New Hampshire DOT

Research Record

Barron Mountain
Rock Reinforcement Evaluation – Phase I
I-93, Woodstock, New Hampshire

Prepared by McMahon & Mann Consulting Engineers, P.C., Buffalo, New York for the New Hampshire Department of Transportation, in cooperation with the U.S. Department of Transportation, Federal Highway Administration

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12. Sponsoring Agency Name and Address:
NEW HAMPSHIRE DEPARTMENT OF TRANSPORTATION
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CONCORD, NH 03302-0483

16. Abstract

The Barron Mountain rock cut is located along the northbound lane of I-93 that was constructed in 1972 in Woodstock, NH. In response to a rockslide that occurred during construction, approximately 250 rock reinforcements consisting of grouted, high strength steel rods were installed along the 800 ft. long, 130 ft. high cut. The NHDOT is concerned with the longevity of the system because more than half of the generally accepted 50-year design life has passed. The NHDOT engaged McMahon & Mann Consulting Engineers, P.C. (MMCE) to perform an initial condition assessment, estimate the remaining service-life of the rock reinforcement, and make recommendations for more detailed condition assessment and future monitoring including invasive testing. MMCE followed the recommended practice described in NCHRP Report 477, "*Recommended Practice for Evaluation of Metal-Tensioned Systems In Geotechnical Applications.*" To evaluate the remaining service-life of rock anchors and rock bolts, environmental factors that may influence corrosion were assessed, and reinforcements were selected for nondestructive testing and condition assessment. Four nondestructive tests (NDT's) were employed for condition assessment including measurement of half-cell potential, polarization current, impact and ultrasonic testing. Twenty-two rock bolts and twenty rock tendons were included in the sample population for NDT. Based on the analysis of weathered rock and groundwater samples obtained from the site, the corrosiveness of the subsurface environment is considered to be between average and corrosive. The estimated remaining service-life for rock bolts and tendons is 14 and 20 years, respectively. Results from NDT tend to support the service-life estimate. The condition assessment is based on results from the NDT and from visual observations made by MMCE during their fieldwork. The assessment includes judgment relative to loss of pre-stress, grout quality, the occurrence of corrosion, and apparent distress to the reinforcement cross section. In general, results indicate that tendon reinforcements are in better condition relative to rock bolt reinforcements. Results from NDT indicate that 30% of the rock bolts may have suffered a loss of pre-stress, 85% have "possibly" experienced corrosion, and 18% have apparent distress in terms of possible loss of cross section from corrosion, or a bend or kink along the bar from deformation of the rockmass. Rock bolts and tendons that have apparently suffered corrosion, or are possibly distressed, are primarily located in a distinctly identifiable area of the site near Station 1775+25.

Summary of Condition Assessment at Barron Mountain

MMCE recommends that the results of NDT be verified by further, more invasive, testing on selected reinforcements; and that reinforcement condition continue to be monitored at regular ten year intervals. Verification of results from NDT should be performed as a second phase to this project. The proposal for Phase II of this research includes invasive testing of selected rock bolts and tendons to verify results from NDT and service-life estimates performed in Phase I. Proposed invasive testing includes lift-off tests; and physical, chemical and metallurgical testing on steel and grout samples retrieved from exhumed reinforcements. Replacement bolts must be installed prior to invasive testing of the reinforcements. MMCE recommends that testing included in Phase II focus mainly on rock bolts within the area of the site surrounding Station 1775+25.

17. Key Words: BOLTS, TENDONS, ANCHORS, REINFORCEMENTS, ROCK, I-93, WOODSTOCK, SERVICE LIFE, NDT

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February 18, 2004

Project No. 03-024

Glenn E. Roberts, P.E.
Chief of Research
New Hampshire DOT, Bureau of Materials and Research
P.O. Box 483
11 Stickney Avenue
Concord, NH 03302-0483

RE: BARRON MOUNTAIN ROCK REINFORCEMENT EVALUATION – PART A NHDOT PROJECT NO. 13733L

Dear Mr. Roberts:

In accordance with our agreement with the State of New Hampshire, dated August 13, 2003, please find the interim report for the above referenced project. An executive summary is included followed by the main body of the report, and Appendix X, which is a scope of work and cost estimate for Phase II of the project. Phase II includes invasive testing needed for verification of the results from nondestructive testing performed during Phase I. Copies of Appendices I through IX of the report are submitted under separate cover.

Please call if you have any questions or comments regarding the report, or the proposal for Phase II.

Very truly yours,

McMahon & Mann Consulting Engineers, P.C.

Kenneth L. Fishman, Ph.D., P.E.
Principal

Telephone: 716-834-8932 • Fax: 716-834-8934

EXECUTIVE SUMMARY

The Barron Mountain rock cut is located along the northbound lane of I-93 that was constructed in 1972, approximately 60 miles north of Concord, NH. In response to a rockslide that occurred during construction, approximately 250 rock reinforcements consisting of grouted, high strength steel rods were installed along the 800 ft. long, 130 ft. high cut.

The New Hampshire Department of Transportation (NHDOT) is concerned with the longevity of the system because more than half of the generally accepted 50-year design life has passed. The NHDOT engaged McMahon & Mann Consulting Engineers, P.C. (MMCE) to perform an initial condition assessment, estimate the remaining service-life of the rock reinforcement, and make recommendations for more detailed condition assessment and future monitoring including invasive testing. This report describes MMCE's approach, and the findings, conclusions and recommendations resulting from the study.

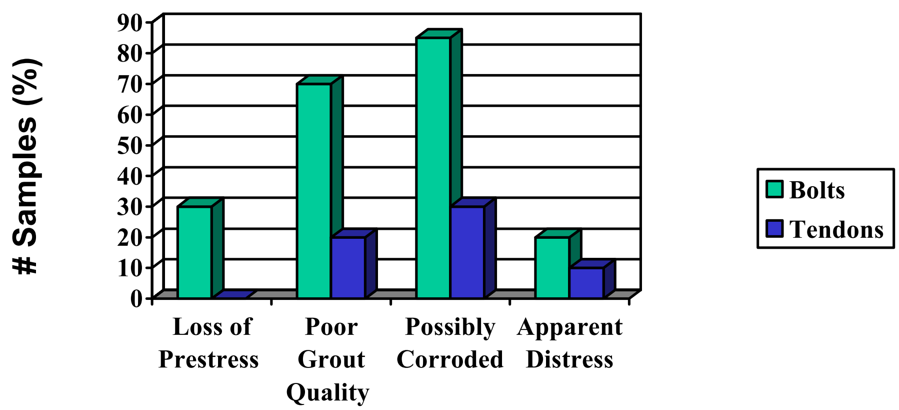
Two basic types of reinforcements are installed at the site including (1) unstressed, full length grouted, high strength steel tendons designed to act as tension elements against down and outward movement of rock and (2) stressed rock bolts designed to improve stability of surface blocks and improve the overall integrity of the rock mass. Due to different installation details, MMCE considered rock bolt and tendon reinforcements separately for the purpose of condition assessment.

MMCE followed the recommended practice described in NCHRP Report 477, "*Recommended Practice for Evaluation of Metal-Tensioned Systems in Geotechnical Applications.*" To evaluate the remaining service-life of rock reinforcement at the Barron Mountain site, environmental factors that may influence corrosion are assessed, and reinforcements are selected for nondestructive testing and condition assessment. Four nondestructive tests (NDT's) are employed for condition assessment including measurement of half-cell potential, polarization current, impact and ultrasonic testing. Twenty-two rock bolts and twenty rock tendons were included in the sample population for NDT.

Based on the analysis of weathered rock and groundwater samples obtained from the site, the corrosiveness of the subsurface environment is considered to be between average and corrosive. The estimated remaining service-life for rock bolts and tendons is 14 and 20 years, respectively. Results from NDT tend to support the service-life estimate.

The condition assessment is based on results from the NDT and from visual observations made by MMCE during their fieldwork. The assessment includes judgment relative to loss of prestress, grout quality, the occurrence of corrosion, and apparent distress to the reinforcement cross section. The condition assessments of

rock bolts and tendons are summarized in the bar chart shown below. In general, results indicate that tendon reinforcements are in better condition relative to rock bolt reinforcements. Results from NDT indicate that 30% of the rock bolts may have suffered a loss of prestress, 85% have "possibly" experienced corrosion, and 18% have apparent distress in terms of possible loss of cross section from corrosion, or a bend or kink along the bar from deformation of the rockmass. Rock bolts and tendons that have apparently suffered corrosion, or are possibly distressed, are primarily located in a distinctly identifiable area of the site near Station 1775+25.



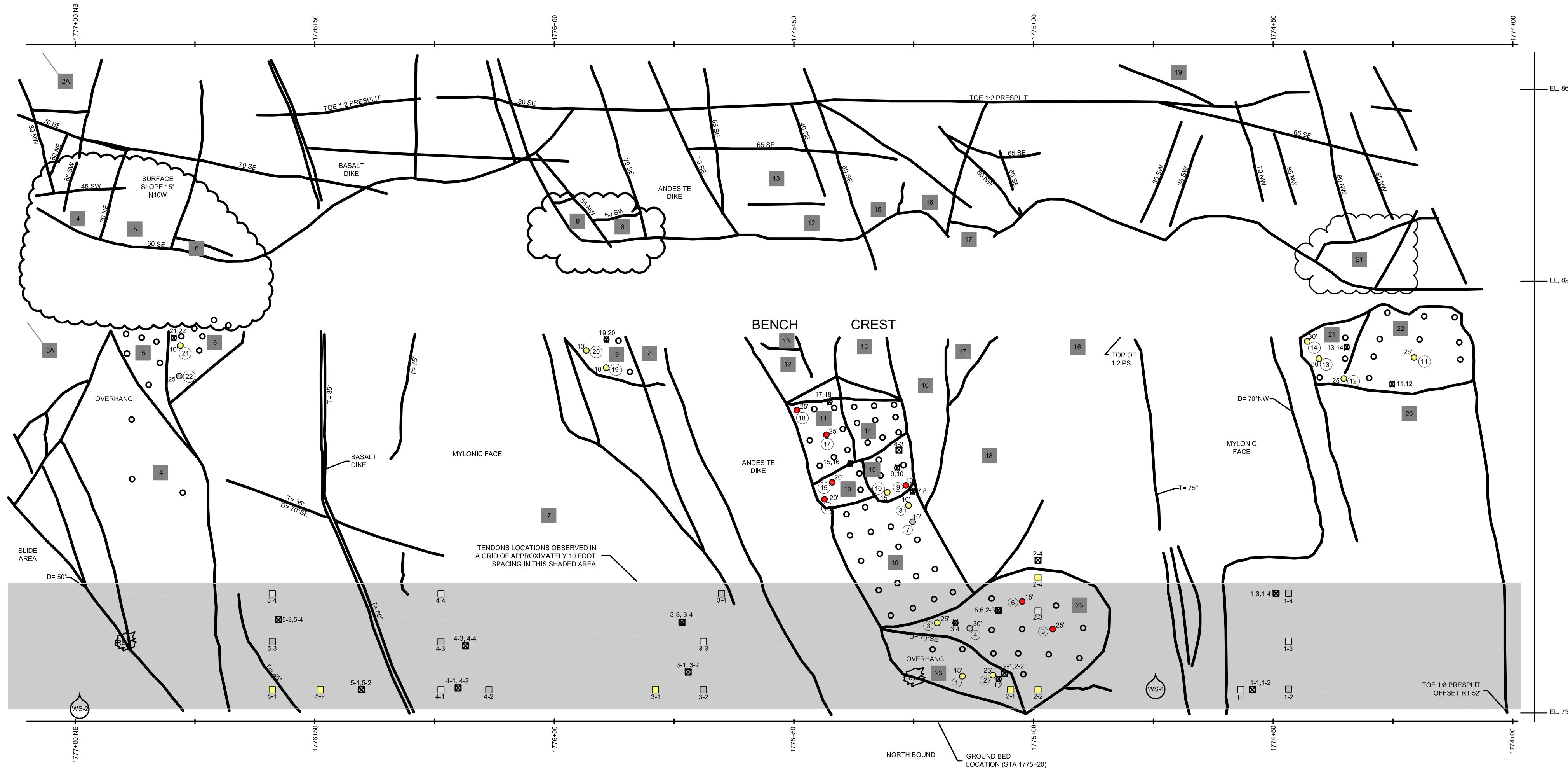
Summary of Condition Assessment at Barron Mountain

MMCE recommends that the results of NDT be verified by further, more invasive, testing on selected reinforcements; and that the NHDOT continue to monitor reinforcement condition at regular ten year intervals. Verification of results from NDT should be performed as a second phase to this project.

The proposal for Phase II of this research includes invasive testing of selected rock bolts and tendons to verify results from NDT and service-life estimates performed in Phase I. Invasive testing shall include lift-off tests; and physical, chemical and metallurgical testing on steel and grout samples retrieved from exhumed reinforcements. Replacement bolts must be installed prior to invasive testing of the reinforcements. MMCE recommends that testing included in Phase II focus mainly on rock bolts within the area of the site surrounding Station 1775+25.

MMCE estimates the cost for Phase II to be approximately $122,000.00 and this includes approximately three weeks of fieldwork, and the cost of a contractor to test and exhume existing reinforcements and install replacements.

TABLE OF CONTENTS



ROCK BOLT TEST LOCATIONS

ROCK BOLT #	APPROX. STATION	BLOCK #	LENGTH[1] (FEET)
1	1775+15	23	15
2	1775+10	23	25
3	1775+20	23	25
4	1775+15	23	30
5	1775+00	23	25
6	1775+05	23	15
7	1775+25	10	10
8	1775+25	10	10
9	1775+25	10	10
10	1775+30	10	15
11	1774+15	22	25
12	1774+30	22	25
13	1774+40	21	30
14	1774+40	21	30
15	1775+45	10	20
16	1775+45	10	20
17	1775+45	11	25
18	1775+50	11	25
19	1775+90	9	10
20	1775+95	9	10
21	1776+75	6	10
22	1776+75	6	25

[1] THE LENGTH IS TAKEN FROM THE GEOLOGISTS' FIELD NOTES MADE DURING CONSTRUCTION. AS-BUILT DETAILS MAY VARY FROM THESE NOTES.

TENDON TEST STATIONS

TENDON TEST STATION #	STATIONING
1	1774+50
2	1775+00
3	1775+70
4	1776+25
5	1776+50

LEGEND

- 15' — Reported Bolt Length
- Bolt Location
- 6 — Bolt Designation
- Bolt Half-Cell Location
- 3,4 — Bolts with which Half-Cell was Tested
- Tendon Location
- 4-1 — Tendon Designation
- Tendon Half-Cell Location
- 4-1, 4-2 — Tendons with which Half-Cell was Tested
- WS-1 — Groundwater Sample Location
- RS-1 — Weathered Rock Sample Location
- 18 — Block Location and Designation
- Clouded areas designate reported bolt locations that were not observed or included in sampling for non-destructive testing

Bolt Corrosion Potential
- Not Likely
- Likely
- Very Likely

Tendon Corrosion Potential
- Not Likely
- Uncertain
- Likely

NOTE:
UNAUTHORIZED ALTERATION OR ADDITION TO ANY SURVEY, DRAWING, DESIGN, SPECIFICATION, PLAN, OR REPORT IS A VIOLATION OF SECTION 7209 PROVISION 2 OF THE NEW YORK STATE EDUCATION LAW.

NOTES:

1.) THIS DRAWING WAS ADAPTED FROM A FIGURE BY HALEY & ALDRICH, INC. TITLED "I-93, P-7889-F SITE CONDITIONS" AND DATED JANUARY 1974.

2.) ALL LOCATIONS SHOWN WERE FIELD MEASURED AND SHOULD BE CONSIDERED APPROXIMATE.

SCALE: 1" = 10'

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BARRON MOUNTAIN SITE ELEVATION VIEW

WOODSTOCK NEW HAMPSHIRE

DRAWN BY: T.W.S.
CHECKED BY: K.L.F.
SCALE: 1" = 10'
DATE: FEBRUARY 2004
JOB NO. 03024
FIGURE 5
DWG. NO. 03024-001
REVISION NUMBER - 0

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

INTRODUCTION

Background

In 1972, the New Hampshire Department of Transportation (NHDOT) installed high-strength steel rods to stabilize a rock cut along the northbound lane of I-93, approximately 60 miles north of Concord, NH. As shown in Figures 1 and 2, the site is located on the east slope of the Pemigewassett River Valley in Woodstock, NH. This section of I-93 involved side-hill rock excavation for a distance of approximately 600 ft where the northbound lane traverses the west slope of Barron Mountain.

In response to a rockslide that occurred during construction, rock reinforcements, including rock bolts and rock tendons, were installed to stabilize the cut. As described by Haley & Aldrich (1973 a and b), long rock tendons were installed to counteract sliding along the anticipated sliding failure plane. Rock bolts were installed to keep the rock mass intact; to preserve the full gravity effect of the rock bench used to maintain global stability, and to prevent minor rock falls onto the highway. The rock reinforcement system has performed well to date and no major rock slides or rock falls have occurred in this area subsequent to construction.

The estimated design life of unprotected rock reinforcement systems is approximately 50 years (Kendorski, 2003). The New Hampshire Department of Transportation (NHDOT) is concerned with the longevity of the system given half the anticipated design life has passed. The NHDOT engaged McMahon & Mann Consulting Engineers, P.C. (MMCE) to perform an initial condition assessment, estimate the remaining service-life of the rock reinforcement, and make recommendations for more detailed condition assessment and future monitoring including invasive testing of the rock reinforcement at the site.

Scope of Services

To evaluate the remaining service life of rock reinforcement at the Barron Mountain site, environmental factors that may influence corrosion are assessed, and reinforcements are selected for nondestructive testing and condition assessment. The following engineering services were performed by MMCE in support of the project objectives as described in our agreement with NHDOT dated August 13, 2003. The study is performed with cooperation from NHDOT. The NHDOT contributions are identified within each task description and in the sections describing assessment of corrosion potential and assessment of rock reinforcement.

1. MMCE made a reconnaissance visit to the site during August 25, 2003 to August 27, 2003. MMCE and the NHDOT evaluated site access and identified reinforcement locations. MMCE prepared a sampling plan for nondestructive testing (NDT), obtained information on rock bolts and rock anchor installations, and retrieved samples of weathered rock for testing. MMCE also identified locations for groundwater samples, which were later retrieved during the fieldwork described in Task 4. Stationing at the site was reestablished on the

basis of known site features that were shown on construction drawings provided by NHDOT. MMCE marked the approximate stationing at 50 feet intervals on the edge of the pavement with orange marking paint.

2. Samples of weathered rock were tested by NHDOT for moisture content, grain size distribution, pH, resistivity, and concentrations of chloride and sulfate ions. MMCE tested samples of groundwater for pH in the field; and NHDOT performed laboratory tests for pH, and concentration of magnesium, chloride, sulfate and ammonium ions.

