E_c = 57000\sqrt{f'c} \tag{4.24}$$

and f'c is the concrete compressive strength in psi

To compute ε_{DL}, all that is now needed is M_{DL}. This can be computed from simple statics knowing the boundary conditions for the span and the distributed load on the beam (including the weight of fill). Once ε_{DL} is estimated, one should check to see if it is indeed less than the cracking strain as assumed. The cracking stress for concrete is taken as

$$f_r = 7.5\sqrt{f'c} \tag{4.25}$$

Since concrete behaves linear-elastically to cracking, the cracking strain is

$$\varepsilon_r = \frac{7.5\sqrt{f'c}}{E_c} = 132\mu\varepsilon \tag{4.26}$$

If the analysis shows that the concrete has cracked, the ε_{DL} can be estimated from the measured ε_{LL}. In this case, both dead load and live load effects can be conservatively assumed to act on the same cracked cross-section. Therefore, a simple ratio of live load strain to live load moment can be equated to the ratio of dead load strain to dead load moment. Doing this gives

$$\varepsilon_{DL} = \left(\frac{\varepsilon_{LL}}{M_{LL}}\right) M_{DL} \tag{4.27}$$

In using this formula, it is important that the M_{LL} include the effect of transverse live load distribution as well as any effect of fill (an overestimate of M_{LL} will produce an non-conservative value for ε_{DL}. If this method for getting ε_{DL} is used either the AASHTO DF or measured DF needs to be applied.

4.3 Figures

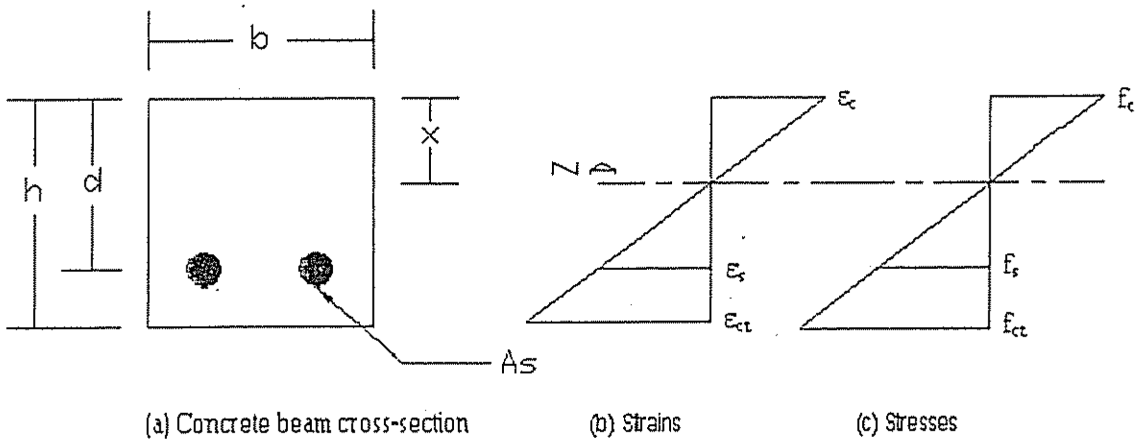


Figure 4.1: The Assumed Strain and Stress Distribution at Allowable Load Levels

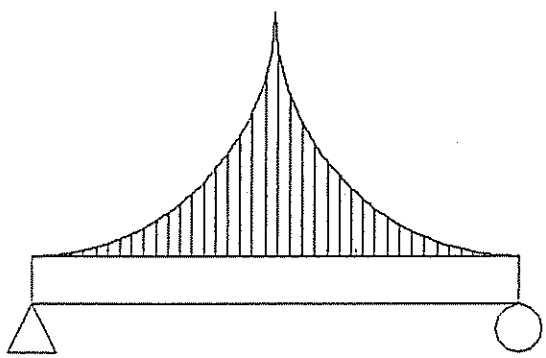


Figure 4.2: The Stress Distribution caused by the Fill when the Wheel Load is applied along the Roadway Surface

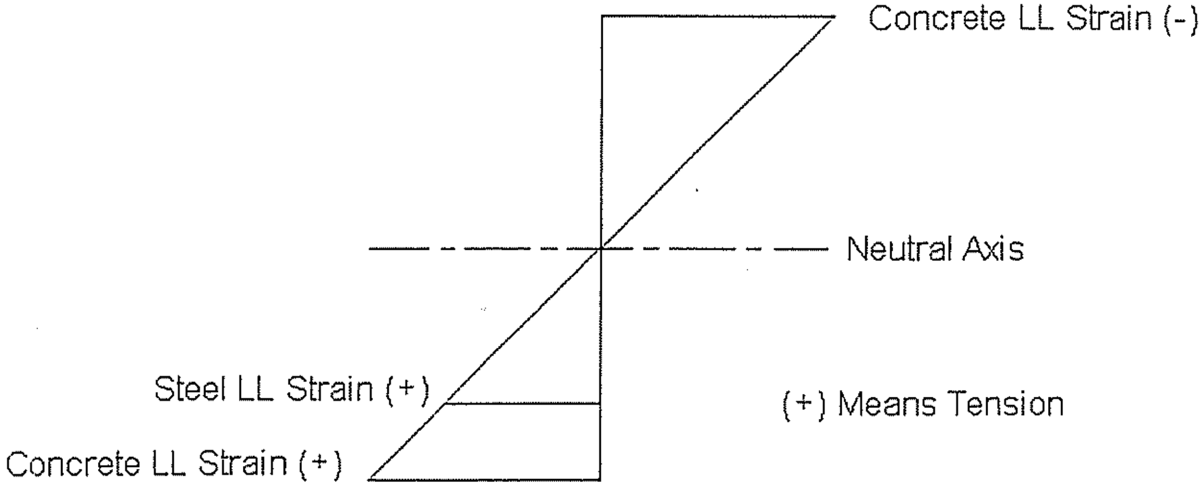


Figure 4.3: The strain distribution through the depth of a concrete beam during the load test

Chapter 5

The Steel Area Method: Laboratory Tests

5.1 Overview

The theoretical basis and resulting equations for the steel area method were presented in Chapter 4. A limited experimental investigation was conducted in the laboratory using three under-reinforced rectangular beams in order to test the steel area method. Displacements and strains due to a three point bending test were measured for each test. Two tests were performed on each beam. The beam was first tested in its pre-cracked state by loading to 80% of its ultimate strength, then tested in its cracked state by loading up to failure. The steel area method was then used to estimate the amount of reinforcing steel in each specimen based on the measured strains and displacements.

5.2 Specimens

The specimens tested measured 4 inches by 5 inches and were 46 inches in length (see Figure 5.1). The beams were fabricated with different amounts of reinforcing steel: one beam with a single #3 rebar, one beam with a single #4 rebar, and one beam with a single #5 rebar. Grade 60 steel rebar was used. Each beam was designed to satisfy the ACI ductility requirement and to fail in flexure.

A concrete mix having a water-cement ratio of 0.47 by weight was used. The mix design was based on Type I Portland cement and a maximum aggregate size of 0.75in. Two batches of concrete were required to fabricate the three beams. The beams were cast in the University of Delaware Structures Laboratory and were allowed to moist cure for 14 days. The concrete strength, determined by cylinder tests, was found to be 5.5 ksi.

5.3 Test Procedure and Instrumentation

The beams were loaded in three point bending in a Tinius Olsen universal-testing machine. The distance between supports was 42 inches. Each beam was first loaded in its pre-cracked state to 80% of its ultimate strength (# 3 rebar, P_{ult} = 1790 lbs; # 4 rebar, P_{ult} = 3130 lbs; # 5 rebar, P_{ult} = 4600 lbs). The load was then removed, then reapplied, and increased until the beam failed. Failure was defined as crushing of the concrete in the compression region. The test setup is shown in Figure 5.1.

Two BDI strain transducers were placed at midspan on the bottom surface of the beam. A dial gage was placed at midspan to measure deflection during testing. The location of both the transducers and the dial gage are shown in Figure 5.1.

5.4 Test Results

The results of the testing program are summarized in Figures 5.2 through 5.7 and Tables 5.1 through 5.3. The test results, as well as the analytical predictions of the steel area, are discussed in the following sections.

5.5 Determination of Steel Area

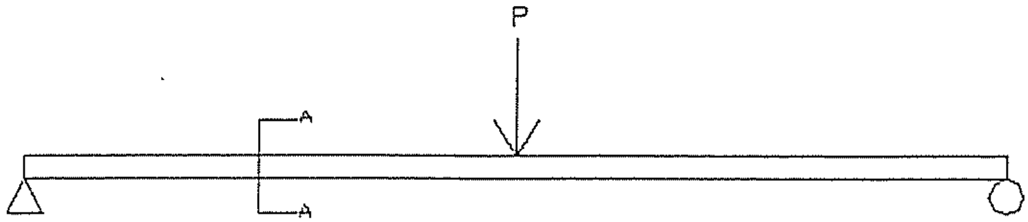
Load vs. strain and load vs. displacement plots were created for each beam using strain and displacement data obtained in the second load cycle in which the beam was loaded to failure. These plots are contained in Figures 5.2 through 5.7. The beam strain stiffness and the beam displacement stiffness was calculated from the slope of the best-fit curve of both plots. The theoretical and measured beam stiffnesses are presented in Tables 5.1 and 5.2. The steel area was then predicted using procedures developed in Sections 4.2.1.1 and 4.2.1.2. Both the actual and calculated steel areas are shown in Table 5.3.

5.6 Discussion

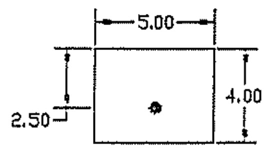
In theory, the beam strain stiffness should increase as the steel area increases. However, as noted in Table 5.1, the beam strain stiffness calculated from the test results does not follow this trend. Likewise, the beam displacement stiffness should increase as the steel area increases. However, as noted in Table 5.2, the beam displacement stiffness calculated from the slope of the load vs. displacement plot decreased as the steel area increased. Consequently, the estimated area of steel decreased rather than increased with the actual area of steel in the specimen. The trends observed are not encouraging, and they indicate a possible problem with the experimental setup. Some factors that may have contributed to the poor experimental results are difficulty in accurately measuring low strains and displacements of small-scale concrete beams, improperly bonded strain transducers and placement or location of the reinforcing steel in the specimens.

Irrespective of the obvious problems with the experimental program, the areas of steel predicted using the strain stiffness are of the correct order of magnitude. Furthermore, the predicted areas for the # 4 and # 5 rebar specimens are within 25 % of the actual area. These results do show that the steel area method based on the strain stiffness is relatively insensitive to experimental error.