3. MMCE estimated the remaining service life for the rock reinforcements using available mathematical models of service life (Romanoff, 1957; NCHRP, 2002), details of the reinforcements identified in Task 1, and results from testing samples of weathered rock and groundwater as described in Task 2.

4. MMCE traveled to the site and performed condition assessment of the rock reinforcements during September 8, 2003 to September 17, 2003. MMCE observed conditions at the exposed ends of the rock reinforcements and performed NDT on selected reinforcements using half-cell potential, polarization, impact (seismic), and ultra sonic test techniques.

5. MMCE reviewed results from NDT for evidence of corrosion and to evaluate the condition of tested reinforcements.

6. MMCE prepared this report describing results and conclusions from Tasks 1 - 5 including a condition assessment of the tendons and rock bolts at the site. The report also includes recommendations and a cost proposal for Phase II of the study, which would involve invasive testing to verify results from NDT.

The scope of the engineering services was based in part on information provided by NHDOT describing the results of geologic studies, design and installation of rock reinforcements, and details of instrumentation and monitoring of conditions along the rock cut subsequent to construction (Fowler, 1976a, 1976b; Haley and Aldrich, 1973a, 1973b, 1974, 1975, 1976, 1979a, 1979b, 1979c). MMCE relied on this information to describe relevant site and rock reinforcement details and to prepare the sampling plan and condition assessment strategy for this project.

Details of Rock Reinforcement

Figure 3 is a typical cross section of the rock cut depicting details of the rock bolts and rock tendons installed at the site. The toe of the 130 ft high cut is at approximately Elevation (EL) 730 ft, and the crest of the cut is approximately EL. 860 ft. The slope at the face of the cut is 1/2 horizontal to 1 vertical (½H:1V), and a rock bench is constructed along the face at approximately El. 820 ft.

As described by Haley & Aldrich (1974) and shown in Figure 3, two basic types of reinforcements are installed at the site:

1. Unstressed, full length grouted, high strength steel tendons designed to act as tension elements against down and outward movement of rock.

2. Stressed rock bolts designed to improve stability of surface blocks and improve the overall integrity of the rock mass.

Tendons. The steel tendons are 1.25-in diameter, Grade 150 ksi steel, Dywidag thread bars. The bars are fully grouted, using Portland cement based grout, in 3-in diameter holes depicted in Figure 4. The tendon is fixed in the hole by steel wedges or other means, but centralizers are not indicated along the lengths of the bars. However, couplings are installed as necessary to achieve the required tendon lengths. Grout was pumped into the annulus between the bar and the drill hole using the grout fill tube shown in Figure 4.

The unstressed steel tendons are 60 feet long passive elements whereby load is transferred to the reinforcements as the rock mass deforms. A significant feature of the unstressed steel tendons is that bearing plates and nuts are not used as a reaction at the rock surface. Figure 4 shows that the ends of the reinforcements are not exposed, and are covered with a grout plug. This is an important detail from the standpoint of access to the ends of the steel tendons required for condition assessment.

Rock Bolts. Two types of rock bolts were installed at the Barron Mountain site including 1-in diameter, polyester resin grouted bolts supplied by Bethlehem Steel Co., and 1-in diameter, polyester resin grouted Dywidag bolts supplied by Inland-Ryerson Steel Co. The bolt head assemblies include bearing plates and nuts. The Bethlehem bolts are made from Grade 80 steel and were prestressed by tightening the surface nut. The Dywidag bolts are Grade 150 steel and were tensioned using a hydraulic center pull jack. Working loads are 20 kips and 40 kips for the Bethlehem and Dywidag bolts, respectively.

For resin-grouted rock reinforcement, the resin is in two components in a frangible Mylar plastic package sized to fit the hole. The package has two components, the resin and a setting catalyst, separated by either a plastic barrier or a thin zone of set-up resin at the interface of the resin or catalyst. By inserting the rock bolt into the hole, the resin and setting catalyst packages are broken, and mixing is achieved by rotating the bolt. The resin is not an adhesive or glue; but rather it is a filler. The resin develops the strength of the bar and rock by filling in irregularities in the drilled hole and bar deformations.

Due to the need to achieve a mixing action within the hole, the diameter is usually kept to within ¼ in of the bar diameter. Centralizers and couplings are not used. Rock bolt lengths at the Barron Mountain site vary from 10 ft to 30 ft (Haley & Aldrich, 1974).

Due to the different installation details including grout type, method of grouting, anchor head details, drill hole diameter, and the lengths of the reinforcements, MMCE considered rock bolt and tendon reinforcements separately for the purpose of condition assessment. One important detail of the installations involves the grout type. Portland cement based grout is alkaline and protects the steel reinforcement by passivating the steel as well as providing a barrier to moisture and oxygen. However, passivation of the steel may be compromised by the presence of chlorides or acidic conditions. Polyester resin grouts are neutral and do not passivate the steel. They protect the steel by creating a barrier. However, due to the method of installation, the amount of cover associated with the resin grout is uncertain. Also, prestressing tends to cause resin grout to crack, particularly near the proximal end of the reinforcements where there is a readily available supply of oxygen that may contribute to corrosion. One of the goals of the condition assessment is to study the integrity of the grouts with respect to providing a barrier surrounding the reinforcements, and the degree to which Portland cement grout is passivating the steel.

Throughout this report the “proximal” end of reinforcements refers to the end closest to the rock face, and the “distal” end refers to the end furthest from the rock face.

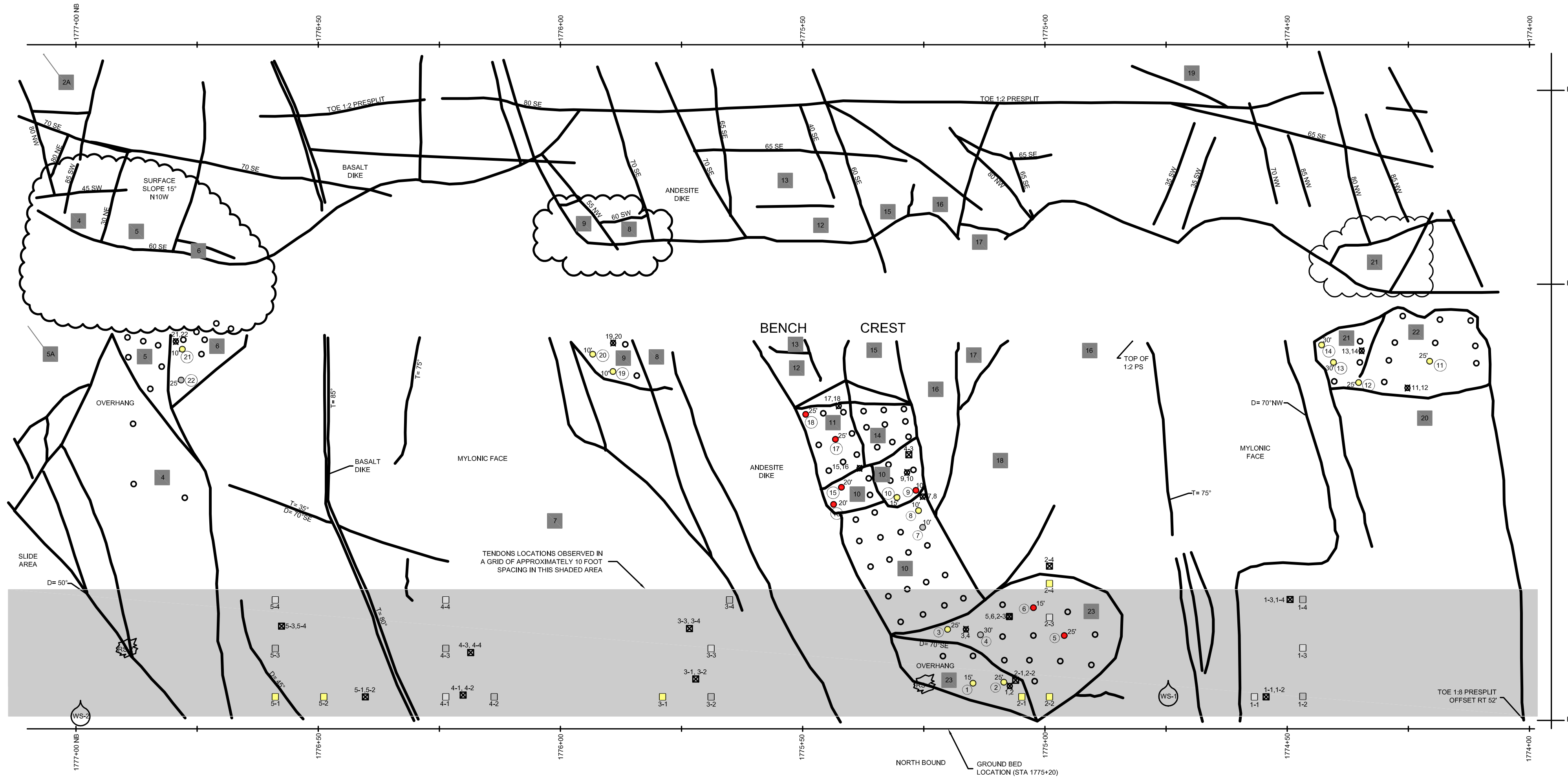
Site Details

Fowler (1976a and 1976b) describes the structural and geologic features of the rock cut at the site. An andesite dike divides foliated and nonfoliated regions of the rockmass. Rocks along the southern end of the cut are less foliated and are predominantly gneiss, while the north end of the site is a foliated schistose. The rock mass is intercepted by two major joint sets, which create a blocky rock mass structure. Many of the joint surfaces are open and some exhibit rust staining. Mylonite seams dipping steeply toward the rock cut contribute to the instability of the rock mass. A rockslide at the north end of the rock cut occurred along a mylonite seam during construction.

Water seeps from the rock seams and open joints exposed at the rock face. The presence of water in the rock seams may contribute to a corrosive environment along the lengths of the metal reinforcements. Additionally, coupled with the presence of water seeping through the rock seams, the northern climate creates cycles of freezing and thawing at the rock face. Freezing at the rock face may contribute to weathering of the rockface, build up of cleft water pressures and cycles of loading and unloading along the reinforcements.

Figure 5 is a detailed elevation view of the site depicting structural features of the rockmass, and rock bolt and tendon locations. Figure 5 is based on the “Geologic Structure Map Woodstock P-7889-F” and field notes provided by NHDOT. Stationing established along the northbound lane of I-93 increases in a northerly direction. Figure 5 includes details between Station 1774+00 and 1777+00.

Dark lines in Figure 5 depict geologic features of the rock mass including rock joints, overhangs, and intrusions (dikes), as well as the slide area at the north end of the site and the rock bench constructed at approximate El. 820 ft. Based on field notes provided by NHDOT, the rock mass is divided into blocks, numbered from 4 to 23, and identified with large, filled, squares in Figure 5. These block numbers are useful to reference bolt locations and descriptions provided in the field notes. In general, blocks are mapped



LEGEND

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TENDON TEST STATIONS

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BARRON MOUNTAIN SITE ELEVATION VIEW

WOODSTOCK NEW HAMPSHIRE

DRAWN BY: T.W.S.
CHECKED BY: K.L.F.
SCALE: 1" = 10'
DATE: FEBRUARY 2004
JOB NO. 03024
FIGURE 5
DWG. NO. 03024-001
REVISION NUMBER - 0

from North to South between Stations 1777+25 and 1774+00.

Most of the rock bolts and tendons adjacent to the northbound lane are located between Stations 1774+00 and 1777+00. Rock bolt and tendon locations shown on Figure 5 are approximate. Rock bolts, identified as small circles in Figure 5, include all bolt locations observed by MMCE below the bench (El. 820 ft). Most of the rock bolts are located in the vicinity of Sta. 1775+25, near the southern contact with the andesite dike. Rock bolts included in the sampling population for condition assessment are identified as filled circles. These circles are color coded based on the results of NDT. Symbols filled with "x" are used to identify half-cell placement for electrochemical tests.

Tendons are located in an approximately 10 ft. by 10 ft. grid pattern, along three rows, within the shaded area between Stations 1774+00 and 1777+00. Tendons included in the sample population for condition assessment are identified with small squares, which are also color coded.

Additional tendons and rock bolts were installed above El. 820 ft., at the north and south end of the site as indicated in the clouded areas of Figure 5.

APPROACH

Details of the recommended practice for condition assessment are described in NCHRP (2002). In general the procedure includes:

- Assessment of the corrosion potential at the site and estimation of remaining service life.

- Condition assessment of rock reinforcement in terms of (1) loss of prestress, (2) grout quality which may affect the vulnerability of the reinforcements to corrosion, (3) the occurrence of corrosion, and (4) distress from loss of cross section due to corrosion or from bending/kinking along the lengths of the reinforcements.

- Compare results of condition assessment to anticipated condition of reinforcements based on site conditions and estimation of remaining service life.

- Recommend an action plan based on condition assessment and estimated remaining service life.

Assessment of Corrosion Potential

Quantitative guidelines are available for assessing the potential aggression posed by an underground environment relative to corrosion(PTI, 1996; FHWA, 1993). Generally, moisture content, chloride and sulfate ion concentration, resistivity and pH are identified as the factors that most affect corrosion potential of metals underground. MMCE retrieved samples of weathered rock and groundwater from the Barron Mountain site. Laboratory tests were performed on these samples to obtain parameters needed as input for corrosion assessment models and associated mathematical models for estimation of remaining service-life.

The following paragraphs describe the sampling and laboratory testing used to assess the corrosion hazard at the Barron Mountain site.

Weathered Rock Samples

MMCE, together with Marc Fish and Dick Lane from the NHDOT Bureau of Materials and Research, collected samples of weathered rock (see Figure 6) from two locations along the face of the rock cut during MMCE's reconnaissance visit on August 26, 2003, and subsequent fieldwork from September 8, 2003 to September 17, 2003:

- The location of Sample #1, shown in Figure 5, was near station 1775+25, approximately 5 to 10 feet above the toe of the rock cut. The sample was retrieved from a rock seam that dipped approximately 60° toward the roadway near the interface between the weathered gneiss and the andesite dike.

- The location of Sample #2, also shown in Figure 5, was near station 1776+80 approximately 15 feet above the toe of the rock cut. The sample was retrieved from a rock seam that dipped approximately 35° towards the roadway near the interface between the weathered mica schist and the mylonite seam along which a major rock slide occurred during construction of the roadway in 1972.

Fragments of weathered rock were loosened with a geologic pick, and collected in a zip-lock plastic bag. The sample was then wrapped in two additional zip-lock baggies, and sealed with tape, before transport to the NHDOT geotechnical laboratory. The samples were transported to NHDOT's geotechnical laboratory and the following tests were performed on each sample:

Test	***Test Method***
Moisture Content	*AASHTO T265*
Grain Size Analysis	*AASHTO T88*
Resistivity	*AASHTO T288*
pH	*AASHTO T289*
Sulfate Content	*California Test 417*
Chloride Content	*AASHTO T291*

Groundwater Samples

MMCE collected groundwater samples from two locations along the rock cut during the fieldwork conducted from September 8, 2003 to September 17, 2003. Both of these sample locations are indicated in Figure 5. The first sample location was from the end of a hollow-core rock bolt exposed at the rock face near station 1774+75, at an elevation approximately 5 feet above the toe of the rock cut. Water was collected as it seeped from the end of the rock bolt as shown in Figure 7.

The second location was from an existing observation well located near the base of the rock cut near Station 1777+00. The observation well, shown in Figure 8, is an approximately four inch diameter steel casing, originally installed to monitor microseisms as part of an instrumentation and monitoring program instituted in 1972, subsequent to the rock slide. MMCE measured the depth of the observation well and the depth to water within the well as 94 feet and 30 feet respectively. A groundwater sample was retrieved from the well after purging three casing volumes of standing water from the well. Well purging is necessary to obtain a sample that is representative of the hydrogeologic regime, and not standing water in the well. Subsequent to purging, a sample of ground water was extracted from the well using a dedicated bailer.

MMCE measured the pH and temperature of samples #1 and #2 in the field before the samples were transported to NHDOT's geotechnical laboratory. NHDOT performed the following laboratory tests on each sample:

Test	***Test Method***
pH	*ASTM D1293*
Magnesium Content	*ASTM E633*
Sulfate Content	*California Test 417*
Chloride Content	*ASTM D512*
Ammonium	*AWWA Method 350.3*

Assessment of Rock Reinforcement

Sample Population

Nondestructive testing and condition assessment of rock reinforcements requires a sampling strategy whereby the appropriate sample size is selected to provide a statistical basis for the test results. MMCE employed a simplified sampling criteria (NCHRP, 2002) based on the probability that the sampled population will represent conditions throughout the site. The recommended sample size is based upon the total number of reinforcements at the site, the importance of the facility relative to the consequences of failure, and a reference or baseline condition for comparison to observations. Figure 5 shows the locations of tendons and rock bolts that were included in the sampling plan.

Twenty tendons distributed amongst five stations were tested. The tendons included in the sample population are located within the shaded area shown on Figure 5. Tendon test stations were selected to achieve a good spatial distribution of samples between elevations El. 730 ft and El. 757 ft, and between Stations 1774+00 and 1777+00. Four tendons were tested at each station; two corresponding to approximate EL. 735 ft and spaced approximately 10 feet apart, one at approximate El. 745 ft, and one at approximate El. 755 ft. Each tendon sample is identified by station and element number, e.g. Tendon 3-2 is located at test station #3 and is the second element in the group at approximate El. 735 ft.

Twenty-two rock bolts were included in the sampling plan. The bolts included in the sample population are shown with filled circles in Figure 5 along with the "known" (based on field notes) bolt length, and an arbitrary number assigned to each tested bolt. Sixteen of the tested rock bolts (approximately 70%) are located along the central portion of the site near Station 1775+25 , four are located at the south end of the site near Station 1774+25 and two test bolts are located at the north end of the site near Station 1776+70.

Some of the reinforcements at the site were located in areas that were inaccessible. These include approximately 24 rock tendons and 50 rock bolts located above the rock bench (EL. 820 ft) between Sta. 1777+00N and Sta. 1778N. MMCE was not able to observe these reinforcements.

Coordination with NHDOT

NHDOT provided access to the site and managed traffic control during the test period. The shoulder of the north bound lane of I-93 was barricaded between approximate Stations 1774+00 and 1777+00. Site access included use of a hydraulic man-lift and a crane and basket to access bolts and tendons along the rock face. All of the tendon locations, and bolts 1 though 10, were accessible with the man lift. The crane and basket were necessary to access bolt numbers 11 through 22. The man lift was onsite for the duration of the fieldwork from September 8, 2003 to September 17, 2003. The crane was available beginning September 15, 2003, and included two operators from NHDOT.

NDT and Condition Assessment

Nondestructive test techniques are used to probe the reinforcements, and the results are analyzed for condition assessment. Four NDT's are employed for condition assessment of rock reinforcements at the Barron Mountain site including measurement of half-cell potential, polarization current, impact and ultrasonic testing. Details of NDT including test procedures are described by NCHRP (2002). Salient details of the test methods and details applicable to the Barron Mountain Site including sample preparation, and placement of instruments are described in this report.