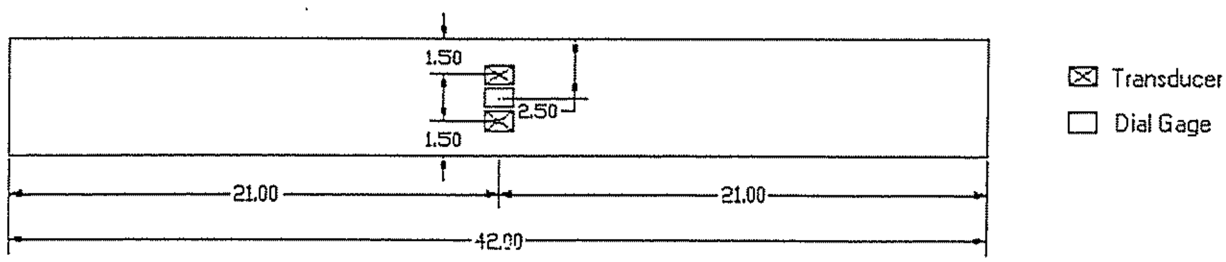
5.7 Figures



(a) Test Beam



(b) Section A-A View



(c) Test Beam Bottom View

Figure 5.1: Test Setup

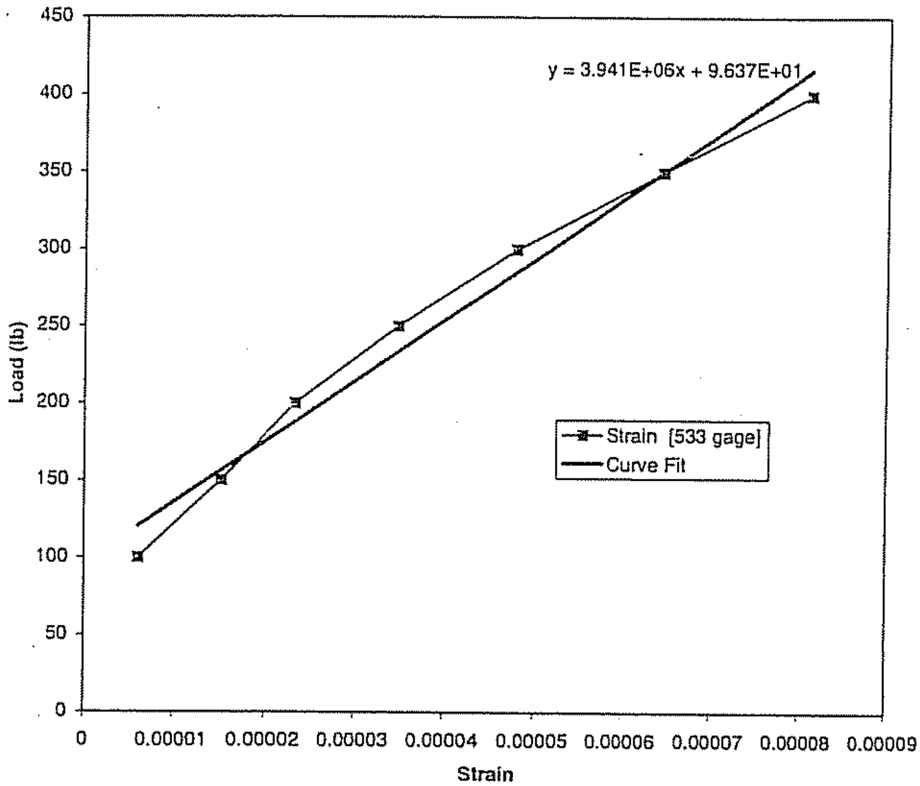


Figure 5.2: Beam Strain Stiffness for the # 3 rebar

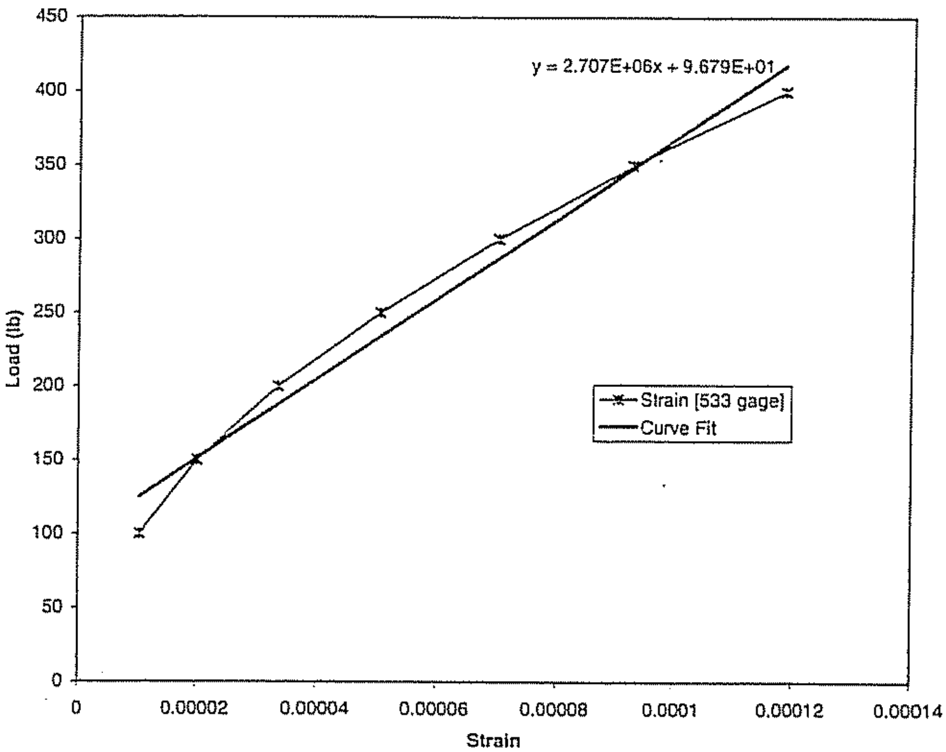


Figure 5.3: Beam Strain Stiffness for the # 4 rebar

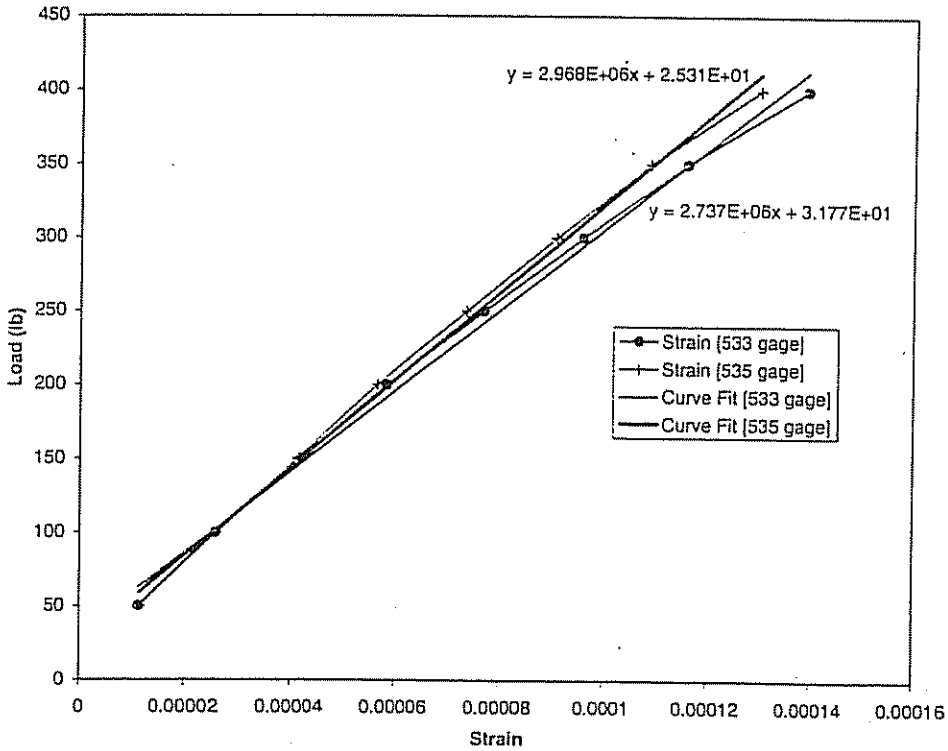


Figure 5.4: Beam Strain Stiffness for the # 5 rebar

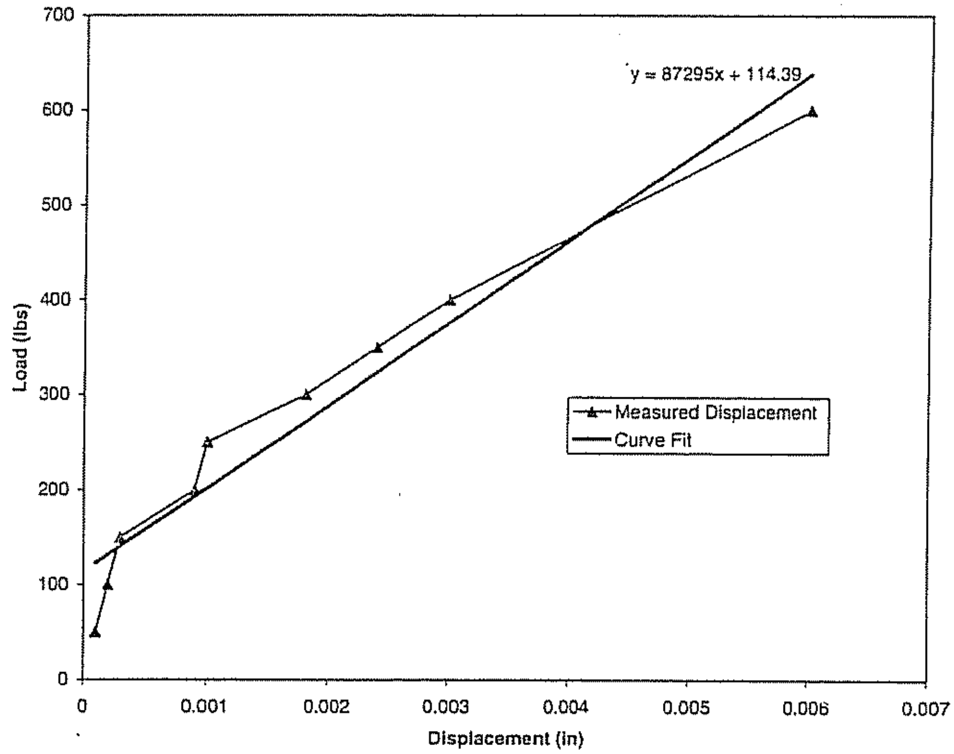


Figure 5.5: Beam Displacement Stiffness for the # 3 rebar

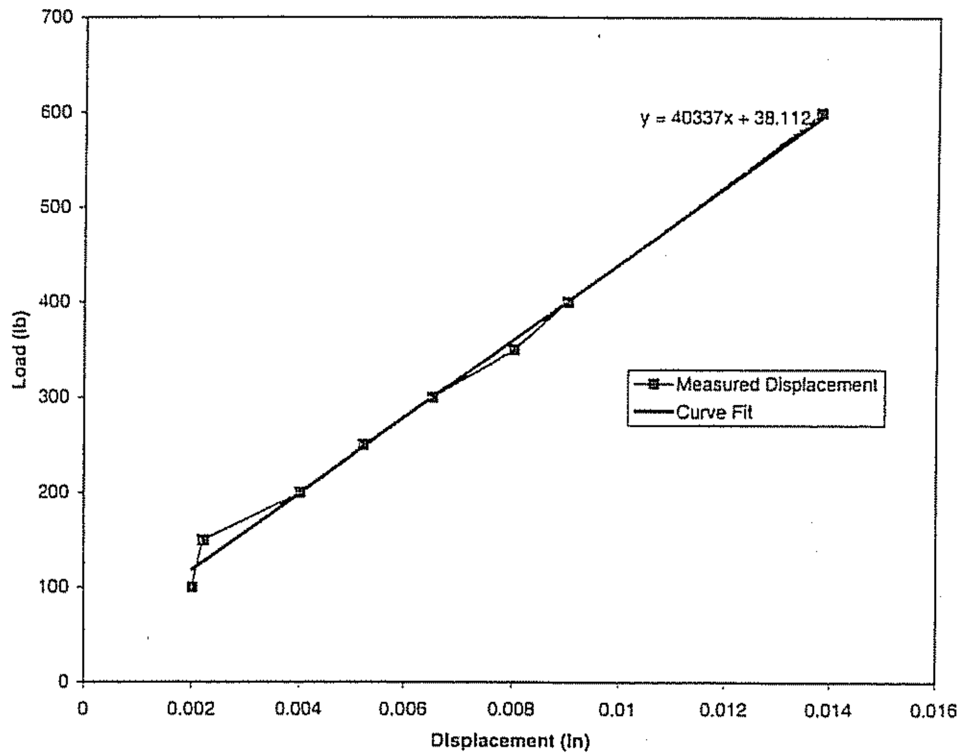


Figure 5.6: Beam Displacement Stiffness for the # 4 rebar

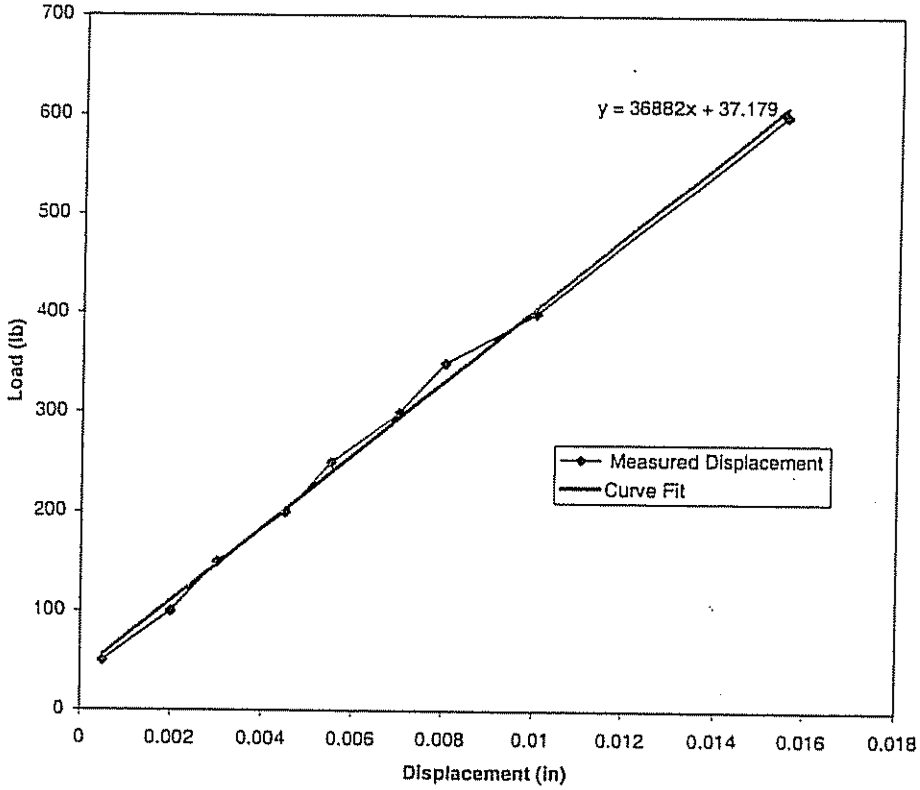


Figure 5.7: Beam Displacement Stiffness for the # 5 rebar

5.8 Tables

Table 5.1: The Theoretical and Calculated Beam Strain Stiffness

# rebar	theoretical k_{strain} [lbs/in]	calculated k_{strain} [lbs/in]
3	1238000	3941000
4	2213000	2707000
5	3756000	2853000

Table 5.2: The Theoretical and Calculated Beam Displacement Stiffness

# rebar	theoretical $k_{displacement}$ [lbs/in]	calculated $k_{displacement}$ [lbs/in]
3	12516.46	87295
4	19830.04	40337
5	27064.32	36882

Table 5.3: The Actual and Calculated Reinforcing Steel Area

# rebar	actual A_s [in^2]	A_s using strain data [in^2]	A_s using displacement data [in^2]
3	0.11	0.341	3.968
4	0.2	0.244	0.571
5	0.31	0.238	0.393

Chapter 6

Bridge Load Testing in the Field

6.1 Introduction

Chapter 6 discusses the use of bridge load test data to validate both the steel area and simplified methods. Several bridges on Route 13 in Delaware were tested in the fall of 1998. Transducers were used to record the strain on the bridge produced by pre-weighed test trucks. After the load tests were conducted, the strain data was used to evaluate and load rate one of the tested bridges using the methods developed in Chapter 4. All of the bridges tested did have plans; however, they are representative of concrete bridges for which plans do not exist. Because plans were available, the accuracy of the methods could be accessed.

The first topic discussed in this chapter is the description of the five Route 13 bridges that were load tested. The second topic discussed in Chapter 6 is the load test of Bridge 1-450 (northbound). The third topic discussed in Chapter 6 is the properties determined from the load test data obtained from the bridge. The fourth topic discussed in this chapter is the application of the steel area method to Bridge 1-450 in order to estimate a load rating. The last topic discussed in Chapter 6 is the use of the simplified method to load rate Bridge 1-450.

6.2 Description of the Tested Bridges

The University of Delaware performed diagnostic load tests on five bridges along U.S. Route 13 during November of 1998. These tests were part of a Delaware Transportation Institute project funded by the Delaware Department of Transportation. The primary purpose of the project was to allow permit vehicles to use Route 13 prior to the final completion of new Route 1. Route 1 will be a modern limited-access highway running south from Interstate 95 to the Delaware beaches. Currently, Route 1 is either under construction or already completed along the entire route, except for the 5-mile section between Odessa and Symra. DelDOT wanted to know if five slab bridges along old U.S. Route 13 could carry heavy permit vehicles that can now be accommodated on the remainder of Route 1. While the primary purpose of the load testing was to evaluate permit vehicle capacity, the data was also used here to access the methods developed for evaluating planless bridges.

The five bridges tested along U.S. Route 13 are denoted as Bridges 1-450, 1-486, 1-487, 1-489, and 1-492. They are all concrete slab bridges with two lanes of traffic in each direction plus a median. Two feet of material exists between the asphalt wearing surface and the superstructure. This material, later referred to as fill and slab, consists of a combination of an old concrete roadway (slab) and soil (fill). U.S. Route 13 was built in two sections: southbound U.S. Route 13 was built in the 1920's while northbound U.S. Route 13 was built in the 1930's.

Southbound Bridge 1-492 was built in 1920 and carries Route 13 over Sawmill Branch creek. The bridge has a span of 6 ft, a section depth of 10 inches and a design concrete strength of 2.5-ksi.

Southbound Bridge 1-489 was built in 1932 and has a span of 8.17 feet. It has a 10-inch section depth and 2.5-ksi concrete design strength.

The southbound and northbound spans of Bridge 1-487 carry Route 13 over Sandom Branch creek. The southbound span was built in 1920 and the northbound span was built in 1931. This bridge has a span of 14.19 ft, a 15-inch section depth and 2-ksi concrete design strength.

Northbound Bridge 1-486 was built in 1931 and carries Route 13 over Hangmans Run creek. The bridge has a span of 6.09 ft, a 10-inch section depth and 2.5-ksi concrete design strength.

Northbound Bridge 1-450 was built in 1931 and carries Route 13 over Herron Run creek. The bridge has a span of 8 ft, a 10-inch section depth and 3-ksi concrete design strength.

Bridge 1-450 was chosen to illustrate the two methods developed for load rating concrete slab bridges without original design plans. This bridge was selected because it had the highest recorded strains of the five bridges tested.

6.3 Load Test of Northbound Bridge 1-450

A semi-static diagnostic load test was performed on Delaware Bridge 1-450 northbound in November of 1998. The bridge was instrumented and tested in approximately two hours. The instrumentation and testing procedure will now be described. In addition to the load tests, concrete cores were taken from three of the five bridges mentioned earlier.

6.3.1 Instrumentation Procedure

Only the span carrying northbound traffic was load tested since both the northbound and southbound spans had similar structural dimensions and properties. A total of 12 BDI strain transducers were installed on the bottom surface of the concrete slab. Nine strain transducers measured three inches in length and three strain transducers measured 12 inches (with the use of extensions). The shorter transducers are sufficient to monitor compressive strains in concrete; however, they may not provide accurate strain readings for concrete in tension when cracking occurs. Longer transducers allow an average strain to be taken over a longer gage length and minimize the effects of high-localized strains resulting from tensile cracking.

Ten strain transducers were placed 3-feet on center along a line transverse to the traffic direction at mid-span since the maximum moment (and therefore strain) was expected to occur there. These gages can be used to determine effective slab widths and transverse load distribution factors. Two strain transducers were placed six inches from the edge of the concrete slab to allow one to evaluate end conditions of the slab. The strain transducers placed at mid-span were spaced three feet apart starting at 29 feet and 9 inches from the concrete slab edge. The gage layout is shown in Figure 6.1. The transducers were attached to the bridge using a quick setting epoxy.

6.3.2 Test Loading

The test vehicle configuration used to load the bridge is shown in Figure 6.2. The test vehicle was a three-axle, single unit truck with a total weight of 29.6 tons. In choosing the test vehicle weight, one wants to make sure that the bridge will not be stressed beyond its linear-elastic range. Based on the current rating of the bridge, 29.6 tons was determined to be a safe load level. All the strain responses were checked to make sure they went back to zero after each unloading.

Reference markers were made in the roadway to locate the truck paths relative to the gages. The origin on Delaware bridge 1-450 northbound was located at 29 feet and 9 inches from the concrete slab edge. Three truck paths were chosen according to the driver's side front wheel.

A temporary white chalk line was placed at the origin in order to determine the load path directions. The first load test was conducted using ambient traffic conditions. The second load test was conducted with the drivers wheel line moving forward and backward along the white chalk line. The third load test was conducted with the passenger wheel line moving forward and backward along the white chalk line.

6.4 Test Results

6.4.1 In-situ Concrete Strengths

Three-inch diameter concrete cores were taken from 3 bridges (one core each). Table 6.1 shows the bridge number, design year, design concrete strength and core strength. From this data, it appears that concrete strength of 4 ksi can be conservatively used for all bridges.

6.4.2 Summary of Strain Data

Although four load cases were conducted, only load cases two and three were used in the evaluation of the steel area and simplified methods. After all the load tests were completed, examination of the data revealed that a maximum strain value of approximately 15 microstrain was recorded. It occurred at transducer 291 during load test three. The maximum strain values recorded during load tests two and three for all gages are shown in Table 6.2. The rest of the load test data is contained in Appendix A. The properties determined from load test data as well as the load ratings obtained by both the steel area method and the simplified method are discussed below.

6.4.3 Properties Determined from Load Test Data

A few significant properties were determined using data from the diagnostic test of Bridge 1-450. These properties included the transverse load distribution factor, the effective wheel load resulting from the effect of fill, the slab strain stiffness and the reinforcing steel area. Their derivation was discussed in Chapters 4 and 5.

6.4.3.1 Transverse Distribution Factor

Presented in Figure 6.3 is a plot of the longitudinal strain along the transverse centerline for the two load cases. From these plots, the effective width and transverse distribution factor can be determined as outlined in Section 4.2.2.1. The effective beam width determined from load test data was 9.1 feet for load test two and 9.3 feet for load test three. Therefore, the average transverse load distribution factor is estimated to be 0.11.

The effective beam width (E) for a concrete slab bridge can also be estimated using the AASHTO formula [11]:

$$E = 4 + 0.06S \tag{6.1}$$

where S = The bridge span length

The transverse load distribution factor is equal to the inverse of the effective beam width (1/E). In general, this formula tends to be conservative compared to the distribution factor calculated from actual load test data. The effective width that was calculated by Equation 6.1 for Bridge 1-450 is 4.48 ft. This results in a transverse load distribution factor of 0.22 which is twice the value determined from the load test.

6.4.3.2 Effective Wheel Load

The effective wheel loading for Bridge 1-450 was calculated considering no effect of the fill or slab, as well as considering the effect of both the slab and fill. The effective wheel load with 2 feet of fill present was calculated using both the

Boussinesq method and the AASHTO method. The effect of fill upon the effective wheel load was significant since the effective wheel load without fill was much higher than the effective wheel load with fill. This shows that the fill causes the wheel load to be distributed longitudinally and transversely over the entire span, thus reducing the magnitude of the effective wheel load. The effective wheel load that was estimated by the AASHTO method is approximately 1.6 times larger than the effective wheel load that was estimated by the Boussineq method. Values for the effective wheel loads for Bridge 1-450 are given in Table 6.3. For example, the average front wheel weight is 7355 lbs. Once the wheel load is multiplied by the computed DF for test two (DF=0.11), P_{eq} = 811 lbs. Considering the effect of fill, P_{eq} = 191 lbs and 302 lbs based on the Boussinesq and AASHTO methods respectively.

6.4.3.3 Strain Stiffness

The strain stiffness was determined from a plot of longitudinal strain versus effective wheel load, considering the front and rear truck axles as independent loads. The strain stiffness was calculated assuming no fill or slab, as well as including the effect of both the slab and fill. The actual wheel load is a concentrated force when both the slab and fill are neglected. This results in the highest slab stiffness since no fill is assumed present to distribute the wheel load. The actual wheel load is distributed across the entire bridge span when either the slab or fill are considered between the roadway and bridge superstructure. This causes the strain stiffness to decrease significantly. The results of these calculations are given in Tables 6.4 through 6.6.

6.4.3.4 The Reinforcing Steel Area

The actual reinforcing steel area for Delaware Bridge 1-450 northbound is 1.05 in^2/ft. The reinforcing steel area for this bridge was estimated using load test data, and the steel area method. The reinforcing steel area was calculated for the following three cases:

Case 1: Superstructure only, effect of fill neglected.

The actual wheel load acts as a concentrated force when no fill is present between the roadway and bridge. In addition, the beam depth is 10 inches when only the concrete superstructure is included in the reinforcing steel calculations. Therefore, the reinforcing steel area is significantly higher than if the effect of fill and roadway slab is included.

Case 2: The effect of the superstructure and fill is included.

The actual wheel load acts as a distributed load when fill is present between the roadway and superstructure. This will lower the reinforcing steel area calculated for the bridge. However, the effect of the concrete slab roadway was not included in this calculation (beam depth is 10 inches). Therefore, the reinforcing steel area will still be higher than if the effects of the superstructure, fill and roadway slab are included.

Case 3: The effect of the superstructure, fill and concrete roadway slab are included

The actual wheel load acts as a distributed load when fill is present between the asphalt roadway and superstructure. In addition, the beam depth is 22 inches when the roadway slab is included. This significantly lowers the calculated reinforcing steel area for the bridge.

The results of the reinforcing steel calculations are presented in Tables 6.7 through 6.11. The actual reinforcing steel area in Bridge 1-450 is 1.05 in^2 per foot of beam width. The average reinforcing steel area obtained for Case 1 was 6.283 in^2. The average reinforcing steel area obtained for Case 2 was 1.525 in^2 when the effective wheel load was calculated by the Boussineq method, and 2.42 in^2 when the effective wheel load was calculated by the AASHTO method. Finally, the average reinforcing steel area for Case 3 was 0.693 in^2 when the effective wheel load was calculated by the Boussineq method, and 1.09 in^2 when the effective wheel load was calculated by the AASHTO method.