Half-cell potential and polarization measurements are electrochemical tests and the impact and ultrasonic techniques are mechanical tests involving observations of wave-propagation. In general, these NDT's are useful indicators of the following aspects of the condition assessment:

- Half-cell potential tests serve as an indicator of corrosion activity.

- Results from the polarization test are correlated with the surface area of steel that may be in contact with the surrounding rockmass, i.e. indicator of grout quality and degree of corrosion protection.

- Impact test results are useful to diagnose loss of prestress, assess grout quality and indicate if the cross section is compromised from corrosion, or from a bend or kink in the bolt.

- Ultrasonic test results are useful for obtaining more detailed information about the condition of reinforcements within the first few feet from the proximal end of the reinforcement.

Some aspects of the condition assessment are common between the polarization test, impact and ultrasonic tests. This provides a means to cross check results obtained from different test techniques and implement quality control. Assessment of grout quality from the polarization test may also be compared to results from impact testing. Ultrasonic test results are useful for verifying results from impact tests relative to the first few feet from the proximal end of the reinforcements.

Sample Preparation

The ends of the reinforcements need to be exposed, cleaned, and ground relatively flat and smooth in preparation for NDT. A surface grinder was used to grind and clean the exposed ends of the rock bolts in preparation for attachment of wires and transducers necessary for NDT. The bolts were identified with a number transcribed on the bearing plate using a grease pen. Figure 9 is a photograph of a typical bolt end that has been prepared for testing.

The ends of the steel tendons were exposed prior to testing by chipping away the grout plug with a pneumatic hammer. This process left the end of the tendon exposed within the borehole at a distance, which varied from 3 inches to 10 inches from the rock surface. An extension to the end of a power drill was then used to clean the end of the tendon. Appendix I shows photographs of the exposed ends of some of the tendons after cleaning.

Subsequent to NDT, MMCE re-grouted the ends of the tendons. Where conditions permitted, MMCE attached wires to the ends of the reinforcements prior to grouting. The wires provide electrical connections, which will be useful for future monitoring without the need for removing the grout plug from the reinforcements. Tendons 1-3, 1-4, 2-1, 2-3, 2-4, 3-2, 4-1, 4-3, 5-1, 5-2, 5-3, 5-4 were centered well enough within the drill hole such that wires could be attached to the proximal ends of the reinforcements with plastic zip ties.

Electrochemical Test Methods

Testing requirements include attaching a wire (AWG 8) to the end of the tested reinforcement. For the rock bolts, the wire was secured using a pair of vice grips as shown in the photograph presented in Figure 10. For the tendons, which were recessed 3 to 10 inches into the rock face, an expandable gripper (nut retriever) was secured to the end of the tendon and the wire was clamped to the end of the gripper as shown in Figure 11.

Half-Cell Locations. In general, half-cells are located close to the reinforcement being tested. In many instances drill holes are available next to the test location where the half-cell may be inserted. Good electrical contact between the half-cell and surrounding rockmass is facilitated by surrounding the tip (porous ceramic plug) of the half-cell with a wet sponge, which is then placed in contact with the rockmass. If a nearby drill hole is not available, then the half-cell is placed within a nearby open rock joint or crevice.

It is important to document half-cell locations because if future measurements are obtained for comparison with previous measurements, half-cell placement should be consistent. Half-cell locations for testing tendons and rock bolts are indicated in Figure 5 using square and circle symbols filled with “x”. Appendix II includes photographs of the half-cell locations for reference, and a table that provides a summary of half-cell locations.

Ground Bed. Three copper plated, 0.5 inch diameter, rods were placed near the base of the rock cut at approximate Station 1775+20 to act as a ground bed as shown in Figures 5 and 12. Each rod was advanced approximately 1 foot into the soil at the base of the rock cut. The rods were wired together and the lead was extended to the test box for connection during testing. Using the ground bed and a six volt battery source, the maximum applied current ranged from 3 to 5 milliamperes. The same ground bed was used for polarization measurements on all of the rock bolts and tendons included in the sample population.

Impact Testing

An accelerometer was attached to the reinforcements with a special mounting base which has threads that fit the base of the accelerometer. The mounting base was glued to the end of the rock bolts and tendons with *Loctite 454* instant adhesive. For the tendons, the base was attached using an expandable gripper that could be inserted into the recess to reach the end of the tendon. After the glue had set, the accelerometer was attached to the base. A hollow copper tube was used to insert the accelerometer into the recess and secure it onto the base with the lead wire attached.

Ultrasonic Testing

Good acoustic coupling between the transducer and the face of the reinforcement is a requirement for ultrasonic testing, and the face of the each reinforcement must be flat and smooth.

MMCE performed the ultrasonic test on twenty-two rock bolt samples. It was not possible to conduct the ultrasonic test on tendon type reinforcements due to difficulties accessing and preparing the ends of the reinforcements for testing.

FINDINGS

Assessment of Corrosion Potential

Laboratory Test Results

Weathered Rock.

Test data submitted by NHDOT are included in Appendix III. Table 1 is a summary of the results from testing samples of weathered rock.

Table 1. Summary of Laboratory Test Results for Weathered Rock

Sample #	% Pass #4	%Pass #200	w %	pH	Resist. Ω-cm	SO_4 ppm	Cl^- ppm
1	90.2%	6%	13	5.1	9590	652	720
2	57.9%	9%	7	4.2	4215	ND	250

As described in Appendix IV, the measured pH, resistivity, and moisture conditions within the weathered rock correspond to a corrosive environment. Measured sulfate and chloride ion concentrations are also at levels high enough to be conducive to a corrosive environment. The corrosiveness classification at the site is between II and III, on a scale where "I" is considered highly corrosive and "IV" is slightly corrosive (Clouterre, 1991). This rating is used to estimate the rate of metal loss anticipated over the service life of the reinforcements.

Ground Water

The following is a summary of the measurements made on samples #1 and #2 on Thursday, September 11, 2003 by MMCE during sample retrieval.

	pH	Temp.	
#1	5.78	27.9° C	From bolt hole ~ sta. 1774+75
#2	5.61	20.6° C	From well ~ sta. 1777+00

Results from laboratory tests performed by NHDOT, and presented in Appendix III, indicate that concentrations of magnesium, chloride, sulfate and ammonium ions in the groundwater are very low or below detectable limits.

Estimated Remaining Service Life

Detailed calculations are presented in Appendix IV. Assuming that no corrosion protection is present, the anticipated remaining service life for rock bolts and rock tendons is 14 and 20 years, respectively. At this time it is estimated that reinforcements may have lost as much as 16% of the original cross section due to corrosion. This amount of loss of cross section is considered to be close to the sensitivity of NDT measurements. *(Note: The calculation of remaining service life for rock tendons assumes that stress levels beyond 60% of the specified minimum tensile strength are allowed by NHDOT. For details see Appendix IV).*

Results from NDT

Half –Cell Potentials

Half-cell potentials are affected by a number of environmental factors including pH. The rock bolts at the Barron Mountain Site are surrounded by polyester resin grout, which does not provide the same high pH environment compared to the Portland cement based grout that surrounds the tendon reinforcements. Therefore, observed half-cell potentials for rock bolts will be considered separately from those observed for tendons .

Results from measurement of half-cell potentials for rock bolt reinforcements are presented in Figure 13. The scale in Figure 13, which describes the possibility of corrosion, is based on the galvanic series of metals in neutral soils and water described by Peabody (1967), and the tendency for the half-cell potential of carbon-based steel to shift in the positive direction as it corrodes in a neutral soil or water environment. Most of the rock bolts (19 out of 22 tested) have half cell potentials higher than -500 mV indicating that corrosion has likely occurred. Half-cell potentials for seven of the rock bolts including numbers 5, 6, 9, 15, 16, 17 and 18 are greater than –300mV indicating that these bolts may have experienced a greater degree of corrosion compared to the other rock bolts where half cells were measured.

Figure 14 shows the rock tendon half-cell potential measurements. The "Possibility of Corrosion" scale shown in Figure 13 is based on ASTM C876 (ASTM 2001) and applies to carbon-based steel that has been passivated in an alkaline environment; i.e. surrounded by Portland cement grout. Leaching of grout was evident in the form of calcite deposits below tendon 2-2, as shown in the photograph in Figure 15. Thus, the environment that surrounds Tendon 2-2 may be modified compared to the other tendons included in the sample population, and this may be why the half-cell potential of Tendon 2-2 is much lower than the others. Half-cell potentials for six of the tendons including 2-1, 2-2, 2-4, 3-1, 5-1, and 5-2 are lower than –350 mV. As described by ASTM C876, this indicates that corrosion is very likely amongst approximately 30% of the sampling population.

Polarization Measurements

NCHRP (2002) describes how polarization curves are prepared and the polarization current, I_P, is observed. Appendix V presents polarization curves for the reinforcements tested at the Barron Mountain site. The polarization current is correlated with the surface area of steel that may be in contact with the surrounding rockmass; i.e. not surrounded by impervious grout. Therefore, the polarization current divided by the known length of the reinforcement (polarization ratio = I_p/L) is a useful indicator of the quality of grout along the reinforcement. Relatively, high polarization current ratios correspond to poor grout quality.

The “known” reinforcement lengths and observed values of I_P and I_p/L are summarized in a table at the beginning of Appendix V. Fifty-nine percent of the rock bolts included in the sample population (13 out of 22) are associated with polarization current ratios equal to or higher than 0.07 mA/ft. Based on results from impact testing, discussed later, it appears that all of the reinforcements with $I_p/L_T \geq 0.07$ are associated with low quality grout along some portion of their length.

Polarization current ratios are lower for tendons compared to rock bolts, and, in general, there is more scatter in the results obtained for rock bolts compared to tendon reinforcements. This suggests that, in general, the condition of Portland cement grout along the tendon lengths is in better overall condition compared to the polyester resin grout surrounding the rock bolts.

Impact Testing

MMCE performed at least three impacts on each reinforcement within the sample population. After results were observed to be repeatable, the three test results were averaged rendering a single test record for each reinforcement. Acceleration amplitude was normalized for each record by dividing the accelerations measured at each time increment by the maximum acceleration observed within the time history. Each test record was studied in terms of the rate of decay (damping) of the initial portion of the acceleration time history, and the amount of signal attenuation evident in the reflected waveforms.

Damping, or the rate of decay, of the acceleration amplitude response has been shown to increase with respect to level of prestress for rock bolts (Rodger et al., 1997). The relationship between rate of decay and prestress provides a means to diagnose loss of prestress in a rock anchorage. The envelop of the positive peaks of the amplitude response over the initial portion of the acceleration time history (damping envelop) portrays the rate of decay as shown in Figures 16 (a) and (b). The damping envelop portrayed in Figure 16 (a) is typical of most of the rock bolts tested at the Barron Mountain site and is considered to represent the response of rock bolts under relatively higher tension levels. Figure 16 (b) is representative of low damping, for which loss of prestress is considered a possibility. Appendix VI presents damping envelops observed for all of the rock bolts tested at the Barron Mountain site. Seven of the tested rock bolts, corresponding to approximately one third of the sampling population, including numbers 5,6,9, 11,12, 19 and 20, are associated with relatively low rates of decay. On the basis of the observed damping envelops, significant loss of prestress is considered a possibility for these reinforcements.

Responses from impact testing are recognized in terms of relatively strong, versus relatively weak, signal attenuation as depicted in Figures 17 (a) and (b), respectively. If the grout surrounding a reinforcement is very high quality, then strong reflections are not expected beyond a distance of approximately ten to fifteen feet (0.0012 (s) to 0.0018 (s)). Therefore, for test results that display relatively strong signal attenuation as portrayed in Figure 17 (a), the tested bolts are presumed to be surrounded by good quality grout; at least for the first ten or fifteen feet from the proximal end of the reinforcement. Results that display relatively weak signal attenuation, such as that portrayed in Figure 17 (b) may be from reinforcements that are not surrounded by good quality grout, include irregularities or anomalies within the first ten to fifteen feet, or have a total length that is less than ten to fifteen feet.

Time histories for impact tests performed on rock bolt samples #1 through # 22 are presented in Appendix VII. Results are summarized at the beginning of Appendix VII in terms of signal attenuation, travel distances to reflections apparent in the signals (L_1 and L_2), and the "known" bolt lengths which serve as a reference for comparison with observations. Based on the information presented in Appendix VII, approximately 36% (8 out of 22) of the tested bolts exhibit relatively strong signal attenuation. Bolts that exhibit strong or weak signal attenuation do not appear to be clustered within any particular area of the site, and this attribute of the bolt installations appears to be randomly distributed.

Appendix VIII includes graphs of time versus acceleration observed from impact tests performed on Tendons # 1-1 through #5-4, and a summary of the results. All of the tested tendons have a "known" length of 60 feet. Based on the information presented in Appendix VIII, approximately 80% (16 out of 20) of the tested tendons exhibit strong signal attenuation indicating that the tendons are surrounded by high quality grout, at least for the first 10 to 15 feet from the heads of the reinforcements.

Ultrasonic

Ultrasonic tests were performed on Test Bolts #1 through 22 and graphs of signal amplitude versus time are presented in Appendix IX. These results are useful to verify and confirm results from impact tests performed on the same reinforcements. In general, relatively high signal attenuation is observed in the results from ultrasonic testing. In most cases, reflections were not observed beyond 1.5 ms corresponding to locations of approximately 12 feet from the free end of the rock bolt. The first reflections (L_1) observed from impact test results are also apparent from results of ultrasonic testing, although in general these appear as subtle reflections within the ultra sonic test signals. Unless the first reflection is from the end of a bolt less than 12 feet long, reflections corresponding to the end of the bolt are not apparent in the ultra sonic test results.

Due to the higher frequency content of the sound waves, reflections from sources within three feet of the proximal end of the rock bolts are more apparent in the results from ultrasonic testing compared to the impact test results. In general, these earlier reflections occur at 0.1 ms to 0.4 ms intervals, corresponding to distances from 0.8 to 3.0 feet from the proximal ends of the rock bolts. The proximal ends of the rock bolts extend approximately 5 to 15 inches beyond the face of the rock accounting for the thickness of the face plate and nut, and extension of the rock bolts beyond the nut. Therefore, these reflections are approximately 0 to 24 inches from the face of the rock, and most likely correspond to the beginning of the grout column behind the face plate, or a change in grout quality at these locations.

CONCLUSIONS

Condition assessments of rock bolts and tendons at the Barron Mountain site are summarized in Tables 2 and 3, respectively. The condition assessment is based on results from the NDT including half-cell potential, polarization measurement, impact and ultra sonic tests; and from visual observations made by MMCE during their site visit. The assessment includes judgment relative to loss of prestress, grout quality, the occurrence of corrosion, and apparent distress to the reinforcement cross section. Poor quality grout may be from installation conditions, or from grout that may have become loosened along the length of the reinforcement during service.

Based on information included in Table 2, approximately 30 percent of the sampled rock bolts have suffered loss of prestress. Figure 18 compares the results of the condition assessment for rock bolts and tendons. In general, results indicate that tendon reinforcements are in better condition relative to rock bolt reinforcements at the site.

Both results from impact testing and polarization measurement support the conclusion that grout quality along the tendon lengths is more consistent compared to the grout quality along the lengths of rock bolts observed in the sample population. The possibility of relatively poor grout quality near the proximal end of the reinforcement was observed for 20% of the tested tendons, compared to 70% of rock bolts. When impact test results include reflections corresponding to the end the reinforcement, grout quality is questionable along a significant proportion of the reinforcement length. However, observations of grout quality near the distal end of the reinforcements are more difficult and considered less reliable than those pertaining to the proximal end of the reinforcement. Therefore, no general conclusions are drawn relative to the condition of the grout quality beyond the first ten to fifteen feet from the proximal end of the reinforcements.

Both observations from polarization measurements and impact tests are useful for assessment of grout quality. Bolt #7 has the highest observed value of I_p/L_T and is associated with weak signal attenuation and a very distinct reflection from near the bolt end. Impact test results on all bolts with $I_p/L_T \geq 0.07$ including bolts #1, 6, 7, 8, 10, 11, 12, 15, 16, 17, 19, 20 and 21 are associated with weak signal attenuation, or reflections are apparent from locations corresponding to the "known" length of reinforcements longer than 10 feet. Thus, observations from polarization measurements are consistent with those from impact testing.

Measurements of half-cell potential indicate that approximately 85 percent of the tested rock bolts have "likely" or "very likely" experienced corrosion, compared to 30% of the tested rock tendons that have "likely" experienced corrosion. Corresponding impact test results suggest that rock bolts that have "very likely" experienced corrosion are surrounded by relatively low quality grout.

Unlike the random spatial distribution of reinforcements with questionable grout condition, the likelihood of corrosion is more strongly correlated with rock condition.

The tested reinforcements shown in Figure 5 are displayed with filled symbols that are color coded to indicate the possibility of corrosion. Dark gray symbols indicate that corrosion is “not likely”, light gray is “uncertain” possibility of corrosion, yellow is for “likely”, and red symbols indicate areas where the possibility of corrosion is “very likely”. The locations of reinforcements that have “likely’ or “very likely” experienced corrosion also correlate with the observation of seeps along the face of the rock cut. These seeps were observed in the vicinity of Stations 1775+25 and 1776+50.

Table 2. Summary of Rock Bolt Condition Assessment

Bolt #	Condition Assessment				
	Apparent Loss of Prestress	Grout Quality		Corrosion	Apparent Distress to Cross Section
		Free End	Bond Zone		
1	No	Good	Questionable	Likely	No
2	No	Good	Good	Likely	No
3	No	Good	Good	Likely	No
4	No	Good	Good	Not likely	No
5	Yes	Questionable	Undetermined	Very likely	Yes
6	Yes	Questionable	Undetermined	Very likely	Yes
7	No	Questionable	Questionable	Not likely	No
8	No	Questionable	Questionable	Likely	No
9	Yes	Questionable	Questionable	Very likely	No
10	No	Good	Questionable	Likely	No
11	Yes	Good	Questionable	Likely	No
12	Yes	Questionable	Undetermined	Likely	Yes
13	No	Good	Questionable	Likely	No
14	No	Questionable	Questionable	Likely	No
15	No	Questionable	Undetermined	Very likely	Yes
16	No	Questionable	Questionable	Very likely	No
17	No	Questionable	Questionable	Very likely	No
18	No	Questionable	Questionable	Very likely	No
19	Yes	Good	Questionable	Likely	No
20	Yes	Questionable	Questionable	Likely	No
21	No	Questionable	Questionable	Likely	No
22	No	Questionable	Questionable	Not likely	No

Table 3. Summary of Rock Tendon Condition Assessment

Tendon #	Condition Assessment			
	Grout Quality		Corrosion	Apparent Distress to Cross Section
	Near Free End	Full length		
1-1	Good	Questionable	Uncertain	No
1-2	Questionable	Questionable	Not likely	No
1-3	Questionable	Questionable	Uncertain	No
1-4	Good	Good	Not likely	No
2-1	Good	Questionable	Likely	No
2-2	Good	Good	Likely	No
2-3	Good	Questionable	Uncertain	No
2-4	Good	Good	Likely	No
3-1	Questionable	Undetermined	Likely	Yes
3-2	Good	Questionable	Not likely	No
3-3	Good	Good	Uncertain	No
3-4	Good	Questionable	Not likely	No
4-1	Good	Undetermined	Uncertain	No
4-2	Good	Undetermined	Not likely	No
4-3	Good	Good	Not likely	No
4-4	Good	Good	Uncertain	No
5-1	Good	Good	Likely	No
5-2	Questionable	Undetermined	Likely	Yes
5-3	Good	Good	Not likely	No
5-4	Good	Good	Uncertain	No

If impact test results include strong reflections, masking those that could be received from the end of the reinforcement, this may indicate that the cross section has been compromised at some point before the end of the reinforcement, or that the reinforcement may have a bend or kink. However, the source of these reflections may not necessarily be due to a distressed reinforcement. These strong reflections could also be the effect of an open joint pierced by the reinforcement, or, in the case of tendons, from a coupling included during installation.