The average reinforcing steel area for Case 1 was significantly higher than the actual reinforcing steel area in the bridge. However, the estimated steel area for Cases 2 and 3 were relatively close to the actual reinforcing steel area. Therefore, the effect of the fill and the prior roadway slab on the estimated reinforcing steel area was significant. In addition, the method used to determine the effective wheel load had a significant effect on the estimated reinforcing steel areas for Cases 2 and 3.

6.5 The Steel Area Method Load Rating

Once the area of reinforcing steel within the superstructure is known, BRASS can be used to determine the bridge load rating. In addition, one can update the BRASS command file to include the transverse load distribution factor and impact factor determined from the load test as well as the effect of fill. The accuracy of the bridge load rating is determined by the accuracy of the area of steel, transverse load distribution factor, and impact factor estimates as well as whether or not the effect of fill is included.

6.6 Load Rating Using the Simplified Method

Northbound Bridge 1-450 was load rated for an HS20-44 truck using the simplified method for Cases 1 and 2 given in Section 6.4.3.4. The ASD inventory rating factor was calculated using Equation (4.18). This equation was a function of the impact factor, the allowable strain, the live load strain and the dead load strain. The impact factor and the allowable strain were known before the load test. The live load strain for the 29.6-ton test truck was measured directly during the load test. Based on the ratio of the weight of the rear tandem axles of the test truck to the weight of the tandem axles of an HS20-44, the appropriate live load strain is found. The dead load strain was conservatively estimated using the procedure presented in Section 4.2.3. To clarify the procedure, the steps followed in the rating of Bridge 1-450 are presented.

Table 6.13 contains the rating factors calculated using the simplified method as well as the data needed to use the simplified method.

The ASD inventory rating factor for the simplified method is given by

$$RF = \frac{\varepsilon_{all} - \varepsilon_{DL}}{\varepsilon_{LL}(1+I)} \tag{6.2}$$

For the bridge, $f_y = 33$ ksi. Based on this this, and using Equation (4.20), which gives an allowable stress for the inventory ratings, we find that

$$\varepsilon_{all} = \frac{0.55 f_y}{E_s} = 626\mu\varepsilon \tag{6.3}$$

Due to the 2 feet of fill, I = 0.2. From the load test, the maximum live load strain on the beam soffit for an HS20-44 truck, is estimated to be

$$\varepsilon_{LL} = 15\mu\varepsilon\left(\frac{32k}{44.44k}\right) = 10.8\mu\varepsilon$$

Therefore, we will conservatively estimate the $\varepsilon_{LL} = 10.8\ \mu\varepsilon$. The final value needed is an estimate of ε_{DL}. Assuming the concrete does not crack we get

Assuming the concrete does not crack we get

$$\varepsilon_{DL} = \frac{6M_{DL}}{E_C bh^2} \tag{6.4}$$

Considering the weight of the fill and roadway slab, and simple supports, we have M_{DL} = 31200 in-lb. Using $E_C = 57000\sqrt{f'c}$ where f'c is taken to be 4000 psi, we get

$$\varepsilon_{DL} = \frac{(5)(31200in-lb)}{[57000\sqrt{4000}psi][12in][10in]^2} = 43.27x10^{-5}$$

$$\varepsilon_{DL} = 43.27\mu\varepsilon$$

Since 43.27με is less than the cracking strain estimated to be 132με using Equation 4.23 is acceptable. In this case, considering fill, the ratio of the live load to dead load moment is

$$\frac{M_{LL}}{M_{DL}} = 0.23$$

Since the small live load strains will not cause cracking, we would expect the ratio of live load to dead load strain to be similar. Indeed, in this case we have fairly good agreement with

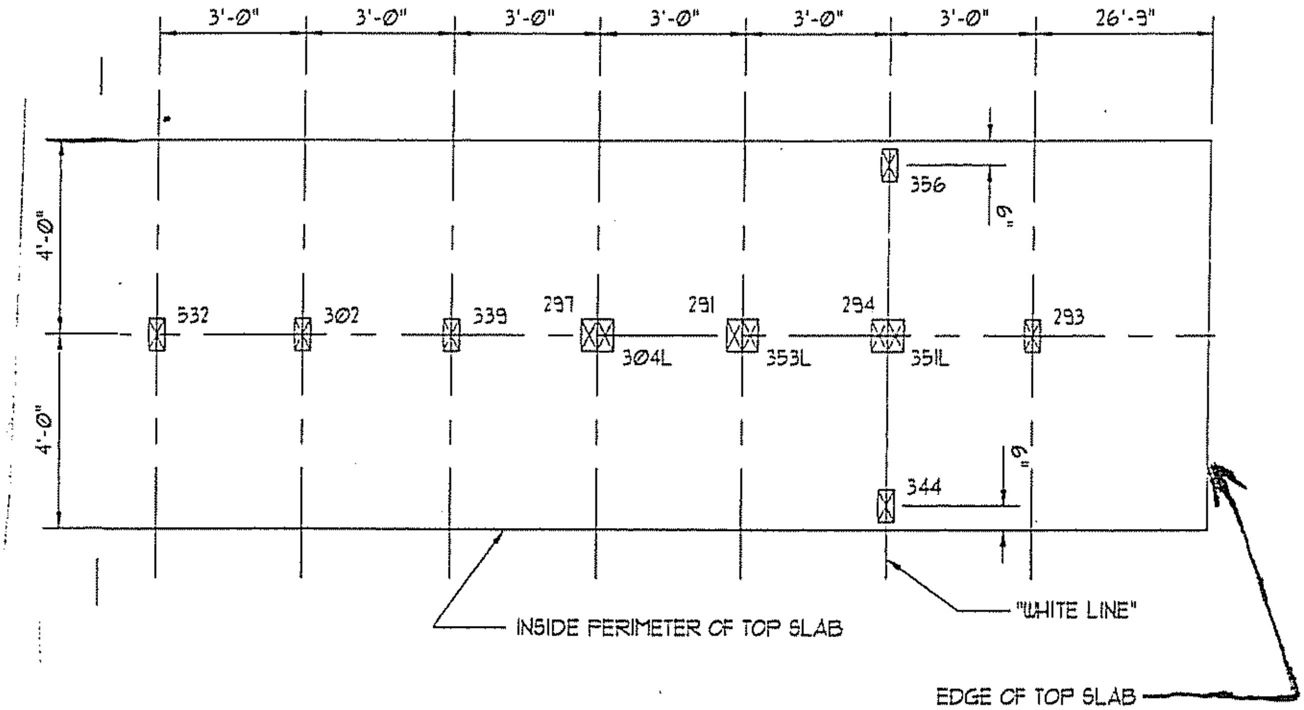
$$\frac{\varepsilon_{LL}}{\varepsilon_{DL}} = \frac{10.8}{43.3} = 0.25$$

Substituting values for ε_{all}, ε_{DL}, ε_{LL} and I into Equation (4.18) we get

$$RF = \frac{626 - 43.3}{10.8(1 + 0.2)} = 45.0$$

To access the results of the simplified rating method, Bridge 1-450 was first rated using known bridge properties given in the BRASS command file and the design plans combined with updated information from the load test. Three cases, A, B, and C were considered. In Case A, the original parameters from the BRASS file were used. This file has I = 0.3, no effect of fill, and a DF = 0.22 computed using the AASHTO formula. In Case B, the AASHTO DF = 0.22 is retained, however the effect of fill is incorporated and therefore I = 0.2 and the effective wheel load is reduced. In this case only the AASHTO method for addressing fill is used. In Case C, the effects of fill are included as well as using the DF = 0.11 found from the test data. This case most closely approximates the in-situ properties of the bridge and its response. Rating factors for the simplified method and Cases A, B, and C are given in Table 6.13.

6.7 Figures



L denotes transducer with a 12 inch gage length

Figure 6.1: The Gage Layout for Delaware Bridge 450 Northbound

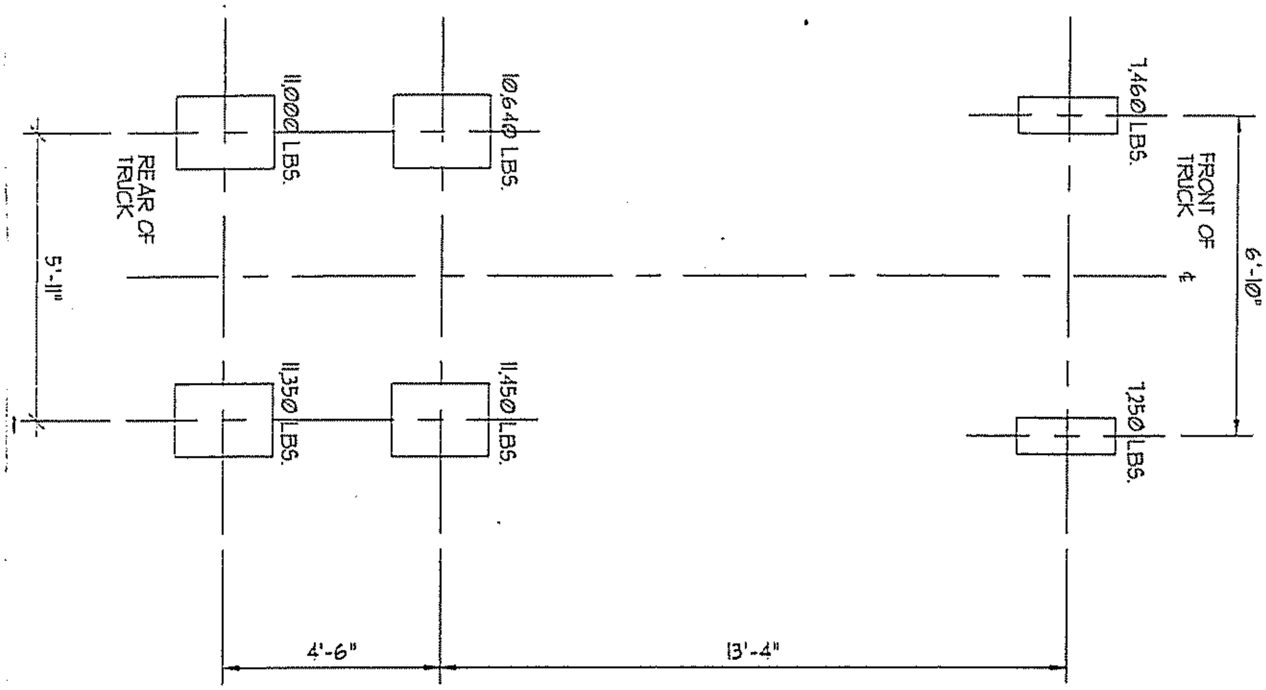


Figure 6.2: Test Vehicle Configuration for Delaware Bridge 450 Northbound

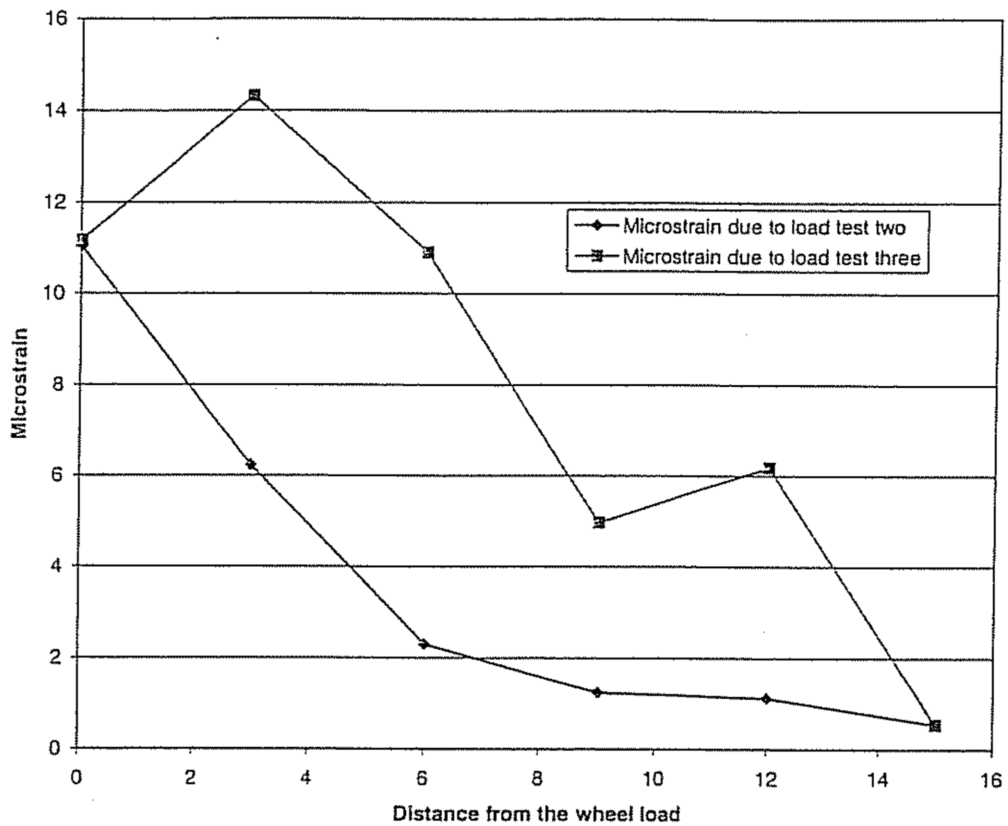


Figure 6.3: The distance from the wheel load vs. microstrain plot used calculate the transverse load distribution factor for load tests two and three

6.8 Tables

Table 6.1: Concrete core strengths determined from selected bridges

Bridge #	Year Built	Design Strength (ksi)	Measured Strength (ksi)
1-487 N	1931	2	6.58
1-489 S	1920	2.5	5.76
1-492 S	1920	2.5	4.76

Table 6.2: Maximum recorded strains from load tests two and three

	Maximum Microstrain Values	
Gage #	Load Test Two	Load Test Three
532	1	3
302	1	6
297	3	11
339	1	5
291	7	15
356	5	5
351*	8.75 (35)	9.5 (38)
353*	5.75 (23)	11.75 (47)
293	12	5
304*	2.5 (10)	11.75 (34)
294	11	11
344	5	6

* Indicates 12" gage length, strain in parentheses is 4 times actual strain

Table 6.3: Effective Wheel Loads for Bridge 1-450

	P_{eq} [Without fill] (lb)	P_{eq} [Boussineq method] (lb)	P_{eq} [AASHTO method] (lb)
Load test two			
Front wheel	811	191	302
Back wheel	1233	289	459
Load test three			
Front wheel	792	186	295
Back wheel	1204	282	448

Table 6.4: Strain Stiffness Neglecting Fill

	Concrete Slab Stiffness [lbs/in]		
	Load test two (transducer 294)	Load test three (transducer 294)	Load test three (transducer 297)
Truck moving forward	7.964×10^7	7.777×10^7	5.977×10^7
Truck moving backward	8.964×10^7	7.774×10^7	6.521×10^7

Table 6.5: Strain Stiffness with Fill (Effective wheel load calculated by the Boussinesq method)

	Concrete Slab Stiffness [lbs/in]		
	Load test two (transducer 294)	Load test three (transducer 294)	Load test three (transducer 297)
Truck moving forward	1.866×10^7	1.823×10^7	1.402×10^7
Truck moving backward	2.099×10^7	1.823×10^7	1.529×10^7

Table 6.6: Strain Stiffness with Fill (Effective wheel load calculated by the AASHTO method)

	Concrete Slab Stiffness [lbs/in]		
	Load test two (transducer 294)	Load test three (transducer 294)	Load test three (transducer 297)
Truck moving forward	2.966×10^7	2.895×10^7	2.225×10^7
Truck moving backward	3.335×10^7	2.893×10^7	2.427×10^7

Table 6.7: Estimated Steel Area for Case 1 (Neglecting Fill)

	Estimated Slab reinforcing steel area [in^2]		
	Load test two (transducer 294)	Load test three (transducer 294)	Load test three (transducer 297)
Truck moving forward	6.66	6.51	5.08
Truck moving backward	7.42	6.51	5.22
Average	6.283		
Actual	1.05		

Table 6.8: Estimated Steel Area for Case 2 (Effective wheel load calculated by the Boussinesq method)

	Estimated Slab reinforcing steel area [in^2]		
	Load test two (transducer 294)	Load test three (transducer 294)	Load test three (transducer 297)
Truck moving forward	1.62	1.58	1.22
Truck moving backward	1.82	1.58	1.33
Average	1.525		
Actual	1.05		

Table 6.9: Estimated Steel Area for Case 2 (Effective wheel load calculated by the AASHTO method)

	Estimated Slab reinforcing steel area [in^2]		
	Load test two (transducer 294)	Load test three (transducer 294)	Load test three (transducer 297)
Truck moving forward	2.57	2.51	1.93
Truck moving backward	2.89	2.51	2.11
Average	2.42		
Actual	1.05		

Table 6.10: Estimated Steel Area for Case 3 (Effective wheel load calculated by the Boussinesq method)

	Estimated Slab reinforcing steel area [in^2]		
	Load test two (transducer 294)	Load test three (transducer 294)	Load test three (transducer 297)
Truck moving forward	0.74	0.72	0.55
Truck moving backward	0.83	0.72	0.6
Average	0.693		
Actual	1.05		

Table 6.11: Estimated Steel Area for Case 3 (Effective wheel load calculated by the AASHTO method)

	Estimated Slab reinforcing steel area [in^2]		
	Load test two (transducer 294)	Load test three (transducer 294)	Load test three (transducer 297)
Truck moving forward	1.17	1.13	0.865
Truck moving backward	1.3	1.13	0.945
Average	1.09		
Actual	1.05		

Table 6.12: The Simplified Method Load Ratings for Northbound Bridge 1-450

	DF	Impact Factor	Allowable Strain (microstrain)	Dead Load Strain (microstrain)	Live Load Strain (microstrain)	Rating Factor
Simplified Rating	N/A	0.2	626	43.27	10.8	45
Case A	0.2	0.3	626	16.64	59.9	7.83
Case B	0.2	0.2	626	43.27	15	32.3
Case C	0.1	0.2	626	43.27	7.24	67.1

Chapter 7

Conclusion

The objective of this research was to develop simple and rational methods for load rating bridges without original design plans using field test data. The first step was to search the Delaware Bridge Inventory to see which bridge types in Delaware had no original design plans. This search indicated that bridges without original design plans in Delaware were most commonly constructed with concrete and steel. The most common bridge types without design plans in Delaware were slabs, arch-decks, stringer/ multi-beams and culverts.