A review of the impact test data for the tendons reveals that ten percent of the results have attributes that could be associated with apparent distress to the reinforcement from loss of cross section or bending. This compares with approximately eighteen percent of rock bolts; and most of these were concentrated within one area of the site (near Sta. 1775+25). Also, based on half-cell measurements, it appears that that rock bolts and tendons that are apparently distressed (possibly associated with loss of cross section) are likely to have experienced corrosion.

Service-life prediction models were used to estimate the loss of cross section. Results from these computations indicate that rock bolts and tendons may have lost between 10 to 15 percent of their cross section due to corrosion over their thirty-plus years of service. Detection of this loss of cross section is near the sensitivity of the impact and ultrasonic test techniques employed in this study. If loss of cross section is concentrated in one location near the end of the reinforcement, it is possible that a signal reflection would be detected. Therefore, results from NDT, and expectations based on site conditions and results from service life prediction models are consistent. Both results indicate that some loss of service has occurred since the reinforcements were installed, and environmental conditions are more severe than normal relative to corrosion.

The predicted remaining service life for the rock bolts is approximately 15 years. The remaining service life for rock tendons is more difficult to predict and depends on the load carried by the reinforcements. Test results indicate that some of the tendons have been depassivated and corrosion is “very likely”. Elements may be depassiviated by diffusion of chlorides through the grout, or from acidic conditions within the weathered rock. If the tendons are allowed to remain in service until the working stress levels reach the ultimate strength of the material, these reinforcements are expected to survive for approximately 20 more years.

We recommend that the results of NDT be verified by further, more invasive, testing on selected reinforcements; and that reinforcement condition continue to be monitored at regular ten year intervals. However, half-cell measurements should be taken at more frequent intervals and results used to evaluate whether additional tendons at the site become depassivated. Wires have been installed an thirteen of the tendons providing easy access for monitoring. A ladder or lift may be necessary to access the bolts, and we recommend monitoring half-cell potentials of these reinforcements at two-year intervals. Equipment and training required for making half cell measurements are minimal, easily obtained and inexpensive. Therefore, we recommend that NHDOT perform these measurements in-house. Half-cell potentials for rock bolts may also be easily monitored.

Verification of results from NDT should be performed as a second phase to this project following the recommendation described in the next section.

RECOMMENDATIONS & PROPOSAL FOR PHASE II

The proposal for Phase II of this research includes invasive testing of selected rock bolts and tendons to verify results from NDT and service-life estimates performed in Phase I. Invasive testing shall include lift-off tests; and physical, chemical and metallurgical testing on steel and grout samples retrieved from exhumed reinforcements. Replacement bolts must be installed prior to invasive testing of the reinforcements. The following recommendations include details of the test program and estimated costs.

Figure 19 is a photograph showing that blocks of rock in the vicinity of Station 1775+25 appear loosened. Also, during their site visit, MMCE observed water seeping from the end of Bolt #5. Some corrosion or deformation of the bolts may have occurred within this zone, which may contribute to strong reflections evident in the impact test data.

Results from tests on Bolts # 5, 6, 7, 8, 9, 15, 16, 17, and 18 (9 of 14 bolts tested in the area of Station 1775+25) support this interpretation. Impact test results suggest that three of these bolts (21% of the bolts tested near this location) may have a compromised cross section from corrosion or a kink from shear displacement. Based on these observations, MMCE recommends that testing included in Phase II focus mainly on this area of the site.

MMCE proposes that a total of five rock bolts be selected for lift-off tests and subsequently exhumed. In addition two tendon reinforcements should be exhumed. Reinforcements selected for invasive testing shall have been previously evaluated by NDT during Phase I and will include some reinforcements with questionable condition and some reinforcements considered to be in good condition based on the results from NDT.

Appendix X is a summary of the scope, tasks, costs and schedule for Phase II. MMCE estimates the cost for Phase II to be approximately $122,000.00 and this includes approximately three weeks of fieldwork, and the cost of a contractor to test and exhume existing reinforcements and install replacements. MMCE has assumed that the NHDOT will provide traffic control and access to the site.

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12. Haley & Aldrich, Inc. (1976) Barron Mountain Cut, Woodstock, NH, I-93-3(62)96-P-7889-F, submitted to State of New Hampshire Department of Public Works and Highways, Concord, NH, October 26.

13. Haley & Aldrich, Inc. (1979a) Barron Mountain Cut, Woodstock, NH, I-93-3(62)96-P-7889-F, submitted to State of New Hampshire Department of Public Works and Highways, Concord, NH, March 2.

14. Haley & Aldrich, Inc. (1979b) Barron Mountain Rockcut (P-2371, I-93(89) 101), submitted to State of New Hampshire Department of Public Works and Highways, Concord, NH, March 26.

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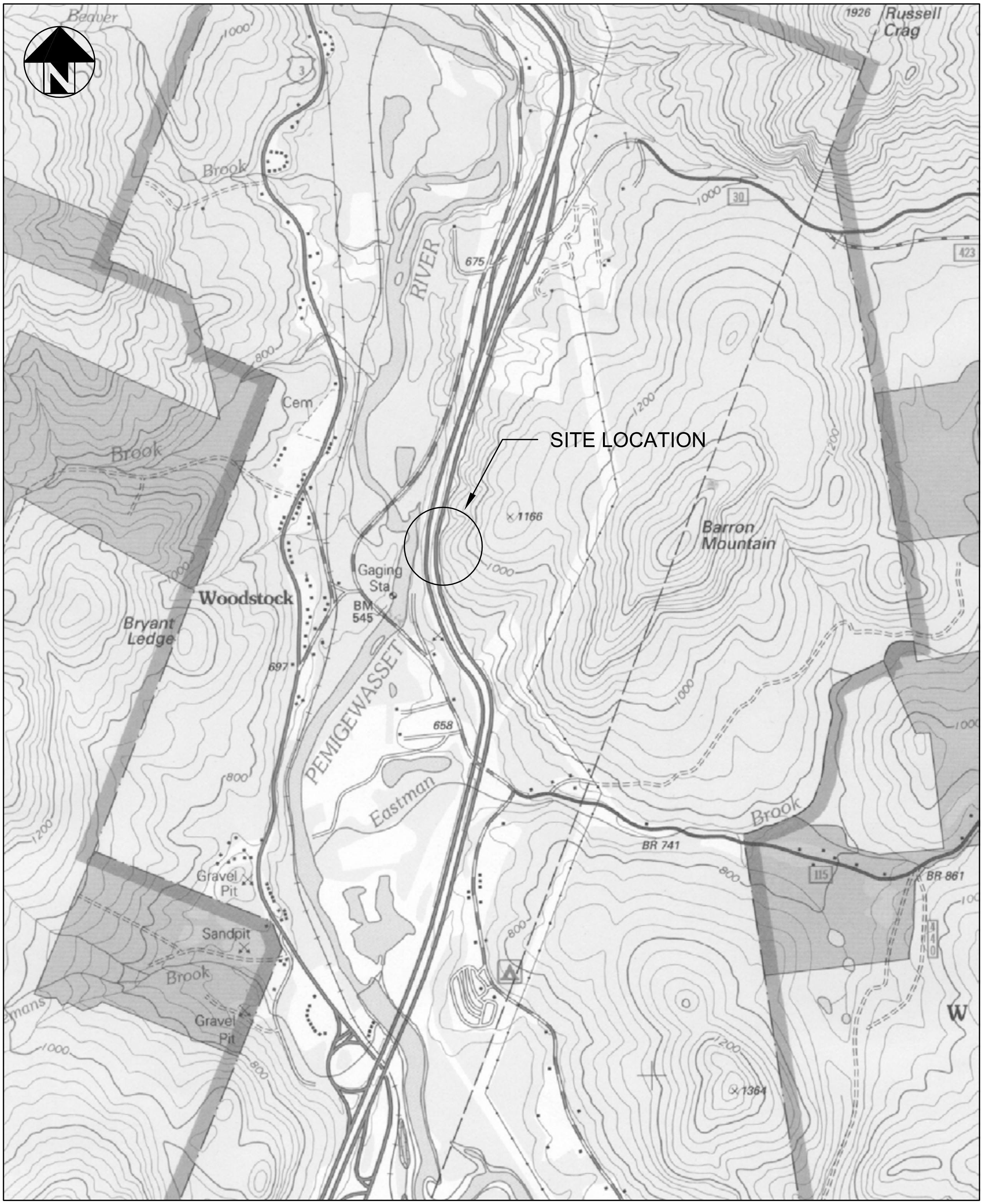
FIGURES



NOTE:
THIS SITE PLAN WAS ADAPTED FROM THE U.S.G.S.
WOODSTOCK, N.H. (1995) 7.5' QUADRANGLE

SCALE 1:24,000

LOCAL SITE TOPOGRAPHY	BARRON MOUNTAIN ROCK REINFORCEMENT CONDITION ASSESSMENT WOODSTOCK NEW HAMPSHIRE	McMahon & Mann Consulting Engineers, P.C.
DWG. NO. 03024-002		2495 MAIN STREET, SUITE 432 (716) 834-8932 BUFFALO, NY 14214 FAX: (716) 834-8934
FIGURE 1		

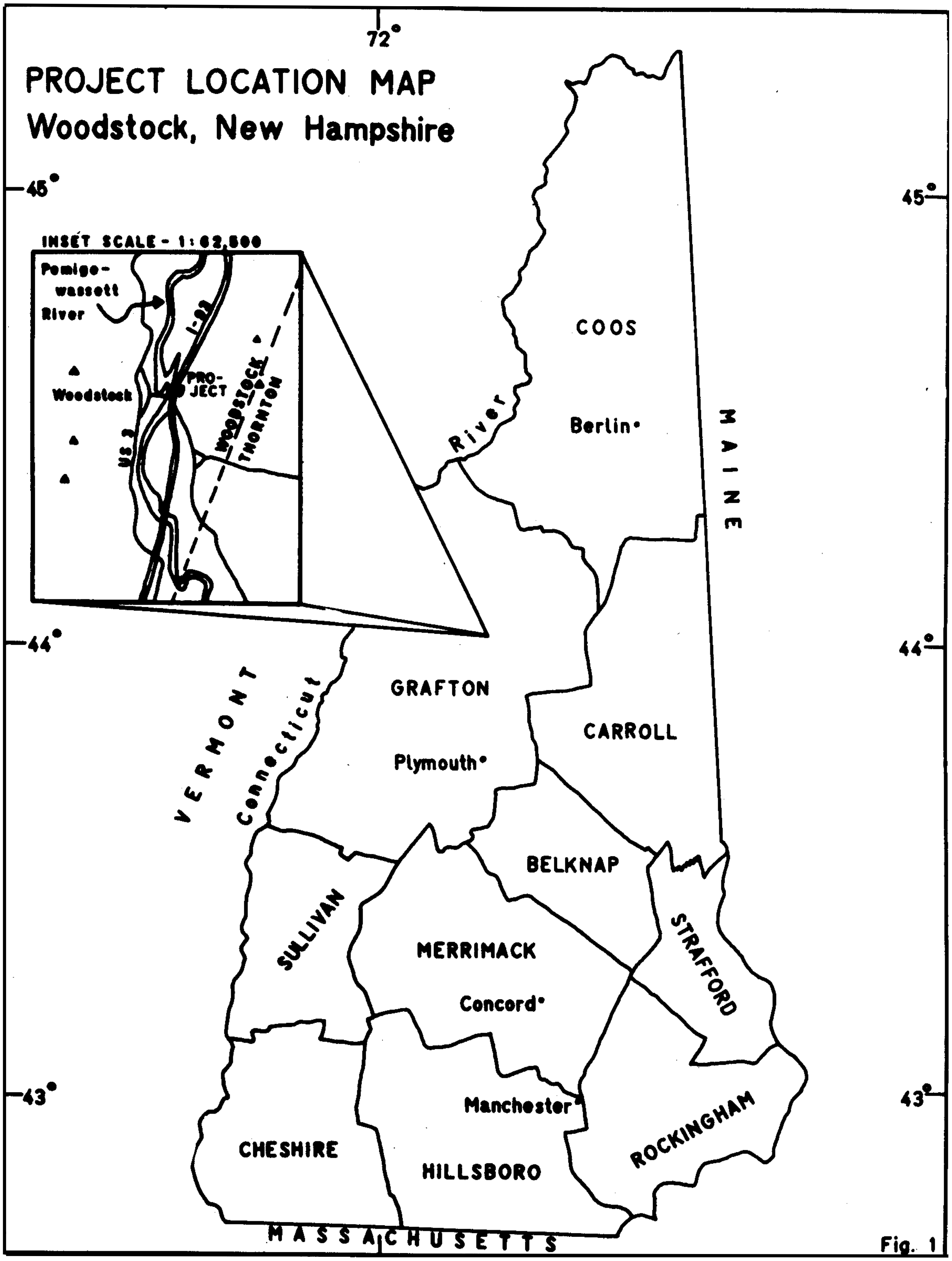


Figure 2. Site Location (Fowler, 1976(b))

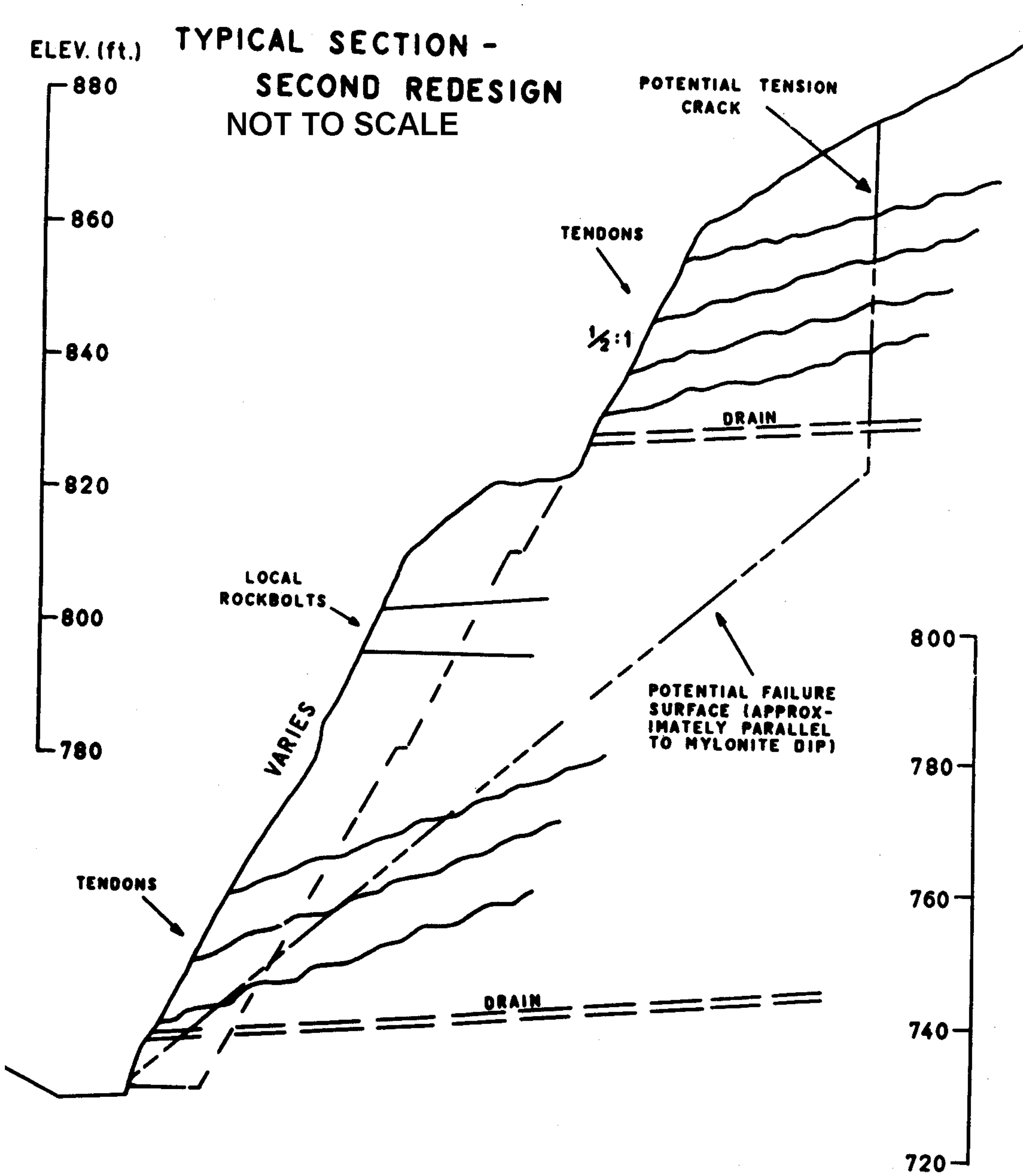


Figure 3. Typical Cross Section of Rock Cut (Fowler, 1976(b))

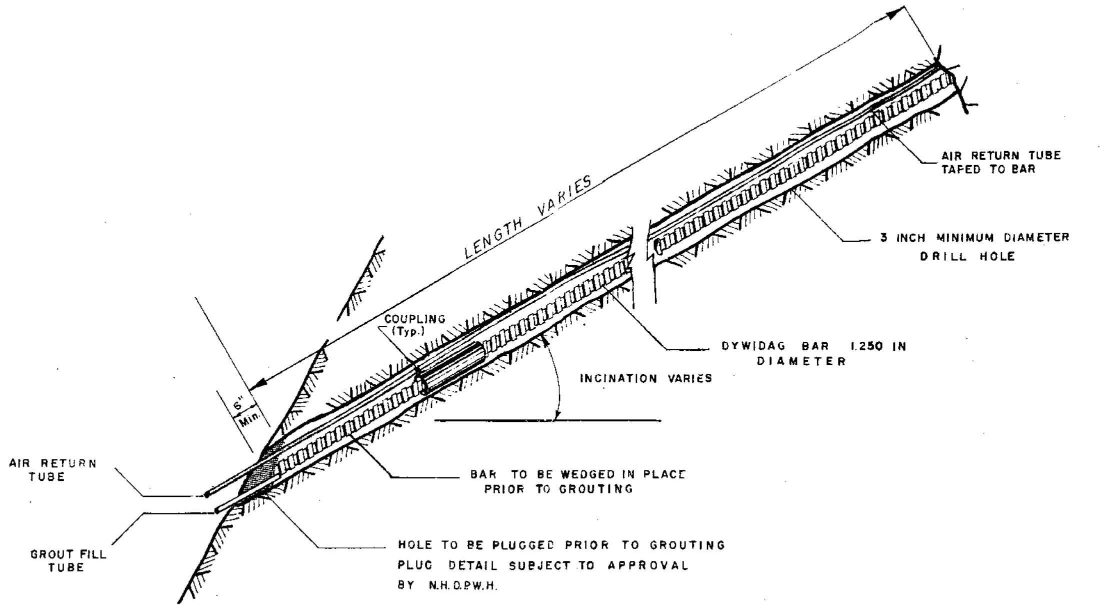


Figure 4. Detail of Rock Tendon (Haley and Aldrich, 1973(b))

Figure 6. Weathered Rock Along Open Rock Joints

Figure 7. Source of Groundwater Sample #1

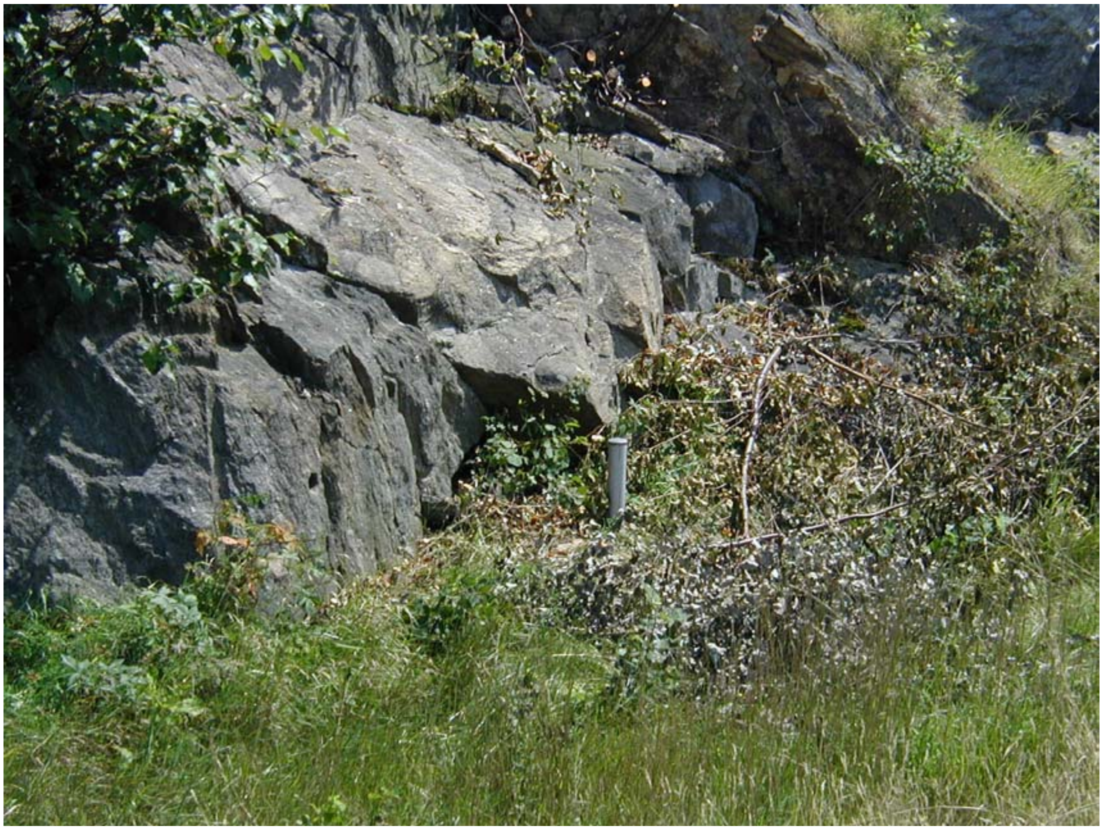


Figure 8. Well Used for Groundwater Sample #2

Figure 9. Typical Rock Bolt End Prepared for Testing

Figure 10. Rock Bolts Wired for Electrochemical Testing

Figure 11. Rock Tendon Element Wired for Electrochemical Testing

Figure 12. Photo of Ground Bed for Polarization Test

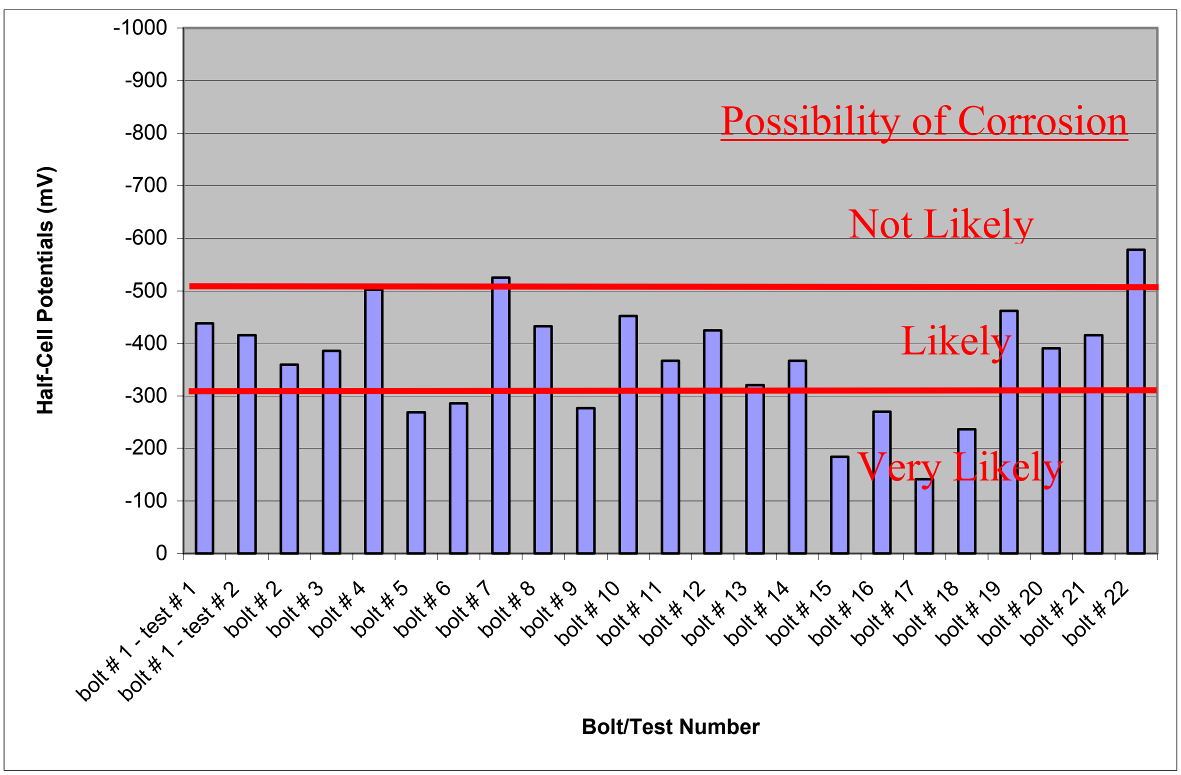


Figure 13. Half-Cell Potential of Rock Bolts

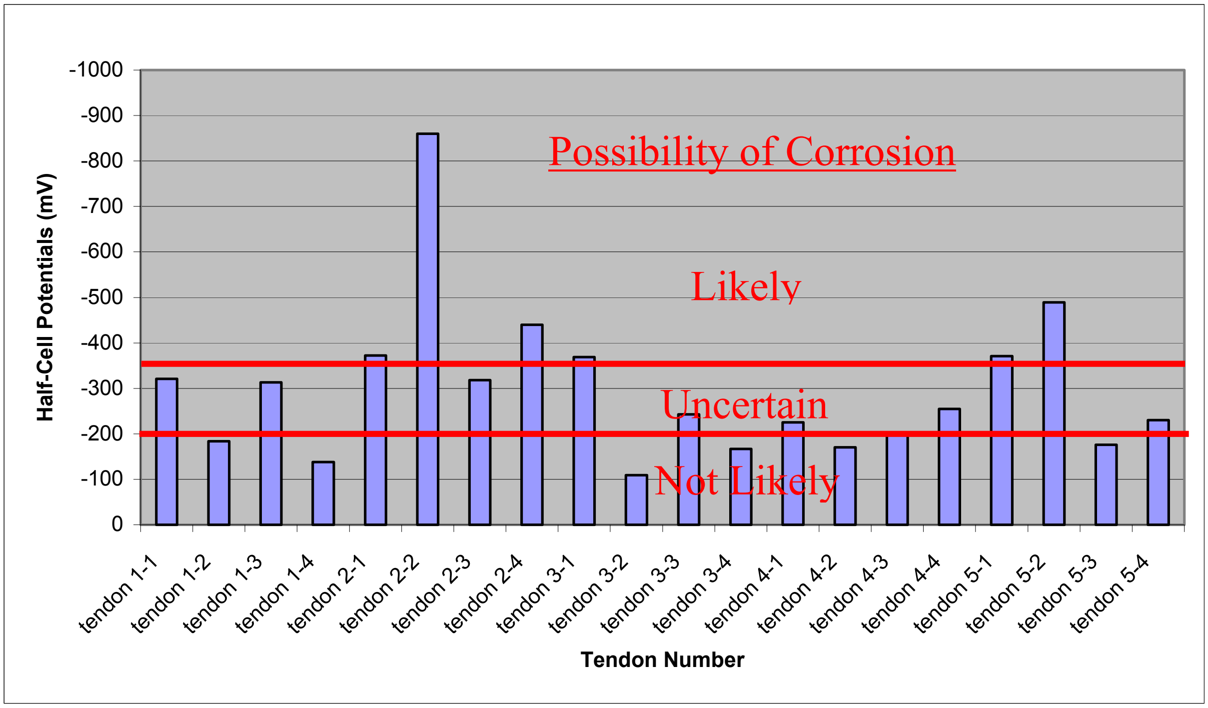
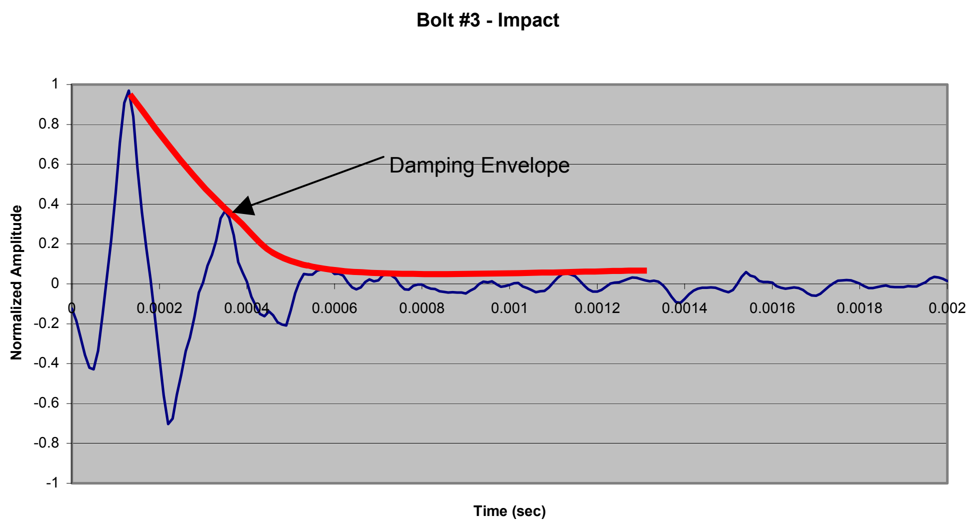
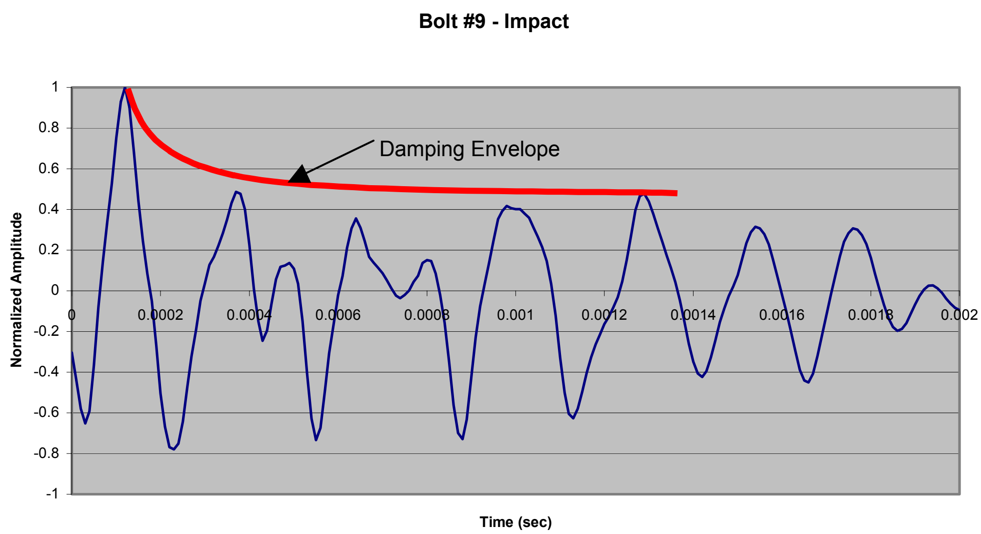


Figure 14. Half-Cell Potential Measurements for Tendon Elements

Figure 15. Leaching of Grout Near Tendon Sample 2-2.

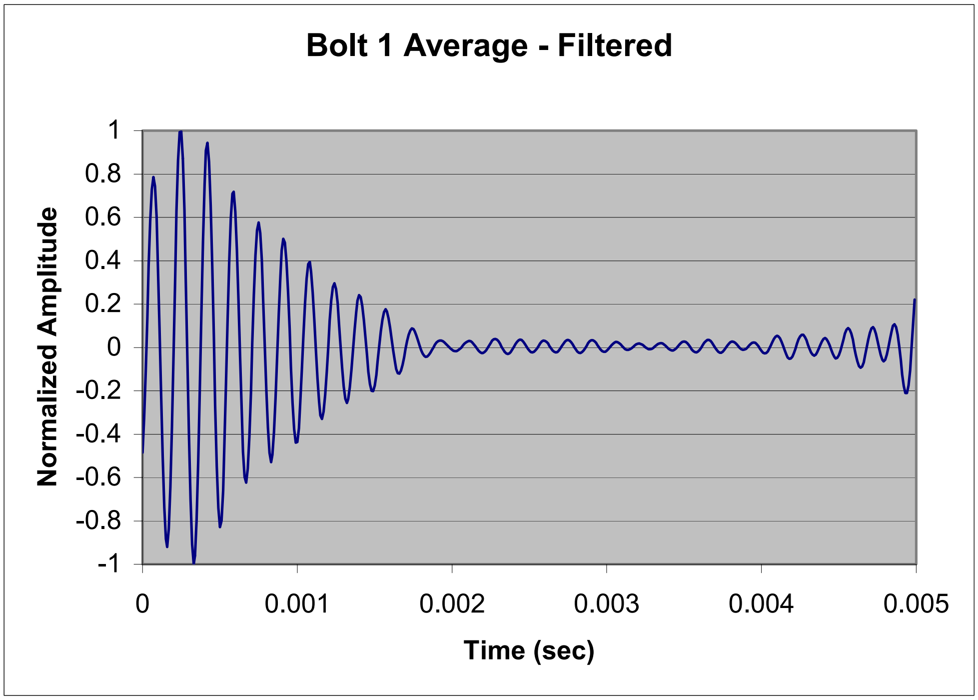


a) High rate of decay indicative of relatively high bolt load.

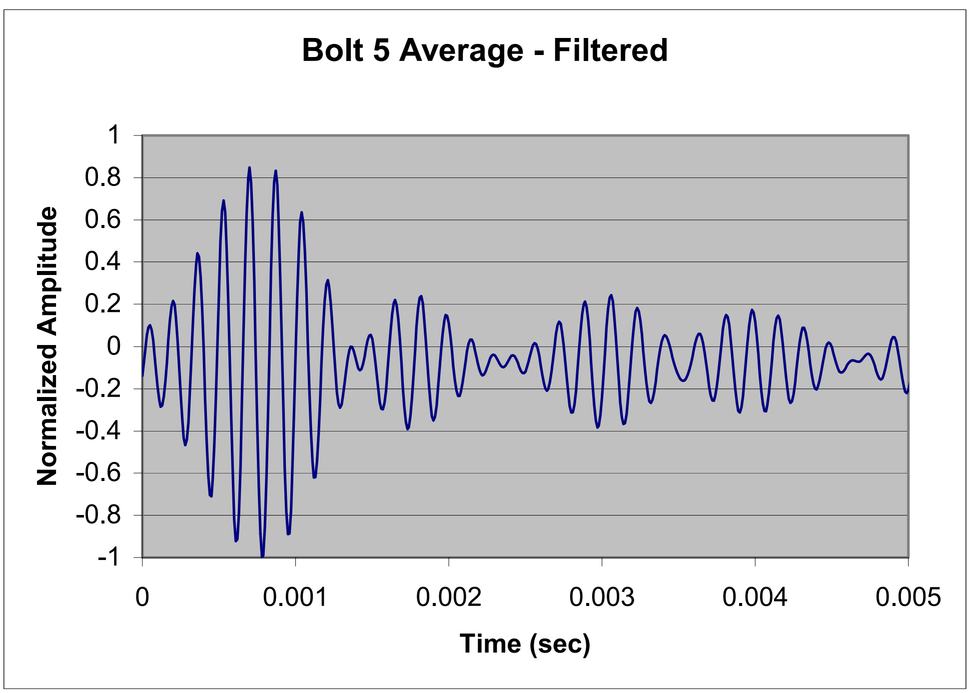


b) Low rate of decay indicative of prestress loss.

Figure 16. Typical damping envelopes from impact testing of rock bolts at Barron Mountain.



a Strong signal attenuation indicative of relatively “good” quality grout.



b) Weak signal attenuation indicative of relatively “poor” quality grout

Figure 17. Typical acceleration responses from impact tests of rock reinforcements at Barron Mountain.

Figure 19. Rockmass Character Near Sta. 1775+25

APPENDIX X

COST PROPOSAL FOR PHASE II

Overview

The proposal for Phase II of this research includes invasive testing of selected rock bolts and tendons to verify results from Phase I, including NDT, condition assessment and service-life estimates. Invasive testing shall include lift-off tests; and physical, chemical and metallurgical testing on steel and grout samples retrieved from exhumed reinforcements. Replacement bolts must be installed prior to invasive testing of the reinforcements. The following recommendations include details of the test program, estimated costs and a proposed schedule.