Concrete slab and culvert bridges without design plans are especially difficult to rate using conventional methods since the reinforcing steel area often is unknown. Therefore, a method was developed to estimate the reinforcing steel area in concrete slabs using field test data. This was done by developing equations relating both the displacement beam stiffness and the strain beam stiffness to the reinforcing steel area. A test program was conducted to investigate the ability of analytical methods combined with test results to predict the amount of steel reinforcement. This program consisted of testing three rectangular concrete beams, each beam having a different reinforcing steel area. The displacements and strains due to a three point bending test were measured. The test results indicated that the method had some error due to a combination of either the simplified stiffness equations or difficulties encountered in measuring accurate low-level strains and displacements for the small-scale beam tests.

The beam strain stiffness was used in the full-scale bridge test since strains were more likely to be accurately measured than displacements.

Two methods were developed for load rating concrete slab bridges without design plans: the steel area method and the simplified method. Both methods were used for load rating Delaware Bridge 1-450 northbound. While the plans are available for Bridge 1-450, the methods were applied assuming details of the reinforcement were unknown. The load rating calculated from both methods indicate that Delaware Bridge 1-450 northbound can carry permit vehicles in its present condition. The main advantage of the steel area method is that the slab stiffness and reinforcing steel area can be determined along with the load rating. However, the steel area method has the following disadvantages:

1. It is sensitive to the amount of fill between the slab and roadway as well as the roadway depth.
2. The equations used to determine the reinforcing steel area contain some errors due to their simplified nature and difficulty measuring low level strains.

Therefore, the steel area method will only provide an estimate of the reinforcing steel area present and therefore an approximate load rating for a given concrete slab bridge. As a result, the inventory and operating load rating for Delaware Bridge 450 northbound should not be expected to match the original BRASS rating even when the effect of the superstructure, fill and roadway slab are included. The simplified method is easy to use and will immediately tell you when a bridge needs to be posted.

However, the reinforcing steel area is not determined using the simplified method. Therefore, the simplified method will not give the parameters for a BRASS data file, but will only give a bridge rating.

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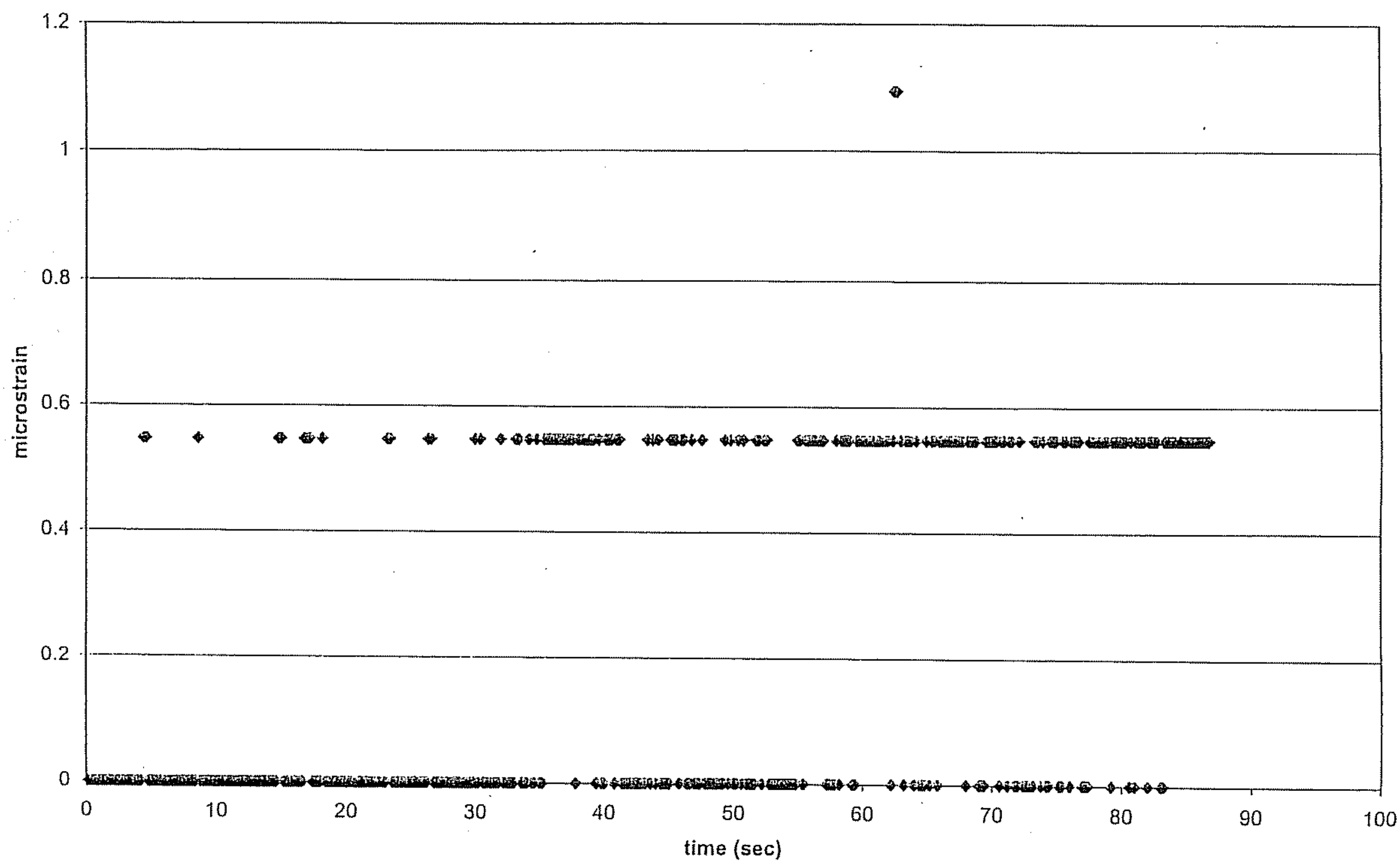
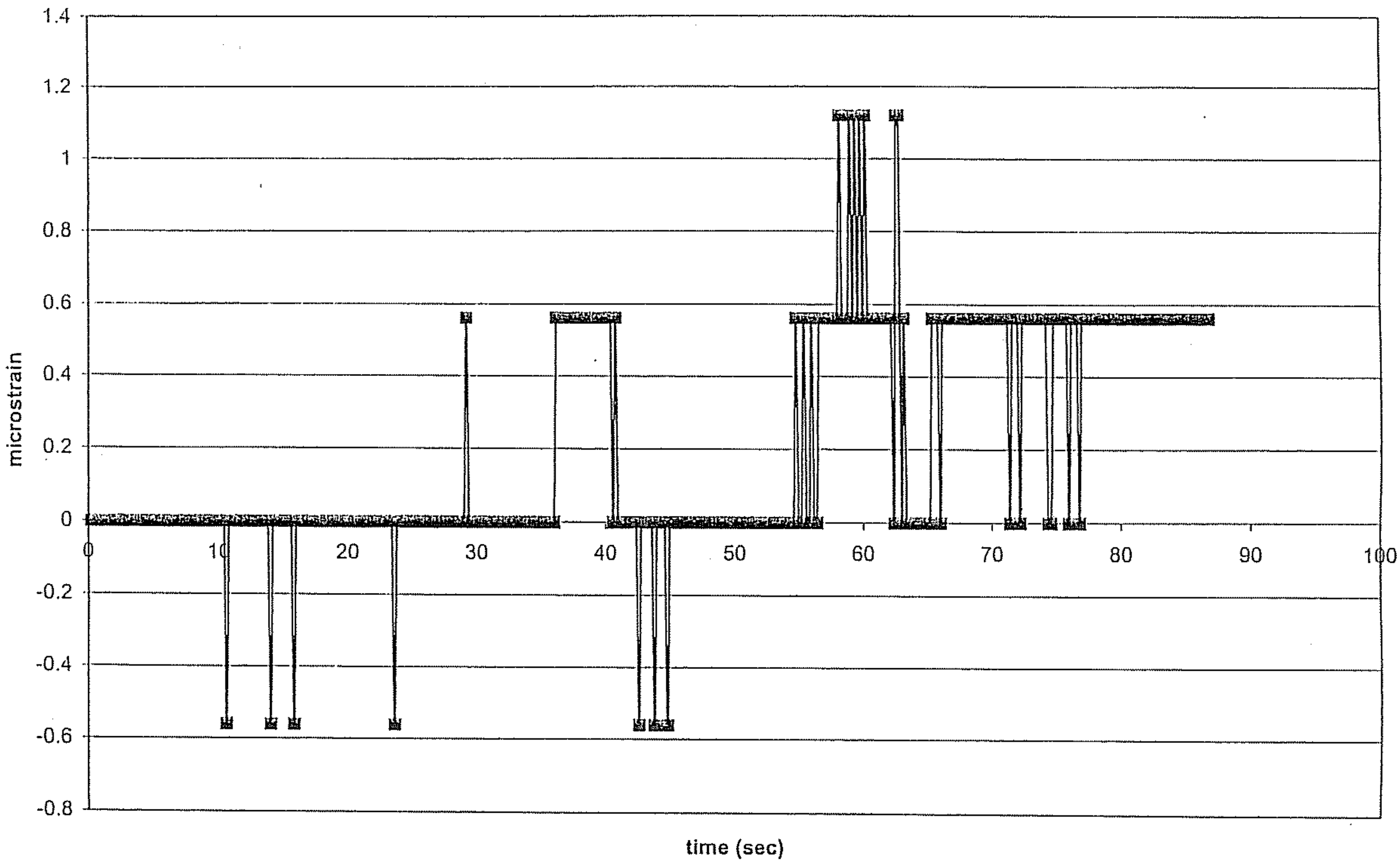
11. AASHTO. LRFD Bridge Design Specifications and Commentary. Washington, DC, 1994.

Appendix A. Test Results from bridge 1-450N

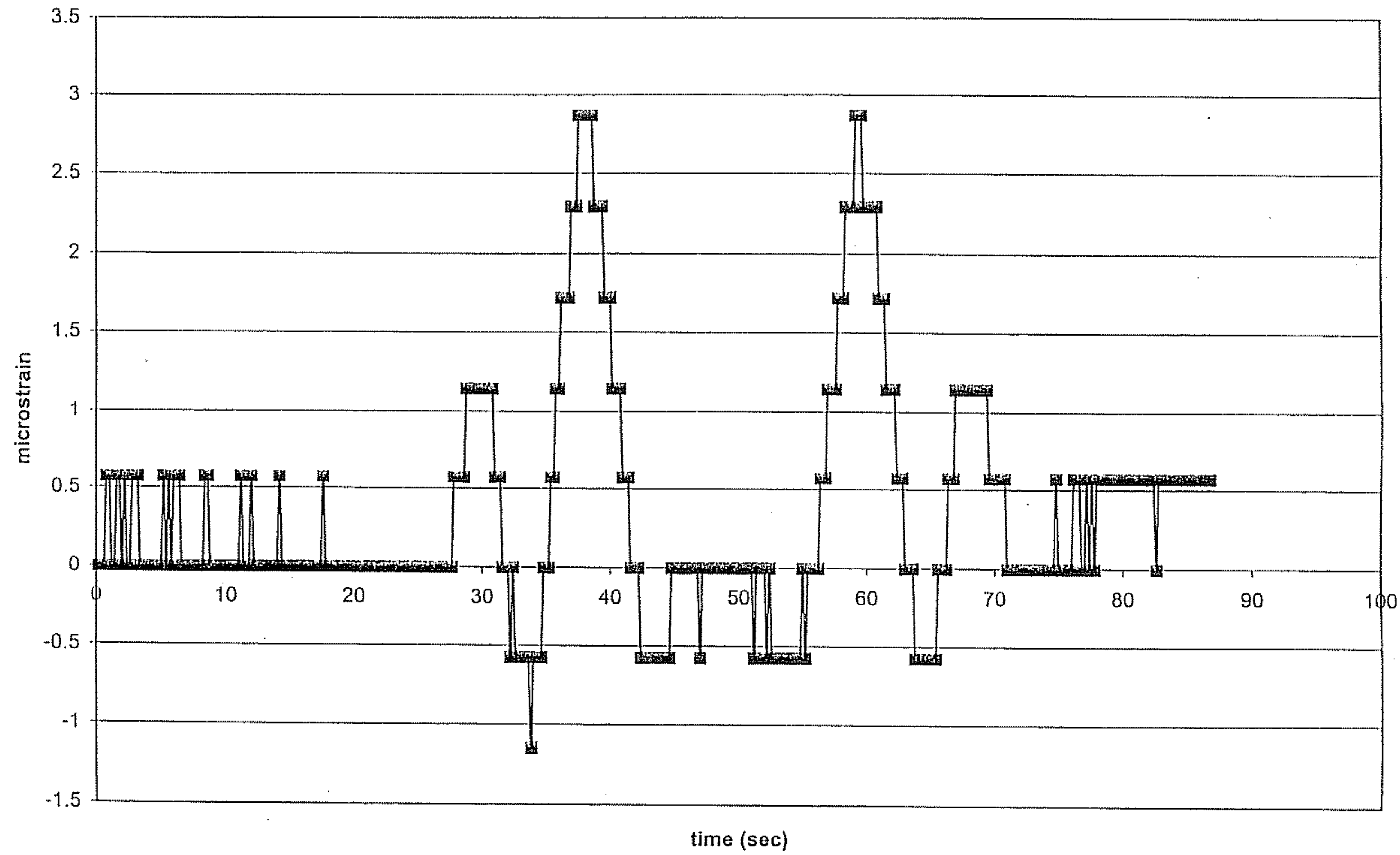
Time history (transducer 532) for pass 2 of northbound bridge 450



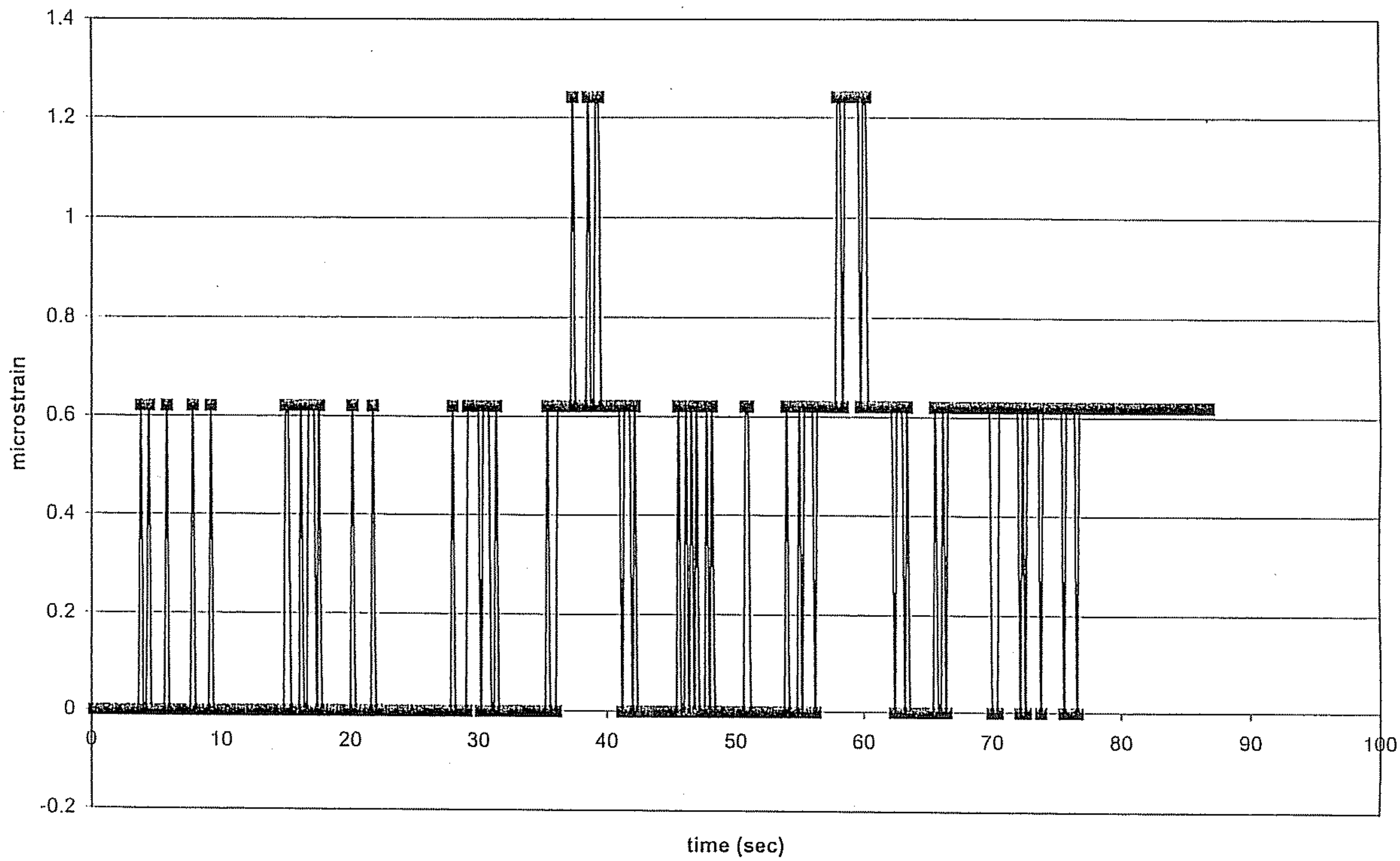
Time history (transducer 302) for pass 2 on northbund bridge 450



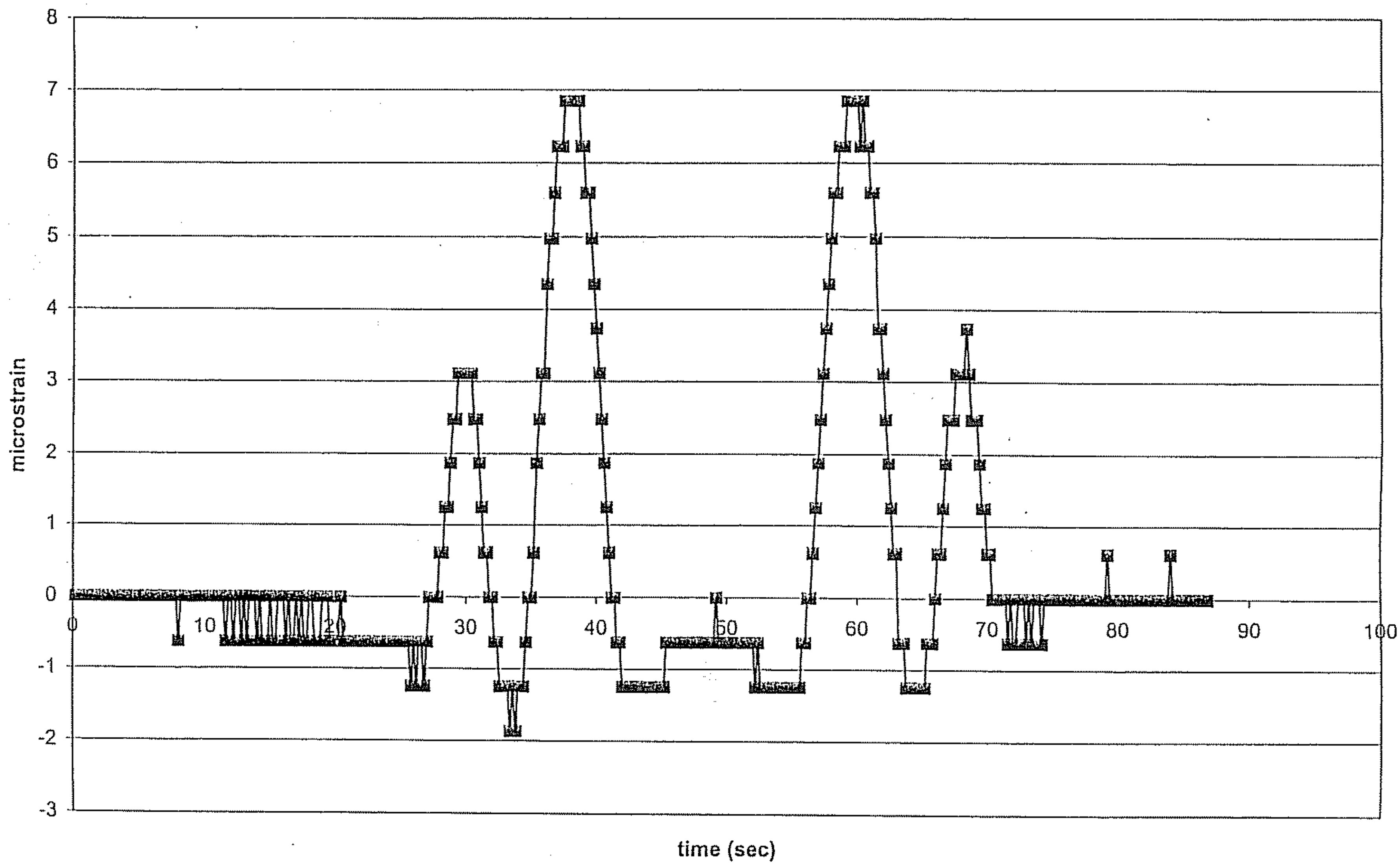
Time history (transducer 297) for pass 2 on northound bridge 450



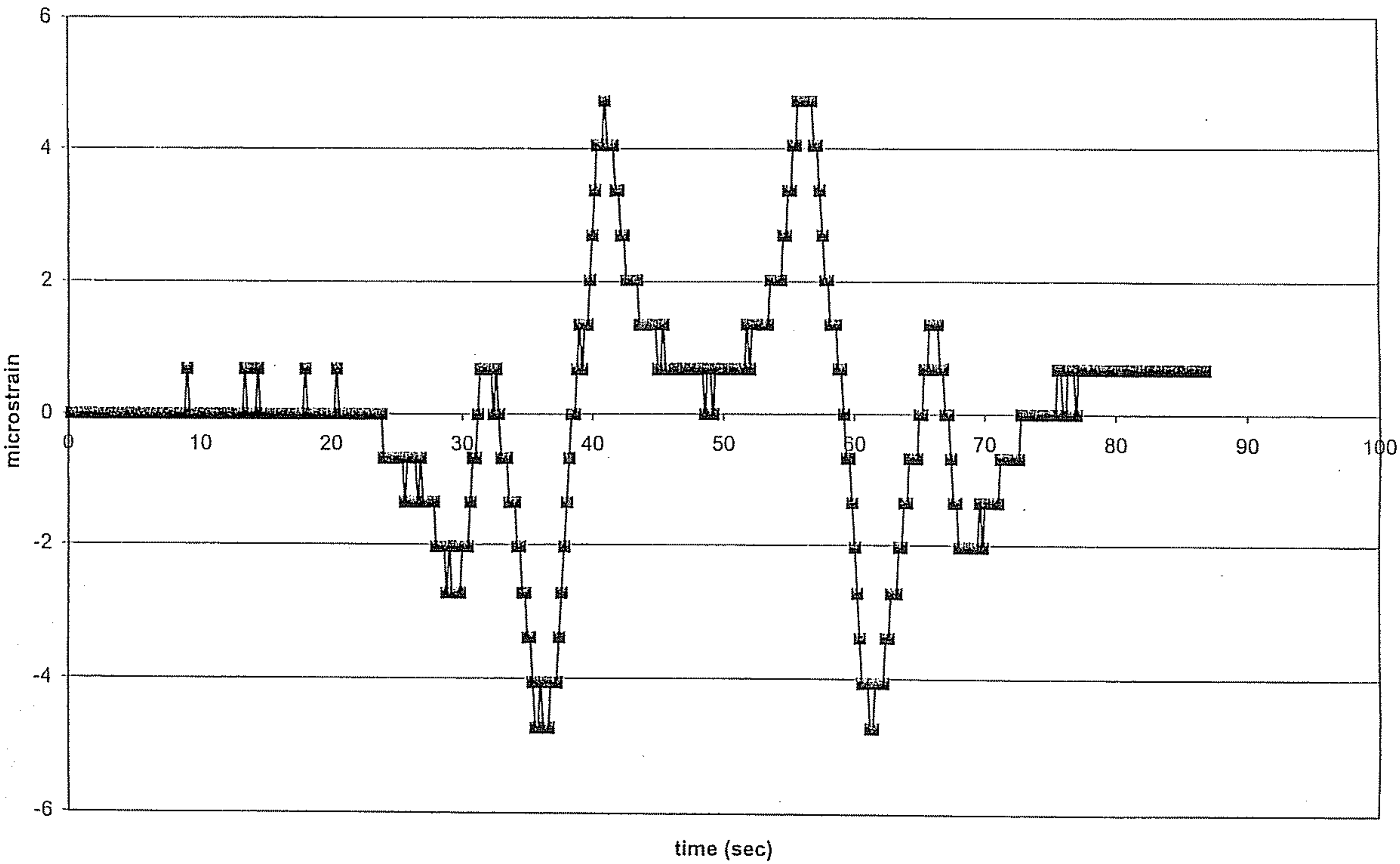
Time history (transducer 339) for pass 3 on northbound bridge 450



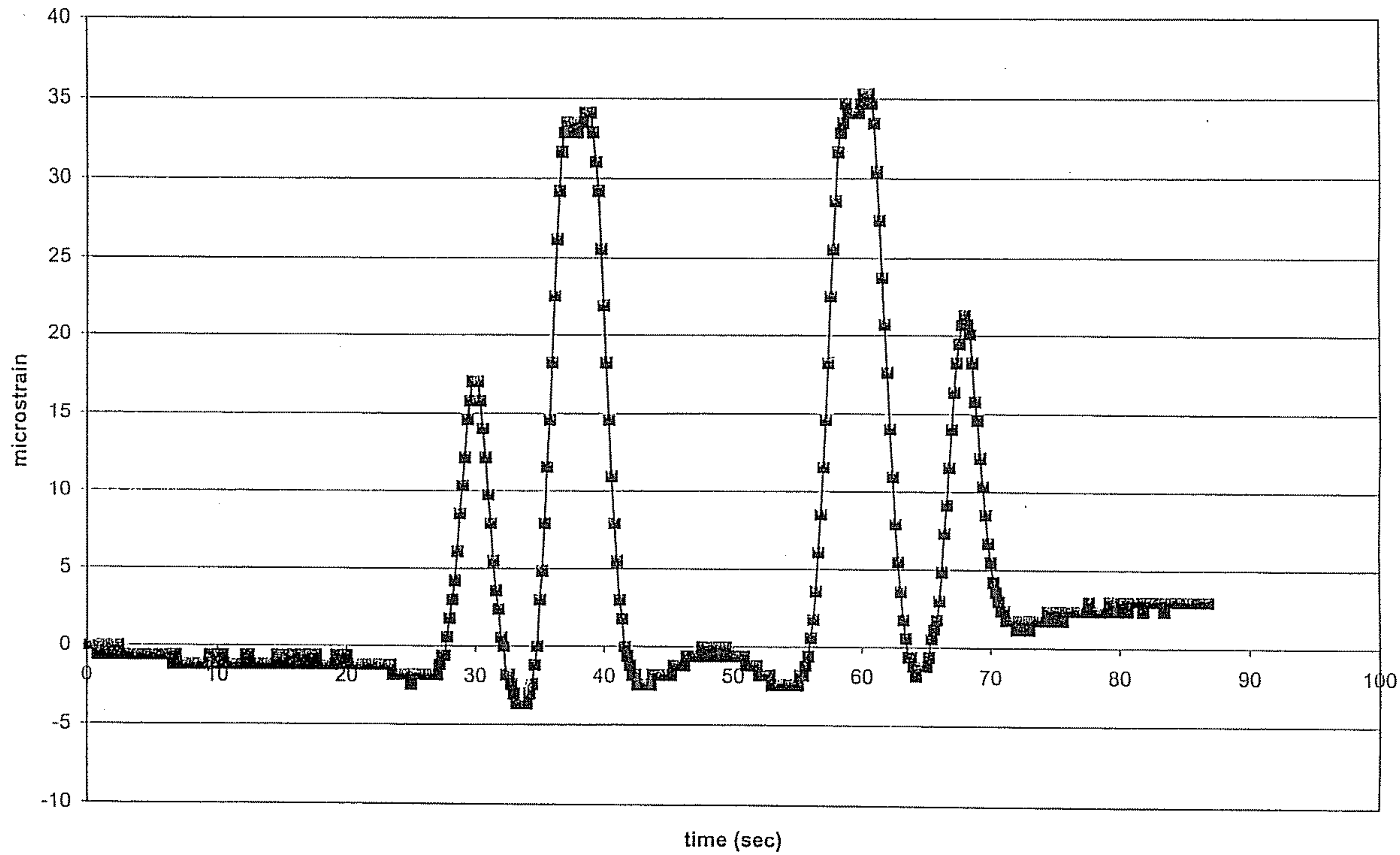
Time history (transducer 291) for pass 2 on northbound bridge 450



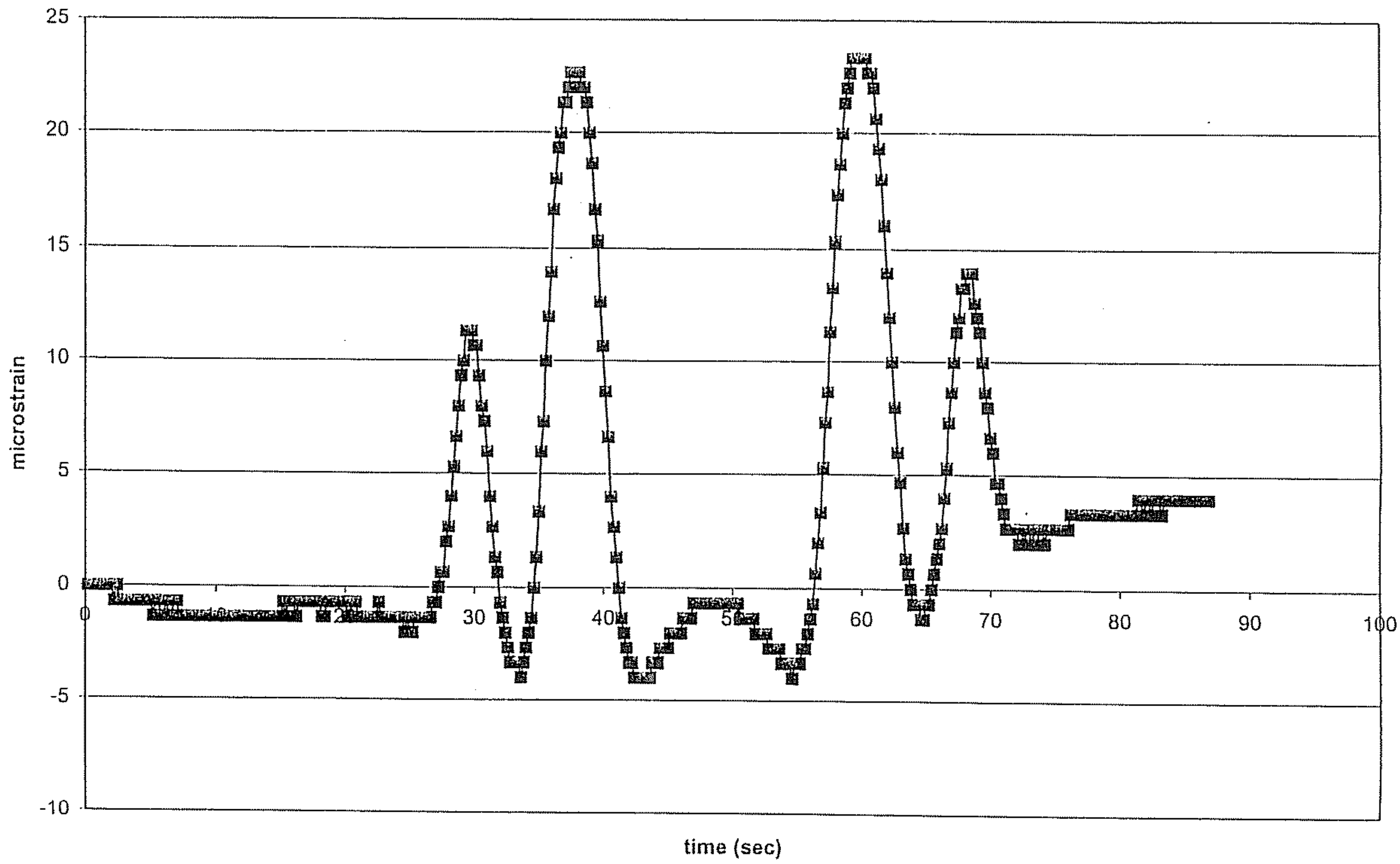
Time history (transducer 356) for pass 2 on northbound bridge 450



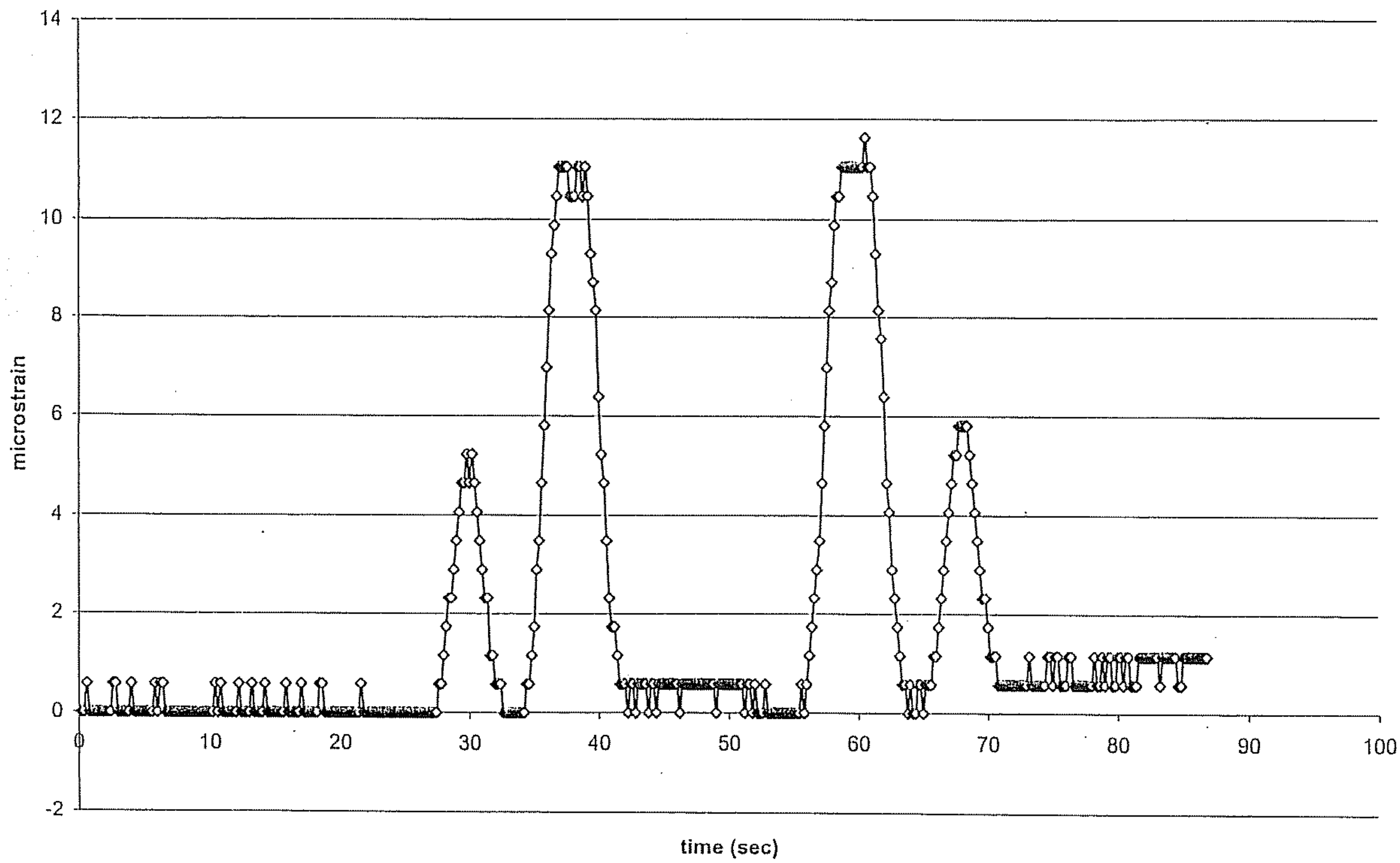
Time history (transducer 351) for pass 2 on northbound bridge 450