MMCE proposes that a total of five rock bolts be selected for lift-off tests and subsequently exhumed. In addition, two tendon reinforcements should be exhumed. The test program will include some reinforcements with questionable condition, and some reinforcements considered to be in good condition, based on the results from NDT. Table X-1 is a summary of the reinforcements we propose to include in the Phase II test program.

Table X-1. Reinforcements Proposed for Phase II Testing

Bolt #	Station	Block Number	Length (ft)	Condition Assessment (NDT)	Comments
4	1775+15	23	30	Good	No apparent loss of prestress; relatively good quality grout; not likely corroded
5	1775+00	23	25	Questionable	Apparent loss of prestress; relatively poor quality grout near free end; possible loss of cross section or kink in bolt; very likely corroded
17	1775+45	11	25	Questionable	No apparent loss of prestress; relatively poor quality grout for full length; very likely corroded
11	1774+15	22	25	Questionable	Apparent loss of prestress; relatively poor quality grout along near the bonded zone; likely corroded
22	1776+75	6	25	Good	No apparent loss of prestress; relatively poor quality grout for full length; not likely corroded

Table X-1. Reinforcements Proposed for Phase II Testing (continued)

Tendon #	Station	Approx. EL. (ft)	Length (ft)	Condition Assessment (NDT)	Comments
1-2	1774+50	735	60	Good	Relatively poor grout quality; not likely corroded
2-2	1775+00	735	60	Questionable	Relatively good grout condition; likely corroded

Description of Testing and Sampling

Lift-Off Tests

Lift-off tests will be performed on selected rock bolts as described in PTI (1996). Lift-off tests are conducted to confirm the magnitude of the load carried by the rock bolt. Load will be applied to the end of the rock bolt with a center hole hydraulic jack. The lift-off load is determined by applying load to the end of the rock bolts to lift the anchor nut off the bearing plate (without turning the nut). Subsequent to determining the liftoff load, the rock bolts will be overcored.

Overcoring

Five rock bolts and two tendons will be selected for exhumation by overcoring. Overcoring may be accomplished using a hydraulic drill unit traveling along a steel bar anchored to the rock face at an adjustable angle. Water shall be used to lubricate the bit and core barrels and to flush away the rock powder. The drilling process should be started after anchoring the drillmount into the rockface and aligning the drill and first core barrel with the bolt. Alignment of the drill and core barrel with respect to the bolt is critical if samples are to be retrieved from the full length of the reinforcement. If this is unsuccessful, short samples of the reinforcements may be retrieved within five to ten feet from the rock face by drilling a second hole to intercept the drill hole and cutoff the reinforcement. Therefore, exhuming the reinforcements may require that diamond drill bits be expended in drilling through steel reinforcements at the cut off location. The diameter of the overcore shall be larger than the diameter of the original drill hole such that the reinforcement and grout surrounding the reinforcement are retrieved. The drill hole diameter for the rock tendons is 3 inches, and for the rock bolts the drill hole diameter is approximately 1.625 inches (letter from NHDOT to Inland Ryerson dated December 11, 1973). A five inch core barrel should be sufficient for both cases, but the outer core of rock will need to be separated from the grout after removal from the drill hole.

There is some experience in the mining industry exhuming rock reinforcements. However, MMCE and could not identify any local contractors with prior experience. One contractor, with experience installing and testing rock reinforcements, was willing to make an attempt, but only on the basis of time and materials (i.e., the contractor does not guarantee success). MMCE expects that exhuming reinforcements will be a difficult task and **the chances for successfully exhuming reinforcements are uncertain.**

Reinforcements that are exhumed will be measured, sketched, photographed and carefully stored on-sight prior to being cut into manageable lengths. The outer part of the core will be split, the separated parts of the core will be photographed, and the steel reinforcements will be examined for signs of corrosion, loss of cross section or kinks/bends. Grout will also be examined to locate areas where cracking has occurred. Samples of cement and resin grout will be separated from the bars with a chisel and sent to the laboratory for testing. When applicable, grout samples will be selected from areas where corrosion of the reinforcements has been observed.

Steel reinforcements will be cut and samples will be selected for metallurgical analysis, and physical testing including laboratory tension tests. Bar diameters will be measured at a number of locations to document loss of cross section, and, if pitting is present, pit geometry and location will be documented.

Laboratory analysis

Grout samples will be subjected to chemical and physical testing. Absorption of the grout will be evaluated as described in AASHTO Test Method T85 " Specific Gravity and Absorption of Coarse Aggregate." These tests will be performed on samples of polymer resin and Portland cement grout. Test results will be useful to examine the porosity of the grout, and assess the potential for diffusion of chloride or sulfate ions through the grouted annulus surrounding the reinforcements and the effectiveness of the grout to coat the reinforcements.

Samples of resin grout will be tested for hardness as described by ASTM D2583 to examine the consistency of the mixture, and areas where grout properties may have degraded.

Portland cement grout samples will be tested for chloride ion content as described by AASHTO Test Method T260 "Sampling and Testing for Total Chloride Ion Content and Concrete Raw Materials." Chloride concentration will be evaluated relative to position within the grouted annulus surrounding the tendon. This data will be useful to study chloride diffusion and the corresponding potential for depassivation of steel surrounded by Portland cement grout.

Samples of metal reinforcements will be subject to tension tests as described by ASTM A370. This will involve application of a measured load sufficient to cause rupture. Percent elongation (strain) will also be measured and the corresponding stress-strain curve will be presented. Results will be compared to ASTM specifications for A722, Grade 150, prestressing steel.

If excessive corrosion, pitting type corrosion, or evidence of stress corrosion cracking or hydrogen embrittlement is observed, then metallurgical tests will be performed on samples of metal reinforcements. Metallurgical test will include spectrographic analysis to assess the metal composition, and metallographic examination to observe the microstructure of the thread bar material.

Additional NDT

The NDT tests performed during Phase I will be repeated on elements selected for invasive testing. These tests are necessary to verify the baseline measurements prior to invasive testing.

MMCE also proposes that additional NDT be performed on reinforcements located above Elevation 820, North of Station 1777+00 that could not be accessed during Phase I. The services of a specialty contractor, equipped to access areas in difficult terrain, will be required to perform this testing. This may be the same specialty contractor retained to perform invasive testing, and install replacement reinforcements. MMCE proposes to test approximately 10 rock bolts and five tendons within this area. Additional ground water and weathered rock samples will also be obtained if feasible.

Proposed Tasks

MMCE will participate in Phase II activities with cooperation from the NHDOT. NHDOT will be responsible for site access and traffic control. This will include use of Jersey Barriers, posting of appropriate traffic control devices, and use of a 40 ft. manlift and truck crane, similar to the equipment used during Phase I.

MMCE will coordinate activities, perform NDT, observe invasive tests and the condition of reinforcements extracted from the site, send selected samples to the laboratory for physical, chemical and metallurgical analysis, compare observed results with those from NDT, and render an opinion on the overall condition of reinforcements at the Barron Mountain site based on the results of NDT and invasive testing.

The following tasks are including within the scope of the Phase II condition assessment:

1. Subcontract for performing lift-off tests, exhuming reinforcements and installing replacement reinforcements.

2. Subcontract with laboratories for chemical and physical analysis of grout; metallurgical analysis, and tension testing of metal specimens.

3. Coordinate activities with NHDOT.

4. Visit site and perform fieldwork with contractor. Perform NDT, observe

lift off testing and exhumed reinforcements.

5. Prepare and send sample to laboratories for physical, chemical and metallurgical analysis.

6. Analyze data collected from Phase II and compare to results from Phase I

7. Prepare final report based on the results of Tasks 1-6. The report will include a summary and interpretation of results from Phase II, and recommendations for future monitoring and/or replacement and retrofit of existing reinforcements.

Schedule

MMCE expects to complete Phase II within an eight-month time frame. The proposed schedule, shown in Table X-2, assumes a project start date of May 1, 2004.

An onsite meeting with MMCE, NHDOT and the specialty contractor will be required as part of Task 1. The purpose of the meeting will be to clarify details of the scope, and discuss scheduling and coordination of activities for Phase II.

MMCE estimates that three weeks will be required for the fieldwork (Task 4), however, a three and a half month time frame is proposed to accommodate the schedules of MMCE, the specialty subcontractor and NHDOT.

Table X-2. Proposed Project Schedule for Phase II

Task	Description	Begin	End
1	Subcontract for liftoff tests, overcoring, and bolt replacement	5/01/04	6/01/04
2	Subcontract with laboratories	5/01/04	6/01/04
3	Coordinate with NHDOT	5/01/04	9/15/04
4	Perform fieldwork	6/01/04	9/15/04
5	Prepare and send samples to laboratory	6/01/04	9/15/04
6	Data analysis and interpretation	9/15/04	10/15/04
7	Final Report	10/15/04	12/15/04

Basis and Estimate of FEES

MMCE estimates the cost for Phase II to be approximately $122,000.00 and this includes approximately three weeks of fieldwork, and the cost of a contractor to test and exhume existing reinforcements and install replacements. MMCE has assumed that the NHDOT will provide traffic control and access to the site.

Itemized Budget

MMCE's estimated costs for the proposed research are presented in Table X-3. The estimated Level of Effort by Task is presented in Table X-4 and an Itemized Summary Budget by Task is presented in Table X-5.

Table X-3 – Phase II Itemized Summary Budget for MMCE

(a)	Direct Salaries:		
	K.L. Fishman	$ 7875.00	
	Senior Engineer	$ 0.00	
	Project Engineer	$ 5356.00	
	Senior Technician	$ 720.00	
	Word Processor	$ 520.00	
	CAD Operator	$ 180.00	
	(a1) Subtotal		**$ 14,651.00**
	Overhead (124% × a1)		**$ 18,167.24**
(b)	Borrowed Personnel:	$ 0.00	
(c)	Consultants:	$ 0.00	
(d)	Subcontracts:		
	Rock Reinforcement Contractor	$ 55000.00	
(e)	Equipment Cost	$ 3500.00	
(f)	Materials and Services:		
	Metallurgy Lab	$ 2000.00	
	Structural Testing Lab	$ 3000.00	
	Grout Testing Lab	$ 3500.00	
(g)	Copying and Shipping:	$ 1250.00	
(h)	Travel:	$ 4990.00	
	Subtotal (b)+(c)+(d) + (e) + (f) + (g) + (h)		**$ 73,240.00**
	Total		**$ 106,058.24**
(i)	Fixed Fee (Total × 15.0%)		**$ 15,908.74**
	Grand Total		**$ 121,966.98**

Assumptions Used to Develop Scope and Cost Estimate

1. Travel expenses for Task 1 includes round trip mileage between Buffalo, NY and the job site, two travel days and three days per diem.

2. Similar to Phase I, MMCE assumes that NHDOT will perform laboratory tests on samples of weathered rock and groundwater as applicable.

3. Agreements with outside laboratory facilities are needed for tension testing of metal samples, metallurgical analysis, and testing of grout

samples. If testing is performed by NHDOT, a cost adjustment may be applied to Tasks 2 and 5.

4. MMCE estimates that the following numbers of samples will be included in the laboratory test program as part of Task 5:

Description	**No. of Samples**
Tension tests of steel reinforcements	40
Metallurgical examination of steel reinforcement	10
Absorption and specific gravity test on grout samples	40
Chloride ion content on samples of Portland cement grout	30
Hardness test for resin grout samples	30

5. MMCE will perform NDT as part of Task 4. NDT, performed in Phase I, will be repeated on five rock bolts and 2 tendon reinforcements identified for invasive testing as shown in Table X-1. In addition, NDT will be performed on 10 more rock bolts and 5 more tendon reinforcements if access is provided to reinforcements located above Elevation 820 ft, North of Station 1777+00.

6. Travel expenses for Task 4 includes round trip mileage between Buffalo, NY and the job site, two travel days and per diem for the duration of the fieldwork. MMCE assumes that the fieldwork will be completed in 15 days and will require two persons for nine of the days, and three persons for another six days.

7. Contractor costs included in Task 4 include a three man crew plus equipment for 12 days of fieldwork, and the cost of materials for replacement bolts. The contractor's costs include a 10% mark-up to the costs of materials. The contractor makes no guarantee as to the number of reinforcements that may be exhumed. MMCE expects that exhuming reinforcements will be a difficult task and could not identify any local contractors with prior experience. **Therefore, the chances for successfully exhuming reinforcements are uncertain.**

8. MMCE suggests that the NHDOT administer the subcontract for installation of replacement bolts, lift-off tests and extraction of in-service reinforcements. Currently, Table X-3 includes this as subcontractor costs.

9. The budget for Task 4 assumes that NHDOT will provide traffic control at the site and access to the reinforcements located below El. 820 ft. Site access will include use of a crane and basket and a forty foot man-lift to access bolts and tendons along the rock face.

10. MMCE anticipates that the rock bolt contractor selected for Task 4, will provide access to reinforcements located above El. 820 ft., north of Station 1777+00. Special rock climbing equipment and techniques may be required to access these reinforcements.

Table X-4. Barron Mountain Project: Phase II Level of Effort by Tasks (Person-Hours and Costs)

Principal Staff Members	Role in Study	Time* (%) Over Contract Period	Task 1	Cost ($)	Task 2	Cost ($)	Task 3	Cost ($)	Task 4	Cost ($)	Task 5	Cost ($)	Task 6	Cost ($)	Task 7	Cost ($)	**Total Hours**	Hourly Rate ($)	**Total Cost ($)**
MMCE																			
K.L. Fishman	Principal Investigator	31.3	30	787.50	10	262.50	10	262.50	150	3937.50	20	525.00	20.00	525.00	60.00	1575.00	300	26.25	7875.00
Staff	Senior Engineer	0.0		0.00		0.00		0.00		0.00		0.00		0.00		0.00	0	21.20	0.00
Staff	Engineer	27.2	20	412.00	10	206.00		0.00	150	3090.00	40	824.00	40.00	824.00		0.00	260	20.60	5356.00
Staff	Senior Technician	8.4		0.00		0.00		0.00		0.00	60	540.00	20.00	180.00		0.00	80	9.00	720.00
Staff	Word Processor	4.2	20	260.00		0.00		0.00		0.00		0.00		0.00	20.00	260.00	40	13.00	520.00
Staff	CAD Operator	2.1		0.00		0.00		0.00		0.00	0	0.00		0.00	20.00	180.00	20	9.00	180.00
Σ Direct labor				**1459.50**		**468.50**		**262.50**		**7027.50**		**1889.00**		**1529.00**		**2015.00**			**14651.00**
Overhead				**1809.78**		**580.94**		**325.50**		**8714.10**		**2342.36**		**1895.96**		**2498.60**			**18167.24**
Total Hours			**70**		**20**		**10**		**300**		**120**		**80**		**100**		**700**		
Total Labor				**3269.28**		**1049.44**		**588.00**		**15741.60**		**4231.36**		**3424.96**		**4513.60**			**32818.24**
Expenses				**490.00**		**0.00**		**0.00**		**60500.00**		**12000.00**		**0.00**		**250.00**			**73240.00**
Fee (15%)				**563.89**		**157.42**		**88.20**		**11436.24**		**2434.70**		**513.74**		**714.54**			**15908.74**
Totals				**4323.17**		**1206.86**		**676.20**		**87677.84**		**18666.06**		**3938.70**		**5478.14**			**121966.98**

Table X-5. PHASE II: ITEMIZED SUMMARY BUDGET BY TASK

	COSTS						
Task	Wages	Overhead	Capital Equipment	Materials & Services	Commun. & Shipping	Travel	Total
Task 1	1,459.50	1,809.78	0.00	0.00	0.00	490.00	3,759.28
Task 2	468.50	580.94	0.00	0.00	0.00	0.00	1,049.44
Task 3	262.50	325.50	0.00	0.00	0.00	0.00	588.00
Task 4	7,027.50	8,714.10	1,000.00	55,000.00	0.00	4,500.00	76,241.60
Task 5	1,889.00	2,342.36	2,500.00	8,500.00	1,000.00	0.00	16,231.36
Task 6	1,529.00	1,895.96	0.00	0.00	0.00	0.00	3,424.96
Task 7	2,015.00	2,498.60	0.00	150.00	100.00	0.00	4,763.60
Total	**14,651.00**	**18,167.24**	**3,500.00**	**63,650.00**	**1,100.00**	**4,990.00**	**106,058.24**

McMahon & Mann
Consulting Engineers, P.C.