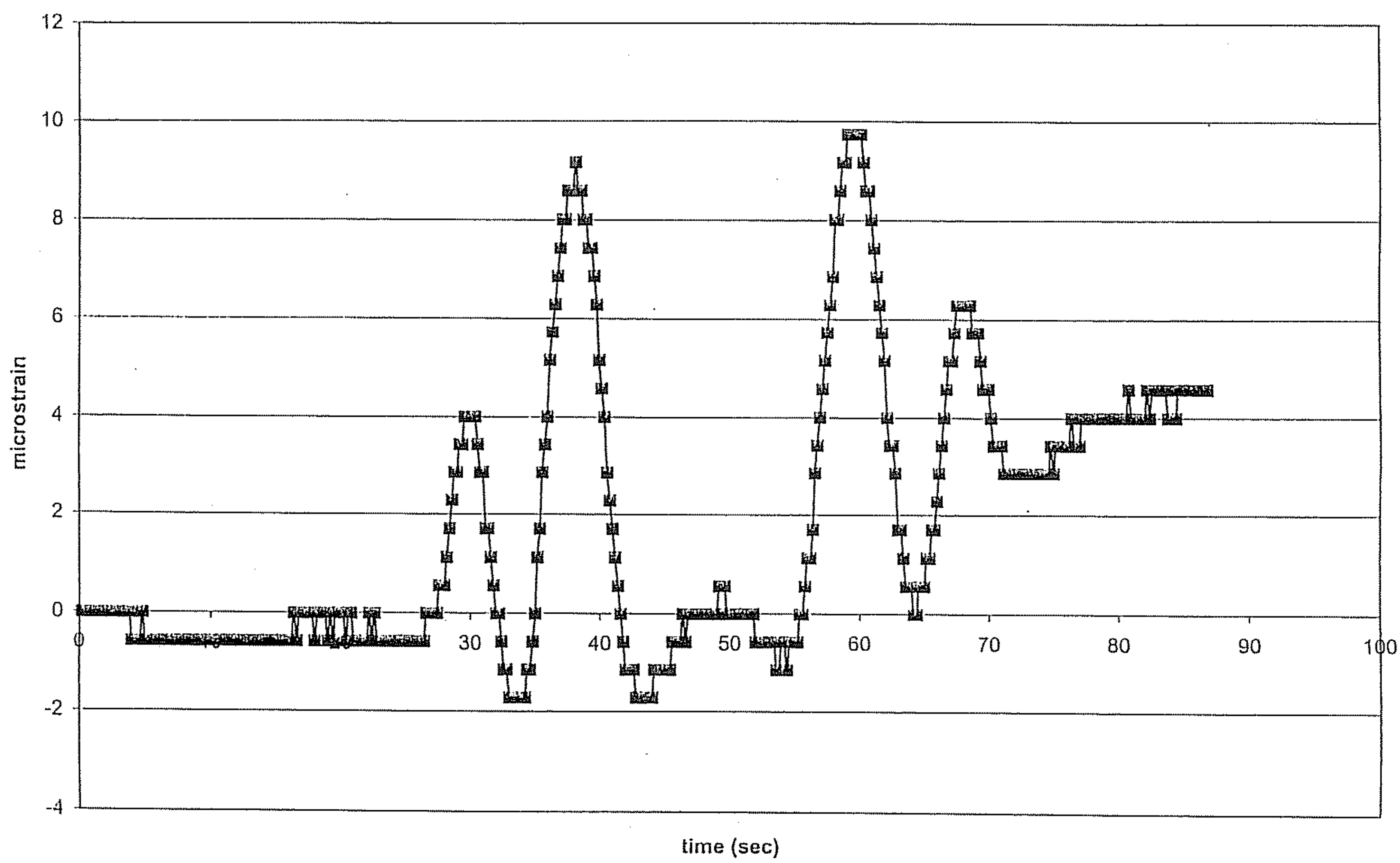
Time history (transducer 353) for pass 2 on northbound bridge 450



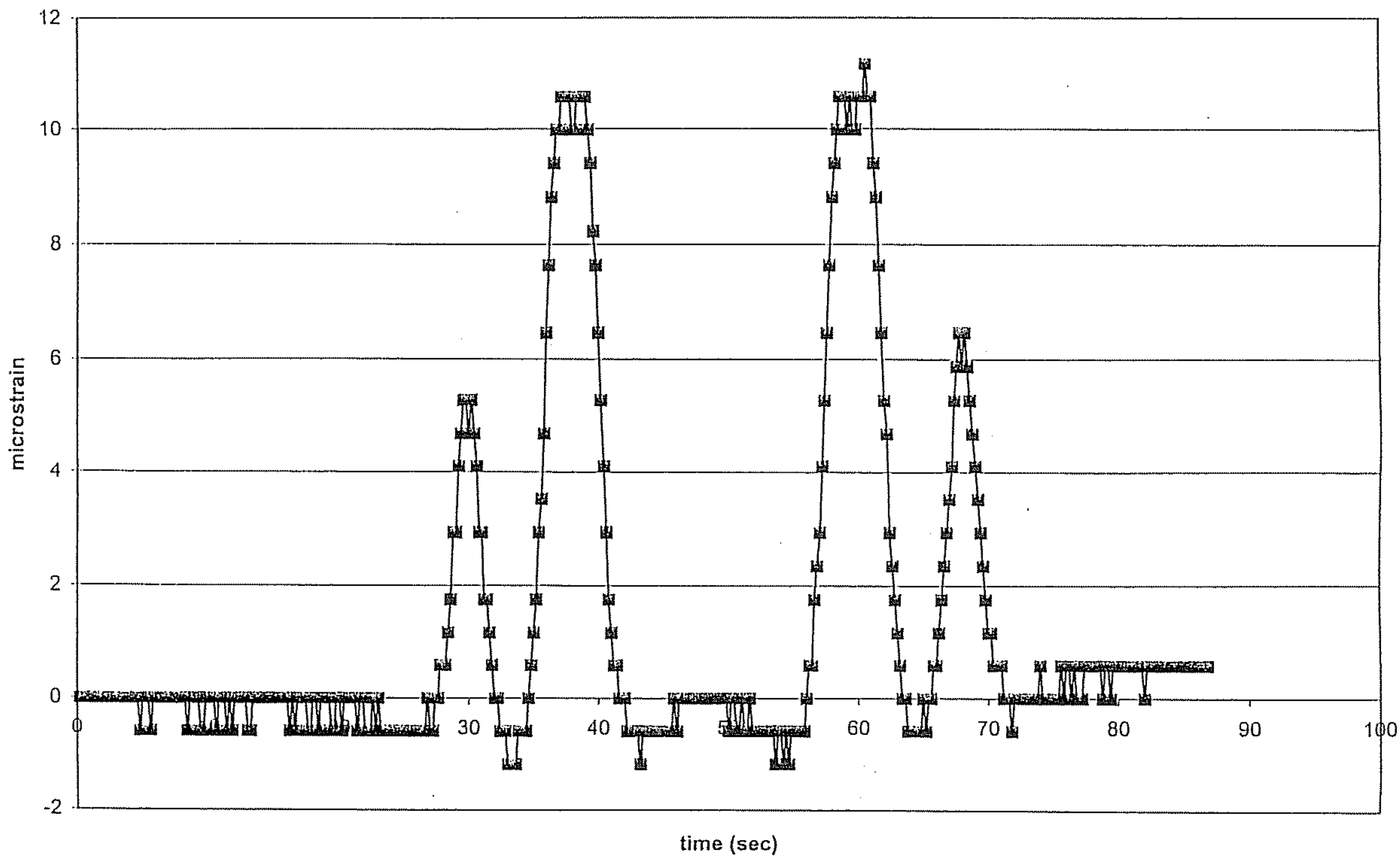
Time history (transducer 293) for pass 2 on northbound bridge 450



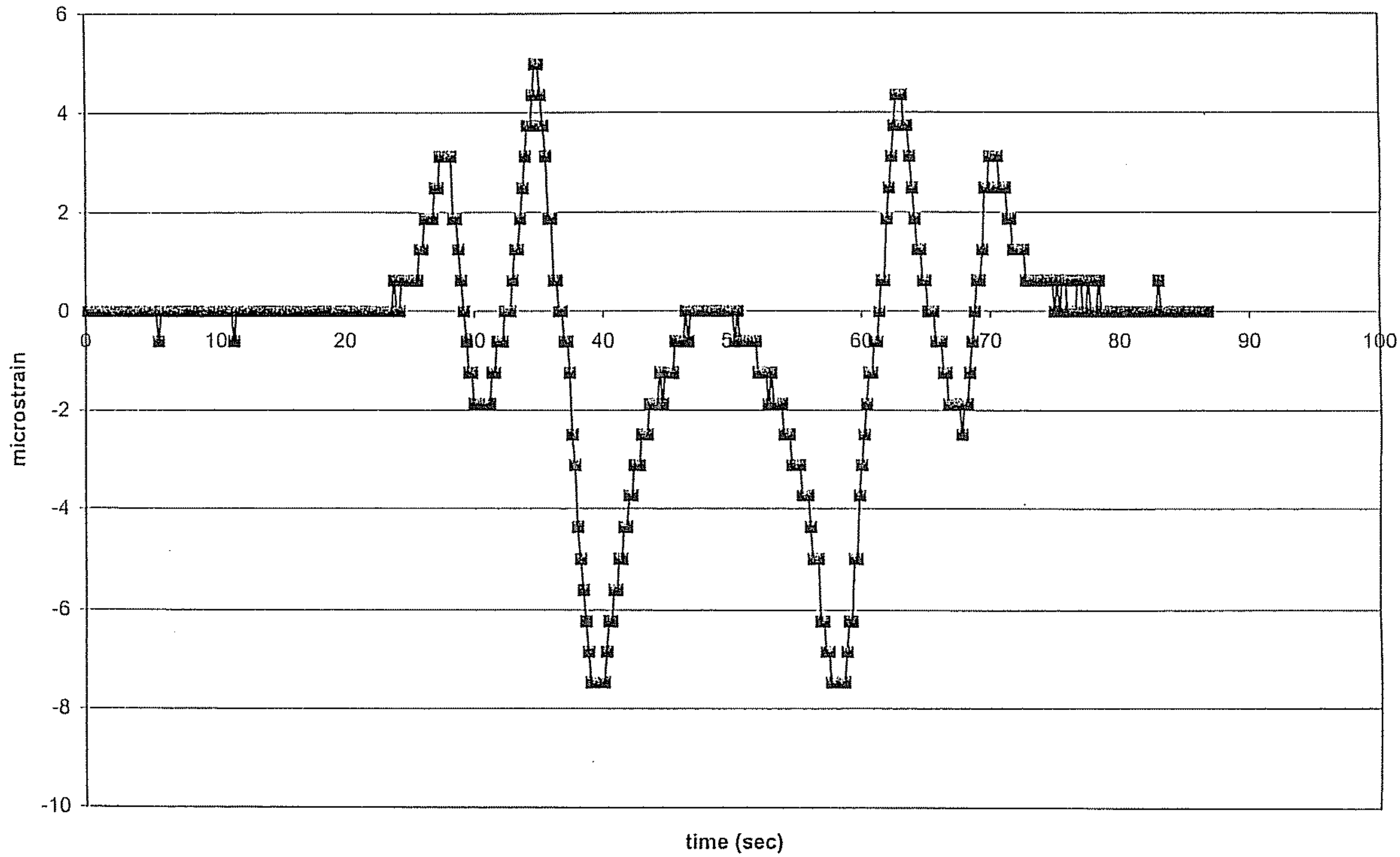
Time history (transducer 304) for pass 2 on northbound bridge 450



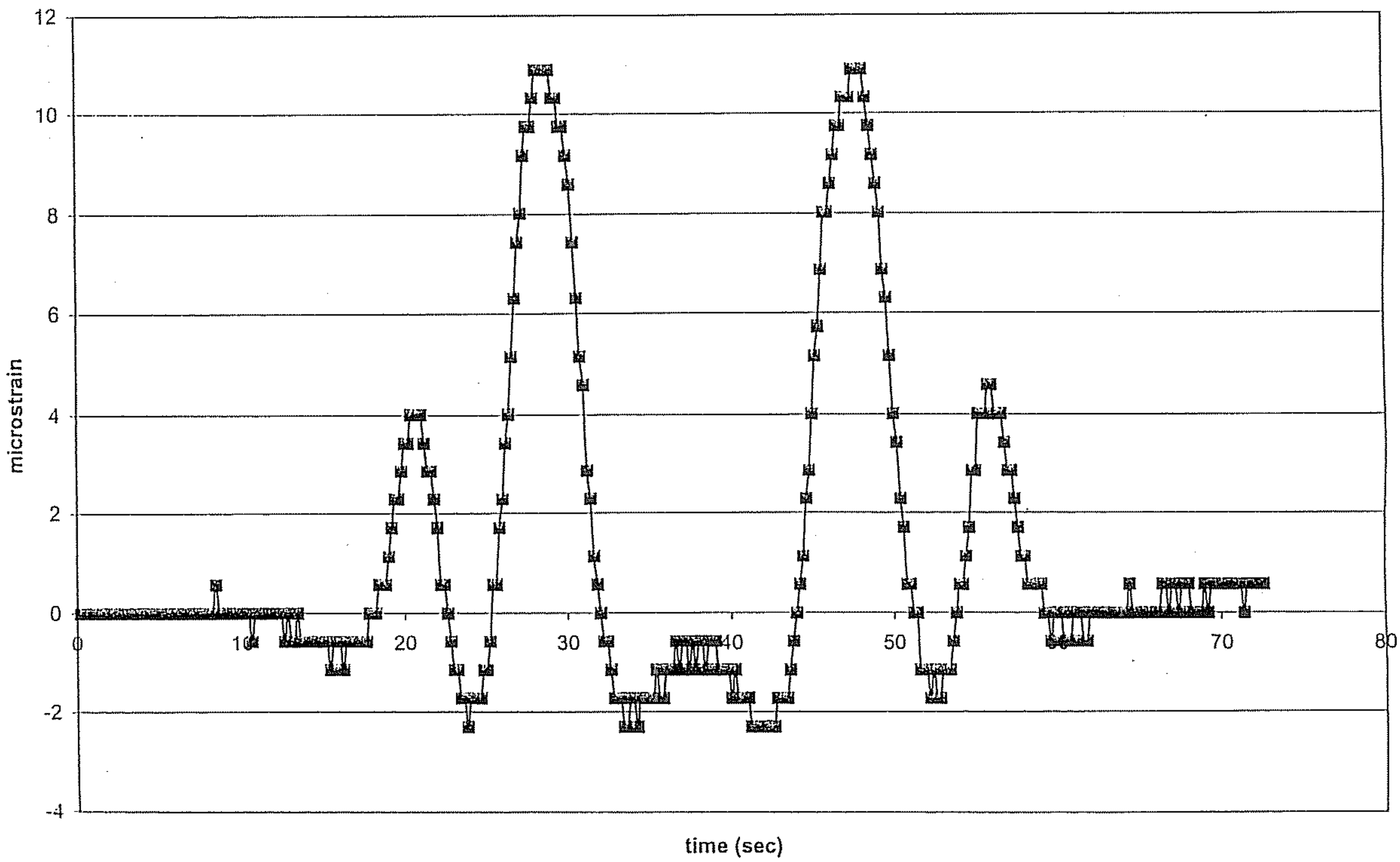
Time history (transducer 294) for pass 2 on northbound bridge 450

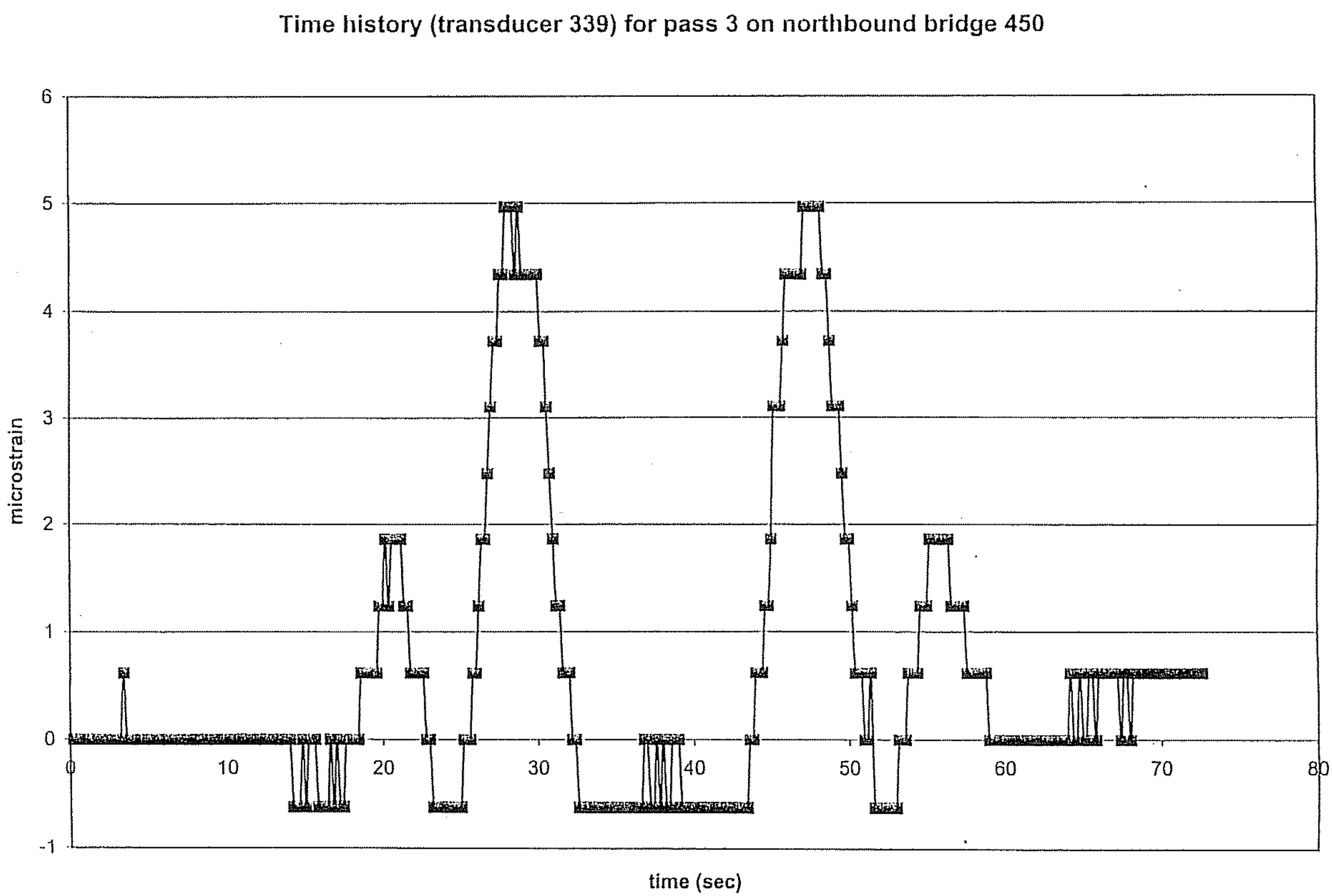


Time history (transducer 344) for pass 2 on northbound bridge 450

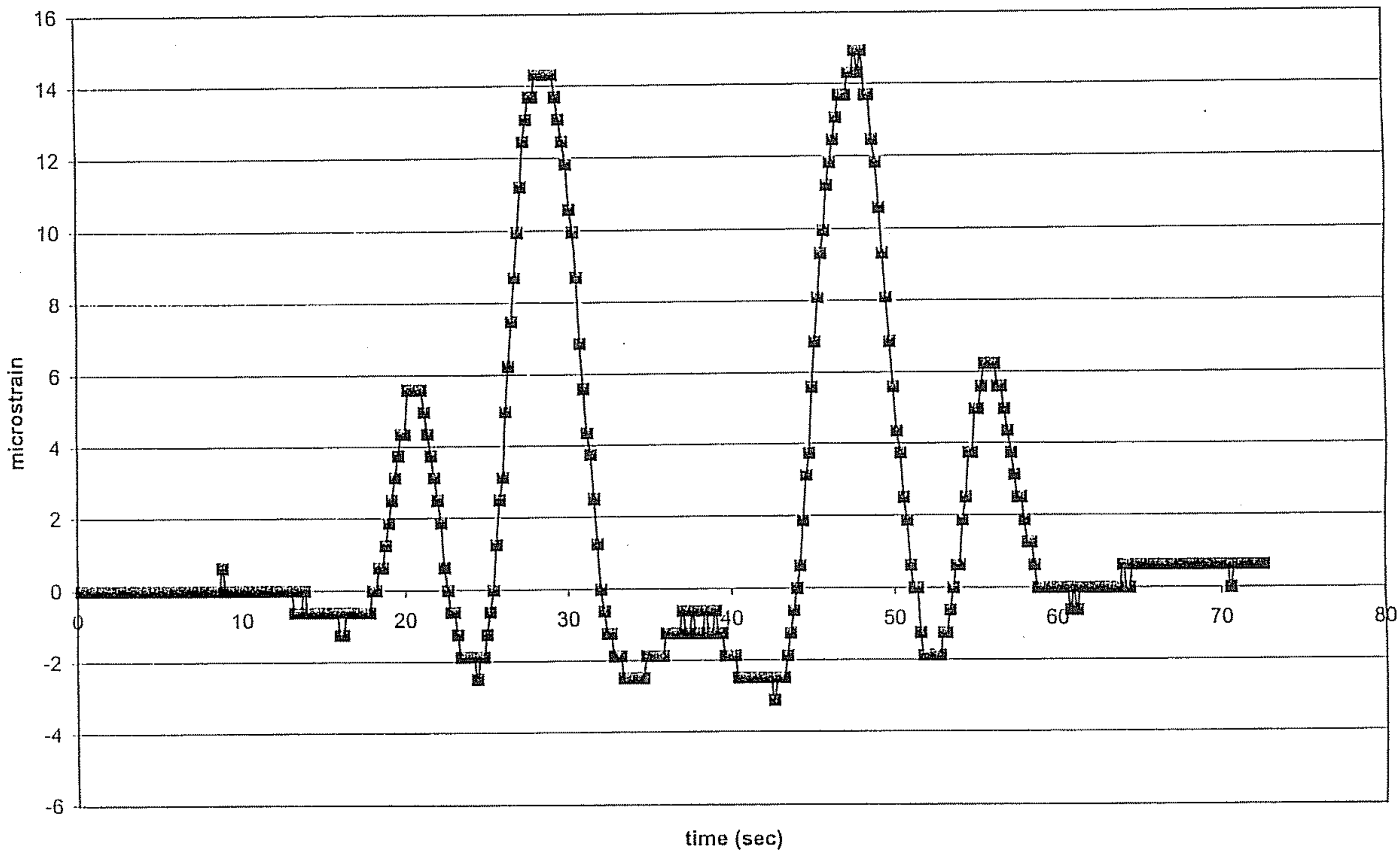


Time history (transducer 297) for pass 3 on northbound bridge 450

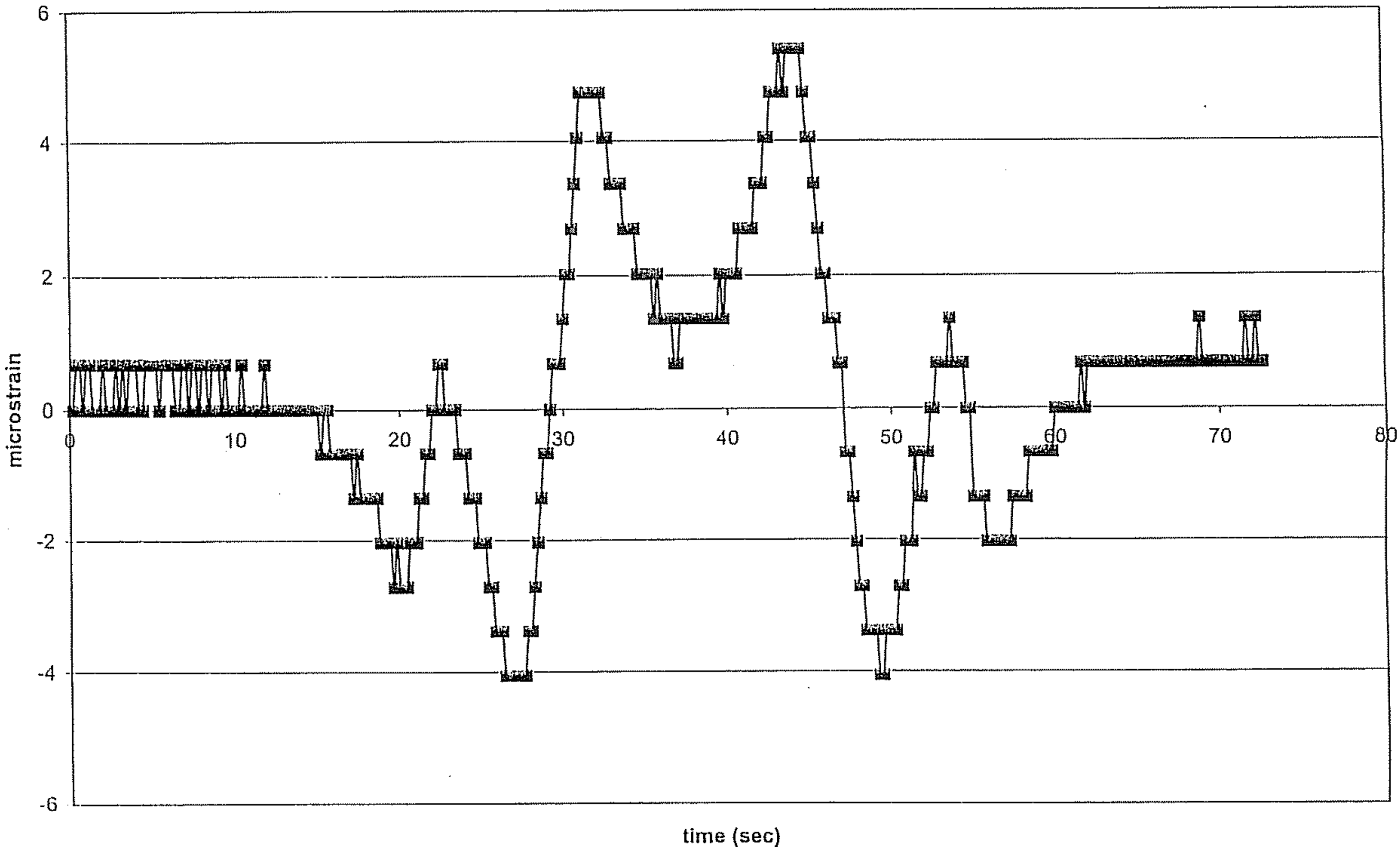



Time history (transducer 339) for pass 3 on northbound bridge 450
6
5
4
3
2
1
0
-1
microstrain
0
10
20
30
40
50
60
70
80
time (sec)

Time history (transducer 291) for pass 3 on northbound bridge 450



Time history (transducer 356) for pass 3 on northbound bridge 450



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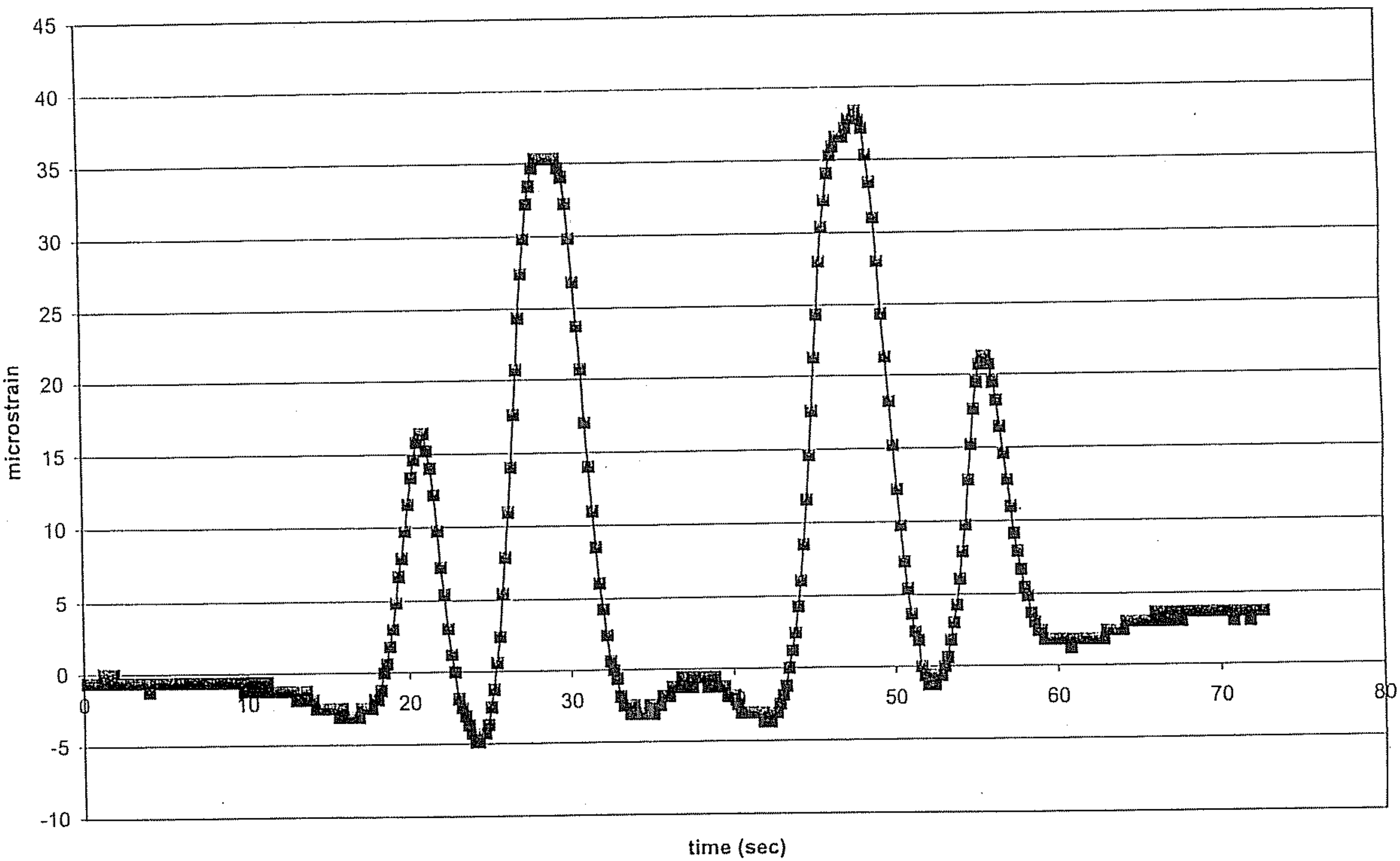
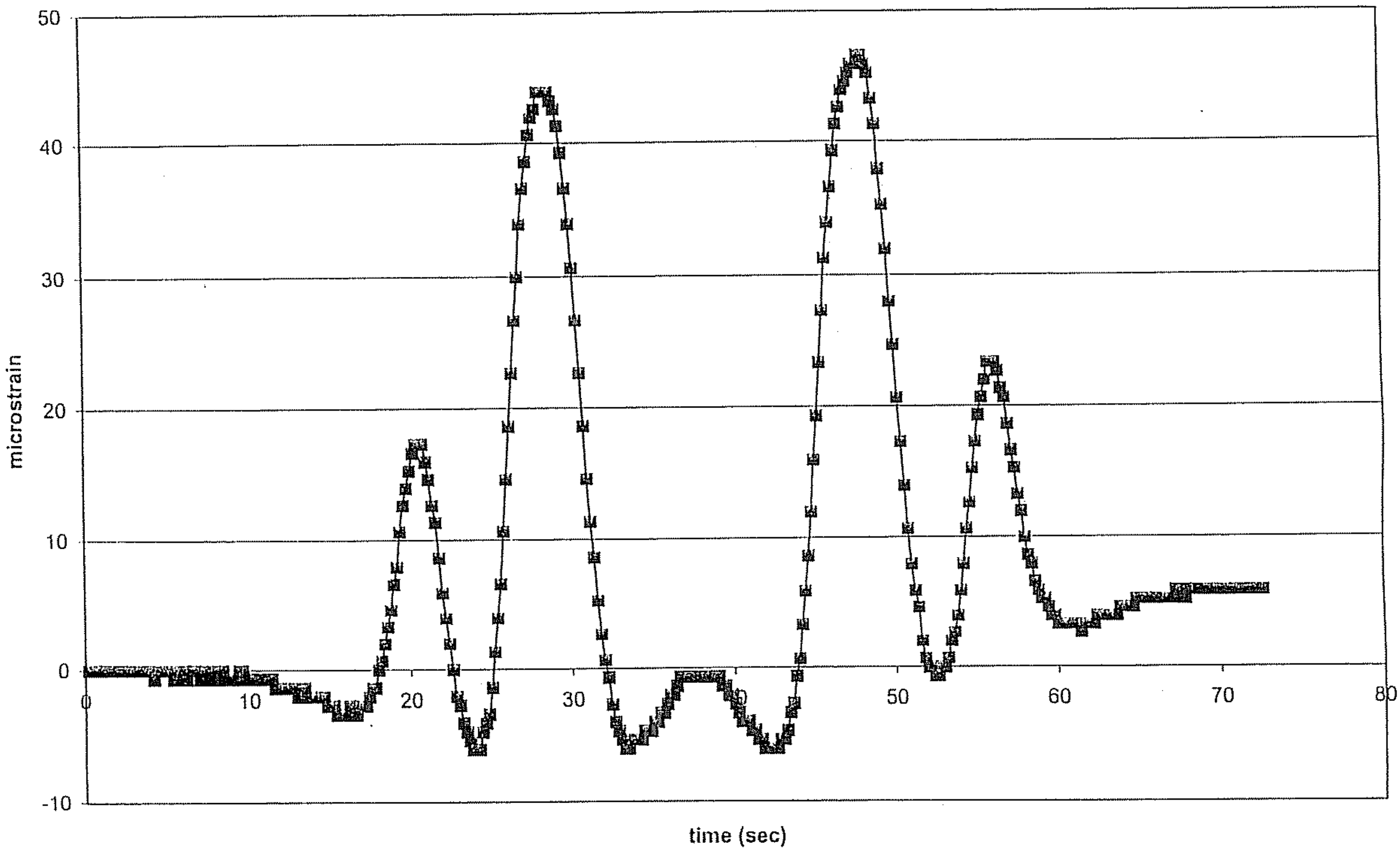