APPENDICES I-IX

PHASE I:
CONDITION ASSESSMENT AND EVALUATION OF ROCK REINFORCEMENT

ALONG I-93

BARRON MOUNTAIN ROCK CUT

WOODSTOCK, NEW HAMPSHIRE

Prepared for:

The New Hampshire Department of Transportation
Bureau of Materials and Research
Concord, New Hampshire

***Prepared by*:**

McMahon & Mann Consulting Engineers, P.C.
2495 Main Street
Buffalo, New York

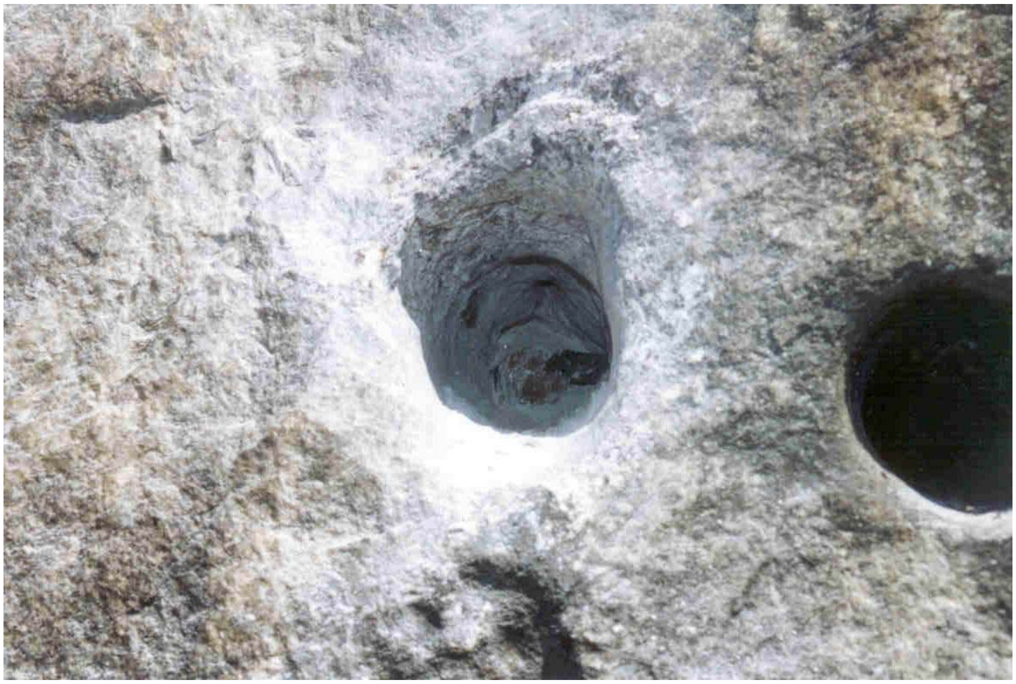
February 2004
File: 03-024

APPENDICES

I	Exposed Ends Of Rock Tendons
II	Half-Cell Locations
III	Lab Test Results From NHDOT
IV	Detailed Calculations For Service-Life Estimates
V	Polarization Curves
VI	Damping Envelopes For Rock Bolts
VII	Rock Bolts: Impact Acceleration Time Histories
VIII	Rock Tendons: Impact Acceleration Time Histories
IX	Rock Bolts: Ultrasonic Test Data
X	Cost Proposal For Phase II (Included under separate cover with report)

APPENDIX I

EXPOSED ENDS OF ROCK TENDONS



Photograph 1 – Exposed end of Tendon 1-1 after chipping

Photograph 2 – Exposed end of Tendon 2-1 after chipping

Photograph 3 – Exposed end of Tendon 2-2 after chipping

Photograph 4 – Exposed end of Tendon 3-1 after chipping


Photograph 5 – Exposed end of Tendon 3-2 after chipping


Photograph 6 – Exposed end of Tendon 4-1 after chipping

Photograph 7 – Exposed end of Tendon 4-2 after chipping

Photograph 8 – Exposed end of Tendon 5-1 after chipping

Photograph 9 – Exposed end of Tendon 5-2 after chipping

APPENDIX II

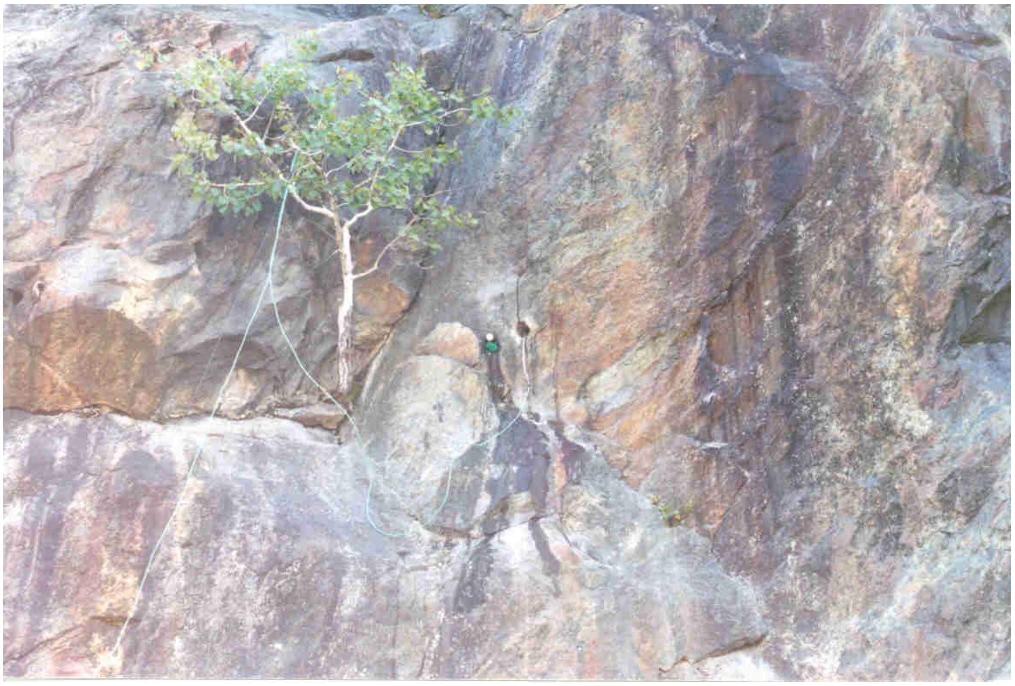
HALF-CELL LOCATIONS

Table II-1. Half-Cell Locations

Tendon #	Half-Cell Location
1-1	Drill-hole adjacent to Tendon 1-1
1-2	Drill-hole adjacent to Tendon 1-1
1-3	Drill-hole approx. 6” North of Tendon 1-4 ; see Appendix II, Photo #1
1-4	Drill-hole approx. 6” North of Tendon 1-4; see Appendix II, Photo #1
2-1	Drill hole approx. 18” South and above Tendon 2-1
2-2	Drill hole approx. 18” South and above Tendon 2-1
2-3	Approx. 10 ft. North of Tendon 2-3
2-4	Vertical drill-hole approximately 3 ft. above Tendon 2-4; see Appendix II, Photo #2
3-1	Rock joint approx. 4 ft. North and above Tendon 3-2; see Appendix II, Photo #3
3-2	Rock joint approx. 4 ft. North and above Tendon 3-2
3-3	Rock joint between and to the North of Tendons 3-3 and 3-4
3-4	Rock joint between and to the North of Tendons 3-3 and 3-4
4-1	Rock joint approx. 3 ft. S. of Tendon 4-1 and 5 ft. N. of Tendon 4-2; see Appendix II, Photo #4
4-2	Rock joint approx. 3 ft. S. of Tendon 4-1 and 5 ft. N. of Tendon 4-2
4-3	Rock joint approx. 5 ft. South of Tendon 4-3
4-4	Rock joint approx. 5 ft. South of Tendon 4-3
5-1	Drain hole approx. 5 ft. South of Tendon 5-3; see Appendix II, Photo #5
5-2	Drain hole approx. 5 ft. South of Tendon 5-3; see Appendix II, Photo #5
5-3	Rock joint and plastic conduit approx. 7 ft. above Tendon 5-3; see Appendix II, Photo #6
5-4	Rock joint and plastic conduit approx. 7 ft. above Tendon 5-3

Table II-1. Half-Cell Locations (cont.)

Rock Bolt #	Half-Cell Location
1	Drill-hole 8 in. South of Bolt 2; see Appendix II, Photo #7
2	Drill-hole 8 in. South of Bolt 2; see Appendix II, Photo #7
3	Rock joint between bolts 3 and 4; see Appendix II, Photo #8
4	Rock joint between bolts 3 and 4; see Appendix II, Photo #8
5	Approx. 10 ft. North of Tendon 2-3
6	Approx. 10 ft. North of Tendon 2-3
7	Approx. 3 ft. South and above Bolt 8
8	Approx. 3 ft. South and above Bolt 8
9	Approx. 1.5 ft above Bolt 9; see Appendix II, Photo #9
10	Approx. 1.5 ft above Bolt 9; see Appendix II, Photo #9
11	Drill-hole below Bolt 11; see Appendix II, Photo #10
12	Drill-hole below Bolt 11; see Appendix II, Photo #10
13	Drill hole south of Bolt 14; see Appendix II, Photo #11
14	Drill hole south of Bolt 14; see Appendix II, Photo #11
15	Rock joint approx. 8 ft. above and South of Bolt 15; see Appendix II, Photo #12
16	Rock joint approx. 8 ft. above and South of Bolt 15; see Appendix II, Photo #12
17	Rock joint approx. 10 ft. South of Bolt 18; see Appendix II, Photo #13
18	Rock joint approx. 10 ft. South of Bolt 18; see Appendix II, Photo #13
19	Rock joint above and South of Bolts 19, 20; see Appendix II, Photo #14 and 15
20	Rock joint above and South of Bolts 19, 20; see Appendix II, Photo #14 and 15
21	Crevice North of bolt 21; see Appendix II, Photo #16
22	Crevice North of bolt 21; see Appendix II, Photo #16



Photograph 1 – Half-cell location for Tendons 1-3 and 1-4.

Photograph 2 – Half-cell location for Tendon 2-4

Photograph 3 – Half-cell location for Tendons 3-1 and 3-2. Half-cell is placed in approximately two-thirds of the way up the crevice in the center of the photograph.

Photograph 4 – Half-cell location for Tendons 4-1 and 4-2. Half-cell is located at the intersection of the vertical and horizontal crevices in the center of the photograph.

Photograph 5 – Half-cell location for Tendons 5-1 and 5-2. Half-cell is placed in the hole directly above the water stain (right).

Photograph 6 – Half cell location for Tendons 5-3 and 5-4. Half-cell is placed beneath the black cable conduit approximately three-quarters of the way up the vertical section located in the center of the photograph.

Photograph 7 – Half-cell location for Bolts 1 and 2. Half-cell was placed in a hole to the right of Bolts 1 and 2 (center).

Photograph 8– Half-cell location for Bolts 3 and 4. Half-cell was placed in a vertical hole between Bolts 3 and 4 (center).

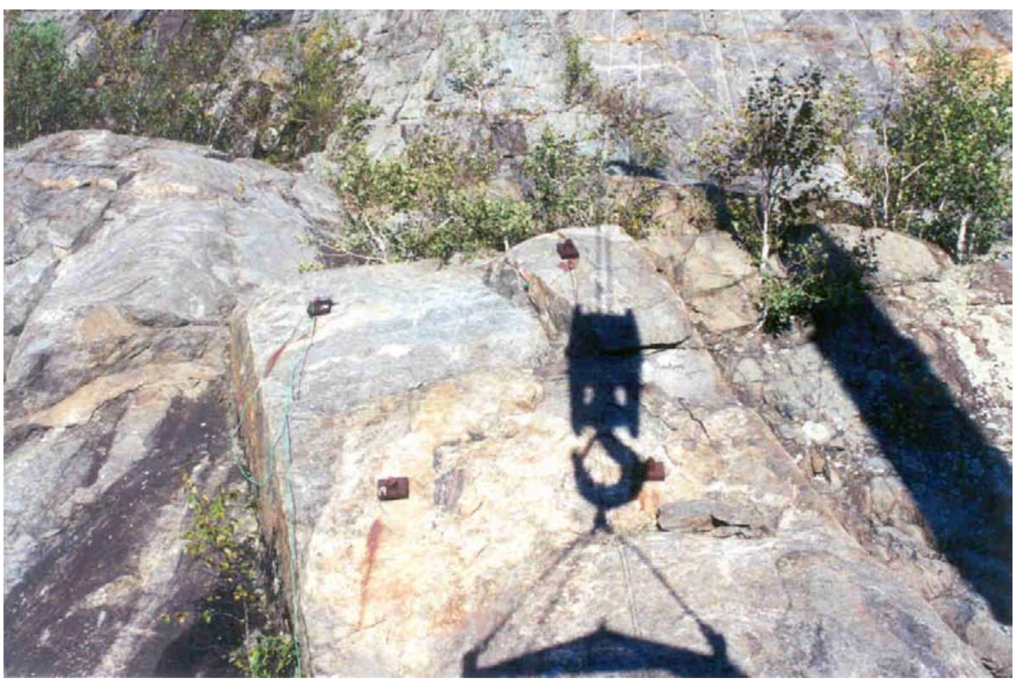
Photograph 9 – Half-cell location for Bolts 9 and 10. Half-cell was placed in a hole directly above Bolt 9 (center).

Photograph 10 – Half cell location for Bolts 11 and 12. Half-cell was placed in a hole between and below Bolts 11 and 12 (lower center).

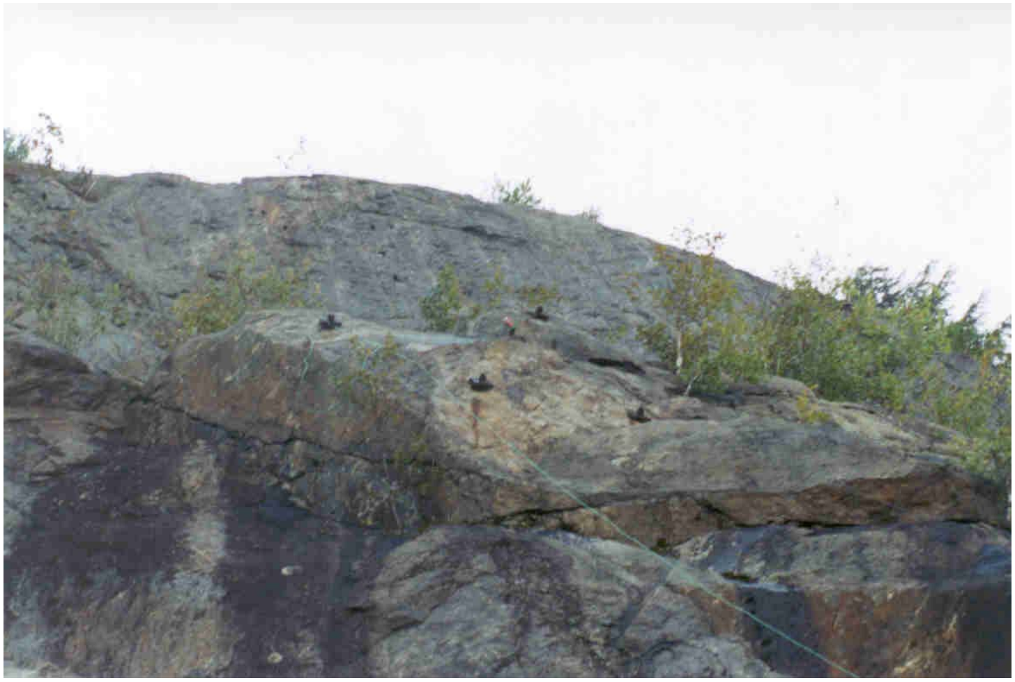
Photograph 11 – Half-cell location for Bolts 13 and 14. Half-cell was placed in a hole to the right of Bolts 13 and 14 (left center).

Photograph 12 – Half-cell location for Bolts 15 and 16. Half-cell was placed in the horizontal crevice, right of the near vertical crevice, in the center of the photo.

Photograph 13 – Half-cell location for Bolts 17 and 18. Half-cell was placed in a small crack approximately 10 feet to the right of Bolt 18.



Photograph 14 – Half-cell location for Bolts 19 and 20. Half-cell was placed in the crack approximately 10 feet to the right of Bolt 20.



Photograph 15 - Half-cell location for Bolts 19 and 20. Half-cell was placed in the crack approximately 10 feet to the right of Bolt 20.

Photograph 16 – Half-cell location for Bolts 21 and 22. Half-cell was placed in the crack approximately 3 feet to the left of Bolt 21.

APPENDIX III

LAB TEST RESULTS FROM NHDOT

Sample 1

Tag #90

— STATION 75+25

— Elevation 6' From bottom of ditch

r: Dickerman, Rick
of New Hampshire - DOT
Date: 09/02/2003 Time: 10:24:54

Result entry for 1 sample from c:\lims\user\~ps1.tmp
Result entry template: ACROSS
Page: 1

User Info			Sample	AA11021
			Loc Code	GRAVEL
			Descript	Gravel, Fine &
			PROJECT	
			JOB_MIX	
			CYLINDER	
IREMARK	Remarks		Result	[redacted]
WTCOR	dry weight coarse Agg - Soils	lb	Result	3.38
SIV_A1	Gilson 6"_150 mm sieve	lb	Result	0
SIV_F1	Gilson 3"_75 mm sieve	lb	Result	0
SIV_H1	Gilson 2"_50 mm sieve	lb	Result	0
SIV_I1	1 1/2" (37.5 mm) sieve	lb	Result	0
SIV_K1	1" (25 mm) sieve	lb	Result	0
SIV_L1	3/4 in (19 mm) Sieve	lb	Result	0
SIV_M1	1/2 in (12.5 mm) Sieve	lb	Result	.05
SIV_O1	#4 (4.75 mm) Sieve	lb	Result	.33
WTFN	dry weight fine Agg - Soils	gm	Result	499.5
SIV_R3	8"_no 10 sieve	gm	Result	66.1
SIV_S3	8"_no 20 sieve	gm	Result	181.7
SIV_U3	8"_no 40_0.425mm sieve	gm	Result	292.7
Y3	8"_no 100_0.150mm sieve	gm	Result	412.4
Z3	8"_no 200_0.075mm sieve	gm	Result	465.4
SVA1P	6 in (150 mm) Sieve	% Passin	Result	100.0
SVF1P	3 in (75 mm) Sieve	% Passin	Result	100.0
SVH1P	2 in (50 mm) Sieve	% Passin	Result	100.0
SVI1P	1 1/2 in (37.5 mm) Sieve	% Passin	Result	100.0
SVK1P	1 in (25 mm) Sieve	% Passin	Result	100.0
SVL1P	3/4 in (19 mm) Sieve	% Passin	Result	100.0
SVM1P	1/2 in (12.5 mm) Sieve	% Passin	Result	98.5
SVO1P	#4 (4.75 mm) Sieve	% Passin	Result	[redacted] 90.2
SVR3P	No 10 (2.00 mm) Sieve	% Passin	Result	86.8
SVS3P	No 20 (0.850 mm) Sieve	% Passin	Result	63.6
SVU3P	No 40 (0.425 mm) Sieve	% Passin	Result	41.4
SVY3P	#100 (0.150 mm) Sieve	% Passin	Result	17.4
SVZ3P	#200 (0.075 mm) Sieve	% Passin	Result	6.8
ANALYST	Tested By		Result	
REVIEW	Reviewed by		Result	

P.H. = 5.12
% Moisture 5.5

Sample #2

#73

– STATION 76+85
– ELEVATION 15' FROM Bottom

: Dickerman, Rick
of New Hampshire - DOT
Date: 09/02/2003 Time: 10:32:34

Result entry for 1 sample from c:\lims\user\~ps1.tmp
Result entry template: ACROSS
Page: 1

P.H. = 4.24
% Moisture
3.9%

User Info			Sample:	AA11034
			Loc Code	GRAVEL
			Descript	Gravel Fine &
			PROJECT	
			JOB_MIX	
			CYLINDER	
IREMARK	Remarks		Result	
WTCOR	dry weight coarse Agg. + Soils	lb	Result	2.54
SIV_A1	Gilson 6" - 150 mm sieve	lb	Result	0
SIV_F1	Gilson 3" - 75 mm sieve	lb	Result	0
SIV_H1	Gilson 2" - 50 mm sieve	lb	Result	0
SIV_I1	1 1/2" (37.5 mm) sieve	lb	Result	0
SIV_K1	1" (25 mm) sieve	lb	Result	.04
SIV_L1	3/4 in (19 mm) Sieve	lb	Result	.26
SIV_M1	1/2 in (12.5 mm) Sieve	lb	Result	.54
SIV_O1	#4 (4.75 mm) Sieve	lb	Result	1.07
WTFN	dry weight fine Agg. + Soils	gm	Result	375.8
SIV_R3	8" - no 10 sieve	gm	Result	91.3
SIV_S3	8" - no 20 sieve	gm	Result	167.7
SIV_U3	8" - no 40 - 0.425mm sieve	gm	Result	215.5
SIV_Y3	8" - no 100 - 0.150mm sieve	gm	Result	277.1
SIV_Z3	8" - no 200 - 0.075mm sieve	gm	Result	317.1
SVA1P	6 in (150 mm) Sieve	% Passin	Result	100.0
SVF1P	3 in (75 mm) Sieve	% Passin	Result	100.0
SVH1P	2 in (50 mm) Sieve	% Passin	Result	100.0
SVI1P	1 1/2 in (37.5 mm) Sieve	% Passin	Result	100.0
SVK1P	1 in (25 mm) Sieve	% Passin	Result	98.4
SVL1P	3/4 in (19 mm) Sieve	% Passin	Result	89.8
SVM1P	1/2 in (12.5 mm) Sieve	% Passin	Result	78.7
SVO1P	#4 (4.75 mm) Sieve	% Passin	Result	57.9
SVR3P	No 10 (2.00 mm) Sieve	% Passin	Result	75.7
SVS3P	No 20 (0.850 mm) Sieve	% Passin	Result	55.4
SVU3P	No 40 (0.425 mm) Sieve	% Passin	Result	42.7
SVY3P	#100 (0.150 mm) Sieve	% Passin	Result	26.3
SVZ3P	#200 (0.075 mm) Sieve	% Passin	Result	[redacted] 15.6
ANALYST	Tested By		Result	
REVIEW	Reviewed by		Result	