Time history (transducer 353) for pass 3 on northbound bridge 450



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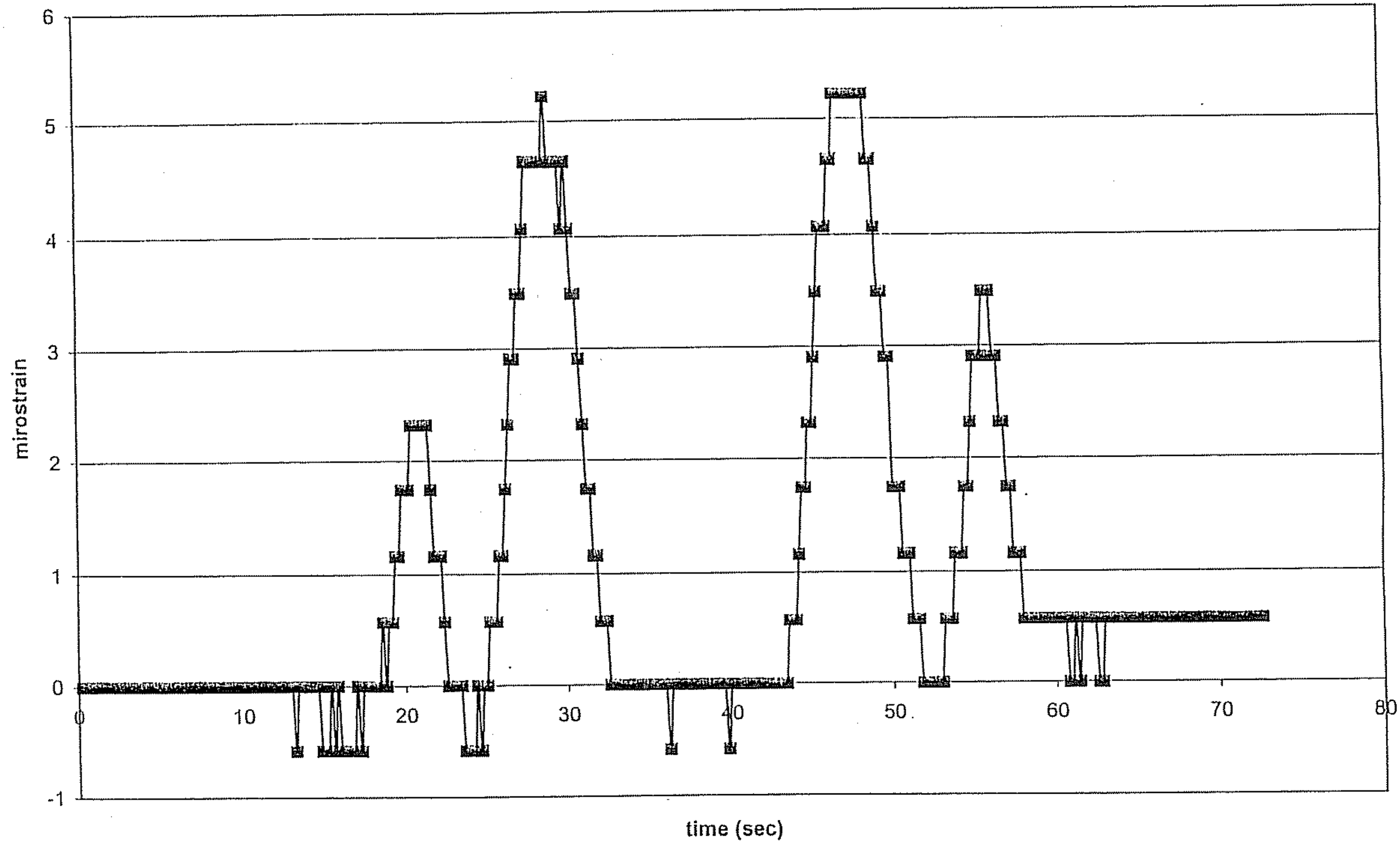
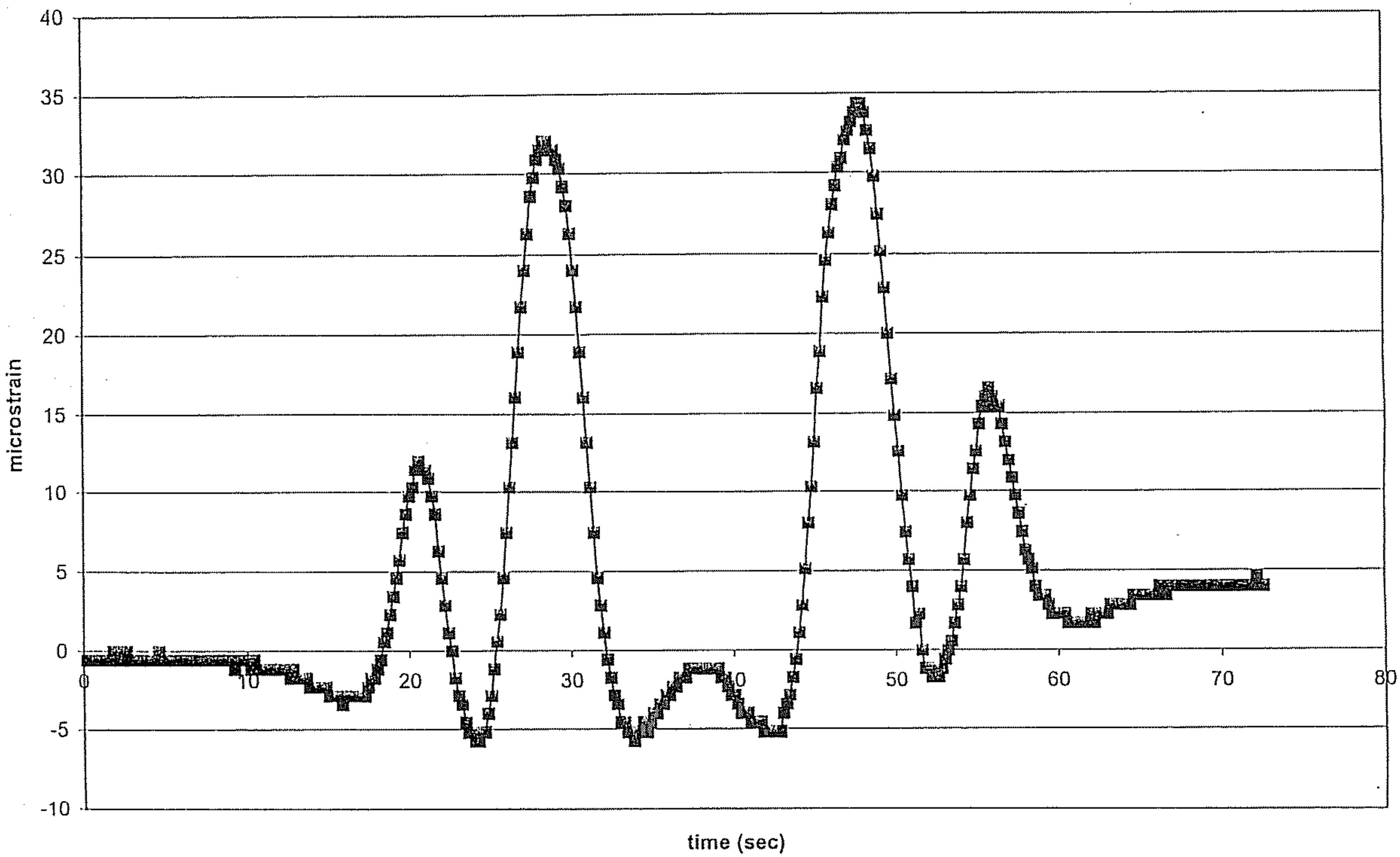
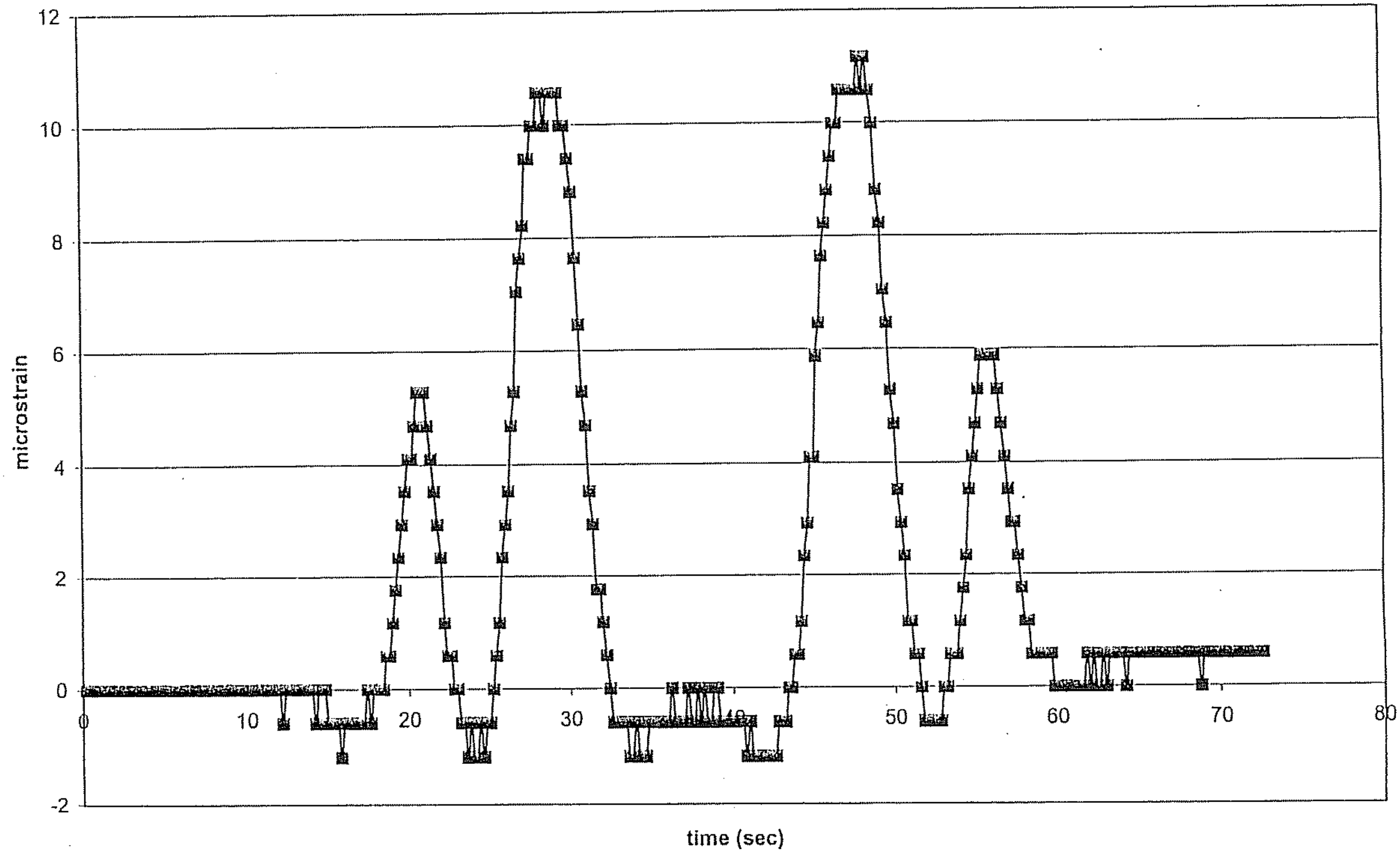


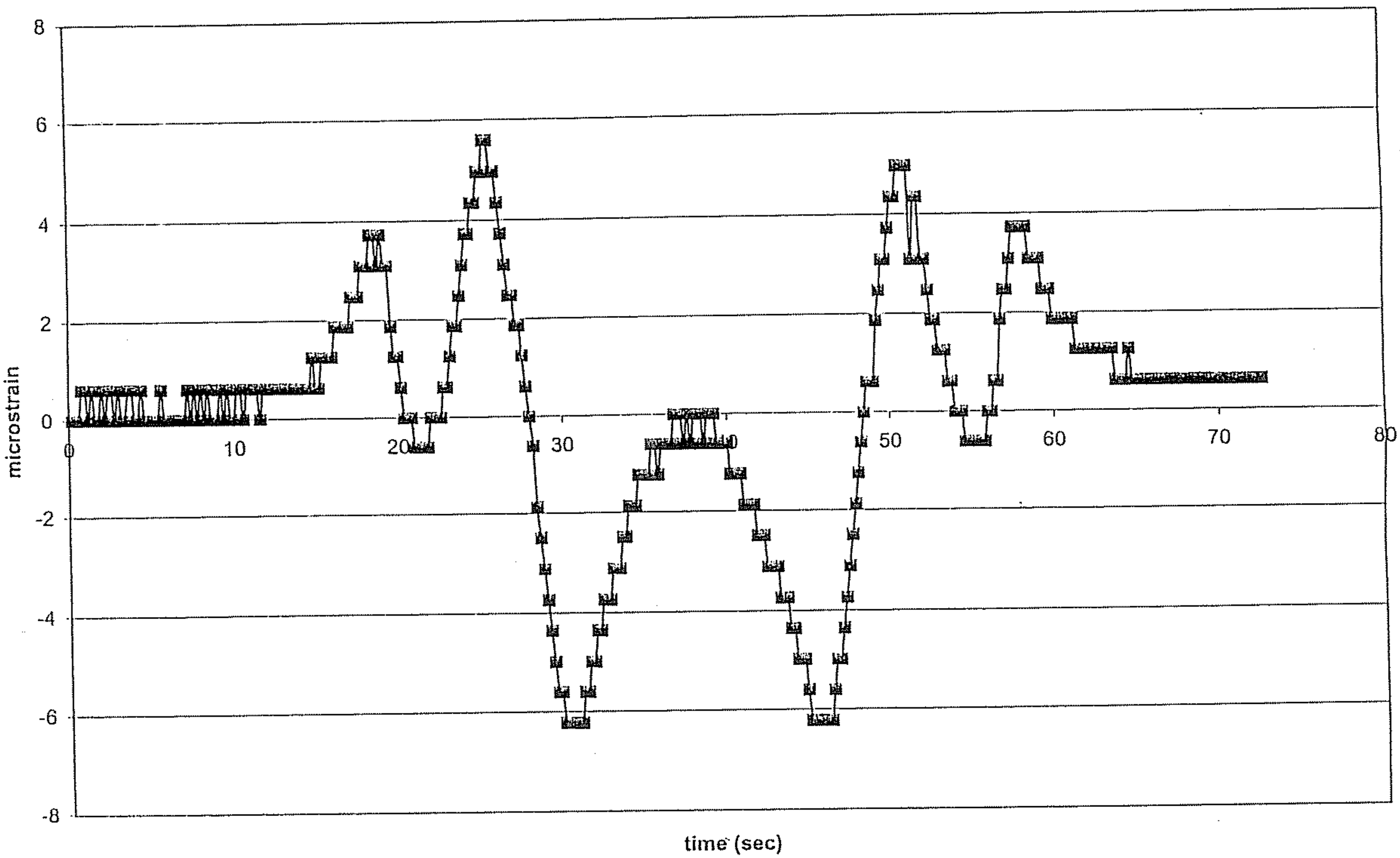
Time history (transducer 304) for pass 3 on northbound bridge 450



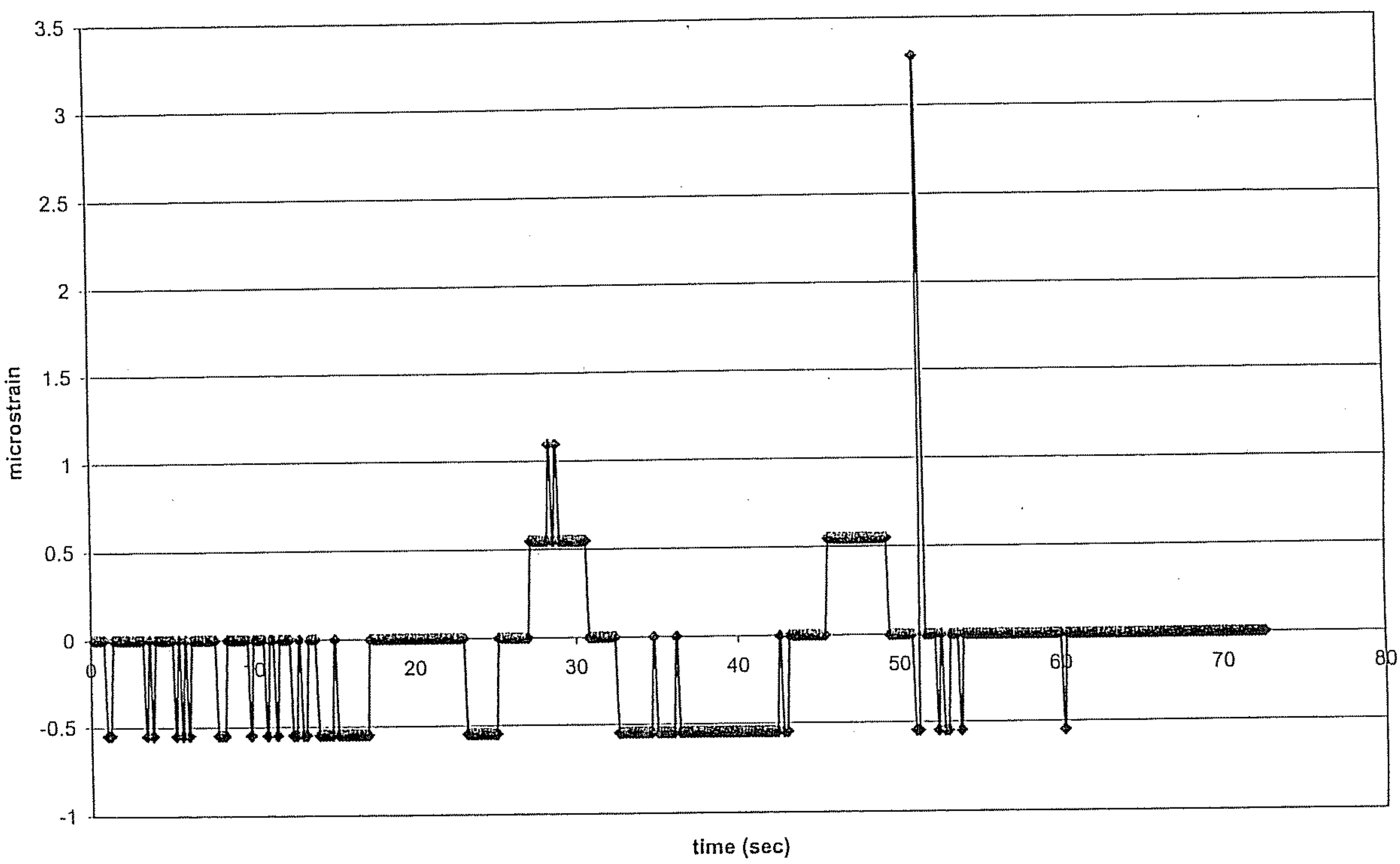
Time history (transducer 294) for pass 3 on northbound bridge 450

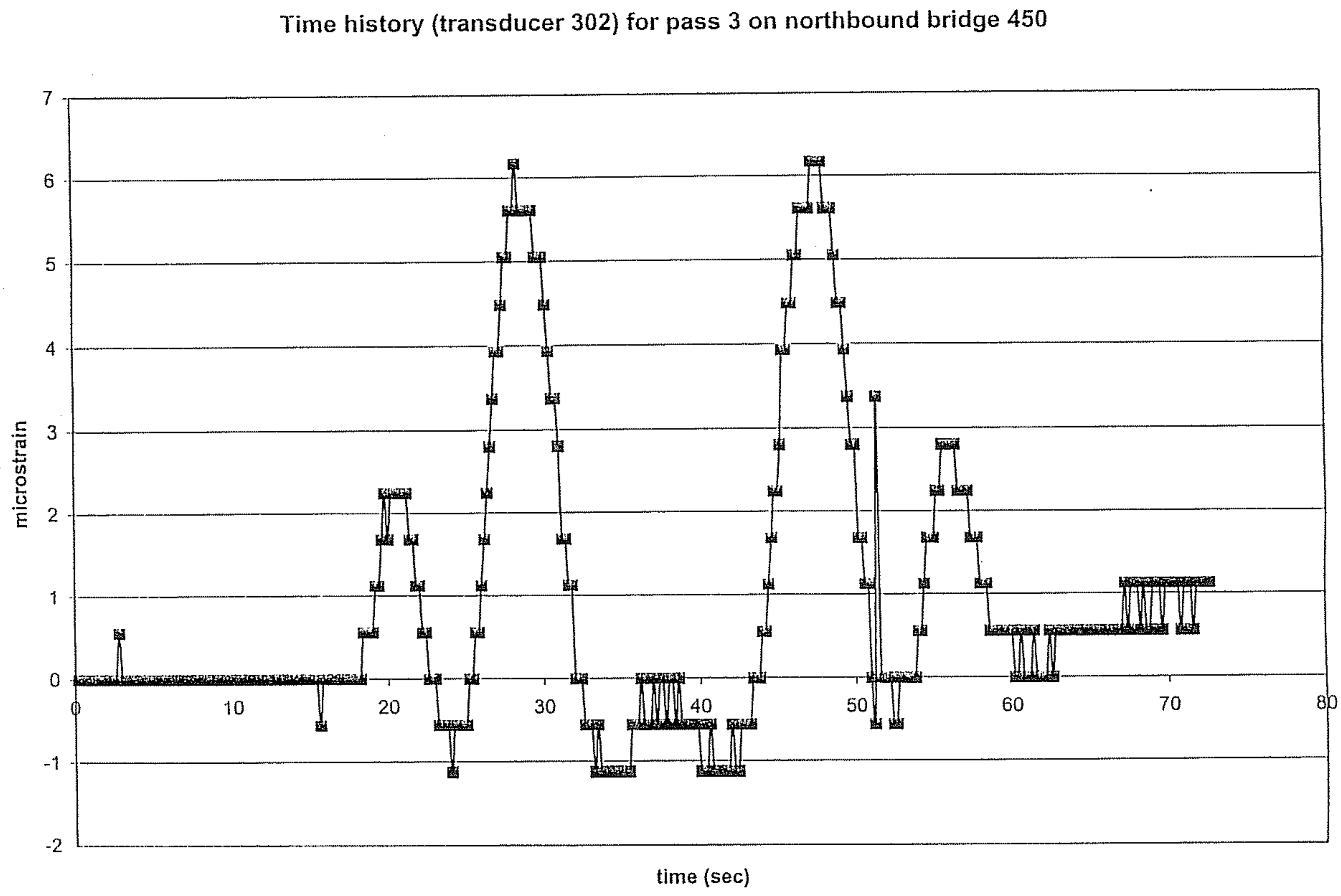


Time history (transducer 344) for pass 3 on northbound bridge 450



Time history (transducer 532) for pass 3 on northbound bridge 450




Time history (transducer 302) for pass 3 on northbound bridge 450
microstrain
7
6
5
4
3
2
1
0
-1
-2
0
10
20
30
40
50
60
70
80
time (sec)

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