New Hampshire DOT

DETERMINING MINIMUM LABORATORY SOIL RESISTIVITY

AASHTO Designation: T288-91 (2000)

DATE: October 20, 2003
PROJECT: Barron Mountain Rock Reinforcement Evaluation, Woodstock, NH
PROJECT NO.: 13733L

APPARATUS:

Iris Instruments -- Syscal Jr. Switch 48	4-Probe plexiglas test box
Mode -- Rho mode	AB/2 = 0.36 ft.
Electrical Array -- Schlumberg sounding	MN = 0.42 ft. = 12.8 cm.
Measurement Time -- 1000 msec	Line = 0.71 ft. = 21.6 cm
Stacks -- 3	K = 1 per resistivity test box literature
V=MilliVolts	
I=MilliAmps	

TEST SAMPLE: Location -- Station 1777+00

Air Dry Weight = 1500 gms.
Natural Moisture = 7.2%

Sample #2

Distilled Water (ml.)		Resistivity Data			Min. Average
Increment	Cumm.	V	I	Ro (ohm-cm)	Ro (ohm-cm)
160	160	2107.472	0.5	4215	
90	250	2343.284	0.12	19527	
100	350	2496.782	0.23	10856	
100	450	2671.145	0.35	7632	
100	550	2580.444	0.38	**6791**	
100	650	2566.336	0.39	**6580**	**6685.5**

TEST SAMPLE: Location -- Station 1775+50

Air Dry Weight = 1500 gms.
Natural Moisture = 13.0%

Sample #1

Distilled Water (ml.)		Resistivity Data			Min. Average
Increment	Cumm.	V	I	Ro (ohm-cm)	Ro (ohm-cm)
150	150	2342.509	0.08	29281	
100	250	2415.687	0.15	16105	
100	350	2548.289	0.24	**10618**	
100	450	2589.276	0.27	**9590**	
100	550	2632.086	0.24	**10967**	**10392**

NHDOT Laboratory Report

Barron Mountain Rock Reinforcement Evaluation -- 13733L

November 2003

WATER

TEST	Sample #1	Sample #2
pH	6.92	6.39
Cl^-	6ppm	6 ppm
MG^+	0.833 ppm	1.48 ppm
SO_4	5.76 ppm	13.2 ppm

SOIL

TEST	Sample #1	Sample #2
Cl^-	0.072%lbs Cl^-/yd^3 (720 ppm)	0.025%lbs Cl^-/yd^3 (250 ppm)
MG^+	0.27% (2702 ppm)	0.48% (4793 ppm)
SO_4	652 ppm	ND

eastern analytical, inc.

professional laboratory services

PRELIMINARY ANALYTICAL RESULTS

Fax results have not been subjected to a final QA/QC review.
If you have any questions on faxed data, please contact us.

Fax Cover Sheet

To: Denis Boisvert — **EAI ID#:** 38432

Company: New Hampshire Dept of — **Client ID:** Barron Mountain, Woodstock / 13733L

Company FAX: 603 271-8700 — **Date Received:** 9/19/2003

Project Manager FAX:

New Hampshire Dept of Transportation FAXing Info: None

Denis Boisvert FAXing Info: None

9/23/03

Complete Report: 2pgs

Partial Report:

Wet Chem	Met	VOC	TPH	COC
✓				

Invoice	Pest/PCB	ABN/PAH	EPH	Field	SUB

Upcoming Events

19th Contaminated Soils, Sediments & Water Annual Conference — **October 20-23, 2003**
University of Massachusetts at Amherst
visit us at our booth
for more information call 413-545-1239

LABORATORY REPORT

Eastern Analytical, Inc. ID#: 38432

Client: **New Hampshire Dept of Transportation** Client Designation: **Barron Mountain, Woodstock / 13733L**

Sample ID:	#1 Sta. 1774 + 70	#2 Sta. 1777 + 00					
Matrix:	aqueous	aqueous					
Date Sampled:	9/11/03	9/11/03					
Date Received:	9/19/03	9/19/03		**Units**	**Date of Analysis**	**Method**	**Analyst**
Ammonia	< 0.05	< 0.05		mg/L	9/22/03	350.3	KL

APPENDIX IV

DETAILED CALCULATIONS FOR SERVICE LIFE ESTIMATES

Assessment of Corrosion Potential

Laboratory Test Results

Weathered Rock.

Test data submitted by NHDOT are included in Appendix III. Table IV-1 is a summary of the results from testing samples of weathered rock.

Table IV-1. Summary of Laboratory Test Results for Weathered Rock

Sample #	% Pass #4	%Pass #200	w %	pH	Resist. Ω-cm	SO_4 ppm	Cl^- ppm
1	90.2%	6%	13	5.1	9590	652	720
2	57.9%	9%	7	4.2	4215	ND	250

These results will be used to access the aggressiveness of the environment relative to corrosion of the rock bolts. The Recommendations Clouterre (FHWA, 1993) considers four main assessment parameters used to evaluate the corrosiveness of soils, including type of soil, soil resistivity, moisture content and pH. Each parameter is assigned a numerical weight, as shown in Table IV-2, that depends on features of the soil. The corrosiveness of the soil is shown as a global index ∑A, obtained from adding together the weighting factors for each of the four evaluation criteria. The last column in the table corresponds to conditions at the Barron Mountain site where the weathered rock samples are described as a sandy cohesionless soil, with resistivity greater than 5000 Ω-cm, seeping joints corresponding to a variable water table, and pH between 4 and 5.

Table IV-2. Overall Corrosiveness Index (Clouterre, 1991)

Criterion	Features	Weight A of Criterion	Barron Mtn.
Type of Soil	Texture - heavy, plastic, sticky impermeable - clayey-sand - light, permeable, sandy, cohesionless soils Peat and bog/marshlands Industrial Waste - clinker, cinders, coal - builder's waste (plaster, bricks) Polluted Liquids - wastewater, industrial - water containing de-icing salts	 2 1 **0** 8 8 4 6 8	 **0**
Resistivity	< 1000 Ω-cm 1000 Ω-cm to 2000 Ω-cm 2000 Ω-cm to 5000 Ω-cm > 5000 Ω-cm	5 3 2 **0**	 **0**
Moisture Content	Water table- brackish water (variable or permanent) Water table – pure water (variable or permanent) Above water table moist soil (water content > 20%) Above water table dry soil (water content < 20%)	 8 **4** 2 0	 **4**
pH	< 4 4 to 5 5 to 6 > 6	4 **3** 2 0	 **3**
	Global Index	Sum of above: ∑A	7

The resulting global index is 7, which corresponds to average corrosiveness (Classification III) as shown in Table IV-3. If a resistivity less than 5000 Ω-cm is considered, the classification is degraded to II, corresponding to corrosive. Given the range of measurements for resistivity, the classification for the weathered rock is considered to be between II and III.

Table IV-3. Corrosiveness of Soils (Clouterre, 1991)

Index $\sum A$	Soil Features	Classification
>13	Highly corrosive	I
9 to 12	Corrosive	II
5 to 8	Average corrosiveness	III
< 4	Slightly corrosive	IV

Ground Water

The following is a summary of the measurements made on sample #1 and #2 on Thursday, September 11, 2003 by MMCE during sample retrieval.

	pH	Temp.	
#1	5.78	27.9° C	From bolt hole ~ sta. 1774+75
#2	5.61	20.6° C	From well ~ sta. 1777+00

Table IV-4, from Xanthakos (1991), describes the range of parameters used for qualitatively assessing the potential aggressiveness of groundwater. Results from laboratory test performed by NHDOT, and presented in Appendix III, indicate that concentrations of magnesium, sulfate and ammonium ions in the groundwater are well below the limits described in Table IV-4. Based on these observations, a measured pH~5.6, and the information described in Table IV-4, the aggressiveness of the groundwater is considered weak.

Table IV-4. Parameter Limits for Aggressive Groundwater Conditions
(Modified after Xanthakos, 1991)

Test	Aggressiveness		
	Weak	Strong	Very Strong
pH	6.5-5.5	5.5-4.5	<4.5
Lime-dissolving CO_2, mg/l	15-30	30-60	>60
Ammonium (NH_4^+), mg/l	15-30	30-60	>60
Magnesium (Mg^{2+}), mg/l	100-300	300-1500	>1500
Sulfate (SO_4^{2-}), mg/l	200-600	600-3000	>3000

Estimated Remaining Service Life

The following equation, proposed by Romanoff (1957), is used to estimate corrosion rate and corresponding service life of buried metal reinforcements:

$$X = Kt^r \tag{IV-1}$$

where,

X = loss of reinforcement thickness or radius (μm)
K = constant (μm)
t = time (years)

Equation IV-1 assumes that attack from the surrounding environment is immediate and unaffected by the presence of grout around the metal. In reality, some measure of corrosion protection is afforded to the metal reinforcements by the surrounding grout. Therefore the following calculations are considered conservative estimates of remaining service life and loss of cross section.

The appropriate parameters for use in the rate equation are based on the corrosiveness index of the weathered rock samples collected from the Barron Mountain site. According to the recommendations described in NCHRP (2002), the parameters "K" and "r" for use in the rate equation are adjusted relative to soil conditions as summarized below.

Table IV-5. Recommended Parameters for Service Life Prediction Model (NCHRP, 2002)

Parameter	Average	Corrosive	Highly Corrosive
K (μm)	35	50	340
r	1.0	1.0	1.0

Based on the measurements of resistivity and pH, and the hydrogeologic conditions at the Barron Mountain site, the corrosiveness of the environment is described as average for the purpose of estimating service life. Therefore, values of K = 35 μm and r = 1 will be used to estimate the loss of cross section and service life of rock bolts and tendons installed at the site. However, the computed service-life will also consider the possibility of localized, pitting type corrosion due to the relatively low pH measured for the weathered rock samples.

For environments with a pH < 5, where high strength steels are used, the effect of pitting corrosion needs to be considered. This is considered by multiplying the loss of reinforcement thickness, X, computed with the rate equation by a factor of two for estimation of remaining service-life (this presumes that the loss of tensile strength is approximately two times the average loss of section due to the effects of localized corrosion (Elias, 1990)).

Calculations for 1" Rock Bolts

Two types of rock bolts were used at the Barron Mountain site including 1-in diameter, polyester resin grouted bolts supplied by Bethlehem Steel Co., and 1-in diameter, polyester resin grouted Dywidag bolts supplied by Inland-Ryerson Steel Co. The Bethlehem bolts are made from Grade 80 steel and the Dywidag bolts are Grade 150 steel. Working loads are 20 kips and 40 kips for the Bethlehem and Dywidag bolts, respectively. If one considers an allowable load equal to 60% of the yield stress for the Bethlehem bolts, and 60% of the guaranteed ultimate tensile for the Dywidag bolts, then the required radius for each is approximately 0.364 inches and 0.376 inches, respectively. Therefore, approximately 0.124 inches (3150μm) of steel thickness can be sacrificed to corrosion given an average radius of 0.5 inches for each bolt. From Equation IV-1, with K = 35, r = 1, and the factor of 2 for pitting corrosion:

$$2(K)t = 3150$$

Hence, t = 45 years.

Therefore, remaining service life = t - age = 45 – (2003-1972) = 14 years.

Considering the age of the reinforcements, Equation IV-1 indicates up to 0.043 inches of steel may have been consumed by corrosion. This corresponds to a loss of cross section of approximately 16%. This amount of loss of cross section is considered to be close to the sensitivity of NDT measurements.

Calculations for 1.25" Rock Tendons

The steel tendons are 1.25-in diameter, Grade 150 ksi (1030 MPa) steel, Dywidag thread bars, fully grouted in 3-in diameter drill holes. The reinforcement cross sections do not include sacrificial thickness, and, apparently, rely on the surrounding Portland cement grout to passivate and protect the steel from corrosion. According to this strategy, the service-life of these reinforcements corresponds to the integrity and the thickness of the Portland cement grout surrounding the reinforcements.

However, the possibility of chloride contamination of the Portland cement grout exists, which may depassivate the steel and initiate corrosion. The time for initiation of corrosion in the presence of chlorides depends on the concentration of chlorides at the grout/rock interface, and the diffusivity and thickness of the grout surrounding the tendon. If the bars are centrally located within the three-inch diameter drill holes, they are protected by approximately 0.875 inches of grout. This amount of cover should provide at least 50 years of protection (since installation), given the subsurface conditions at the Barron Mountain site.

However, during the condition assessment, MMCE observed that not all of the reinforcements are centrally located within the drill hole. Because the ability of the Portland cement grout to protect the steel tendons is uncertain, we will estimate the "time to failure" of the reinforcements assuming they are unprotected.

The rock tendons are not prestressed, but it is assumed that passive resistance corresponding to the allowable stress levels in the reinforcements may be generated in response to rock deformation. Based on strain measurements described by Haley & Aldrich (1976), some of the reinforcements may be loaded to the allowable stress level of 60% of the minimum specified tensile strength (F_{PU}). For the purpose of illustration, we estimate the time for unprotected reinforcements to corrode to failure as follows:

1. Compute the critical radius as described by Briaud et al. (1998), corresponding to a reduction in cross sectional area to a level where failure will occur by overload. This calculation assumes that stress levels beyond 60% of F_{PU} are allowed by NHDOT. As described by ASTM A-722, the minimum specified tensile strength of the tendon is 150 ksi (1035 MPa). Assuming the tendon is loaded to 60% of the UTS under a constant load, the magnitude of the constant load can be calculated as F = 0.6* (150 ksi) * ($\pi d^2/4$). Given the diameter of the bar, d = 1.25 in (32 mm), F= 0.6 * 150* (π*$1.25^2/4$) = 110 kips. Therefore:

$$\mathrm{UTS} = 150(\mathrm{ksi}) = \frac{110(\mathrm{kips})}{\pi r_{\mathrm{critcial}}^{2}} \rightarrow r_{\mathrm{critical}} = 0.483(\mathrm{in}) = 12.27(\mathrm{mm})$$

   The critical radius computed above represents a symmetrical loss of thickness of the reinforcement equal to 32 mm/2 – 12.27 mm = 3.73 mm = 3730μm.

2. From Equation IV-1, with K = 35, r = 1 and the factor of 2 for pitting corrosion:

$$2(K)t = 3730$$

   Hence, t = 53 years.

Therefore, the estimated "time to failure" of unprotected tendons = t - age = 53 – (2003-1972) = 22 years.

Considering the age of the reinforcements, the service life Equation IV-1 indicates up to 0.043 inches of steel may have been consumed by corrosion. This corresponds to a loss of cross section for the tendons of approximately 13%.

APPENDIX V

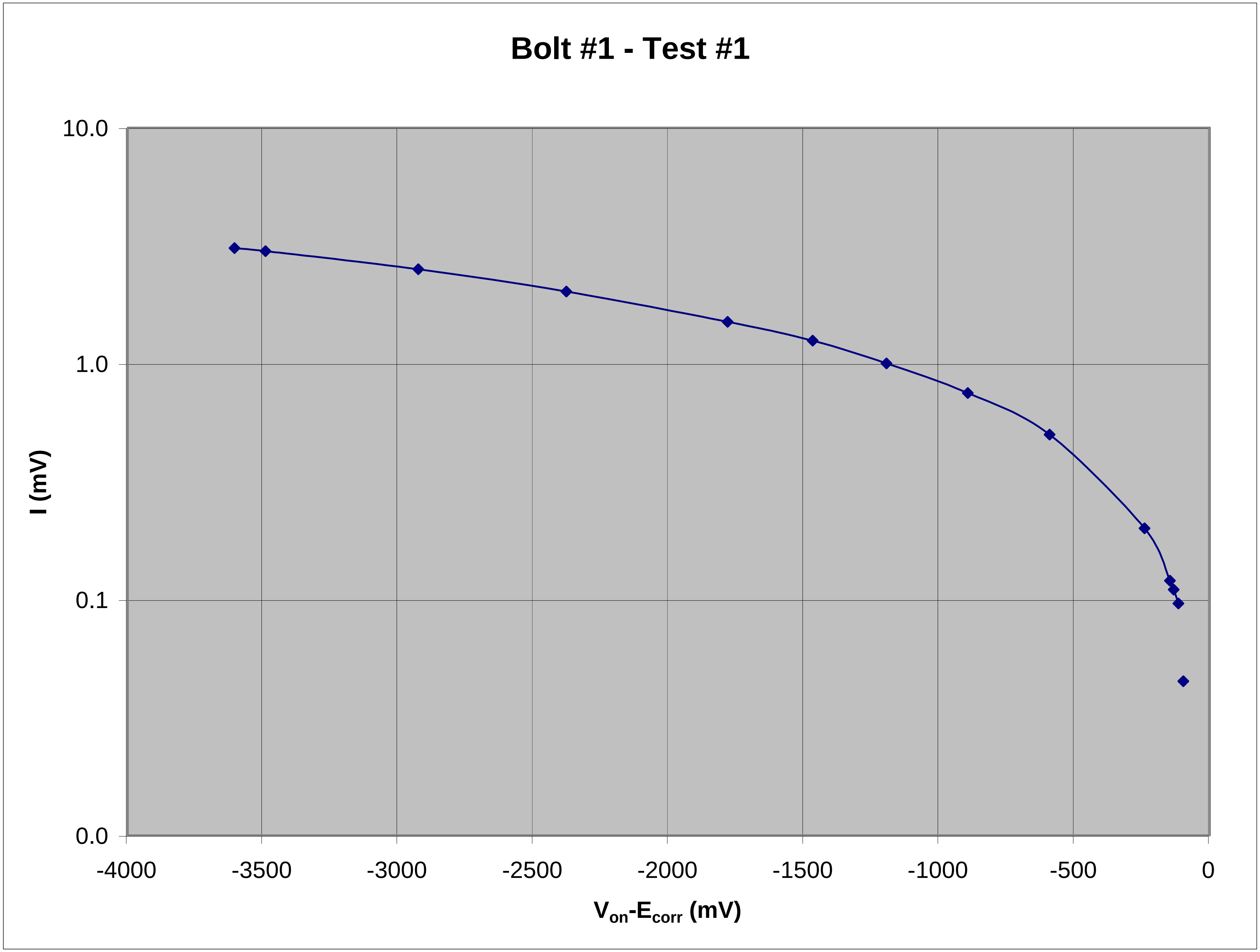
POLARIZATION CURVES

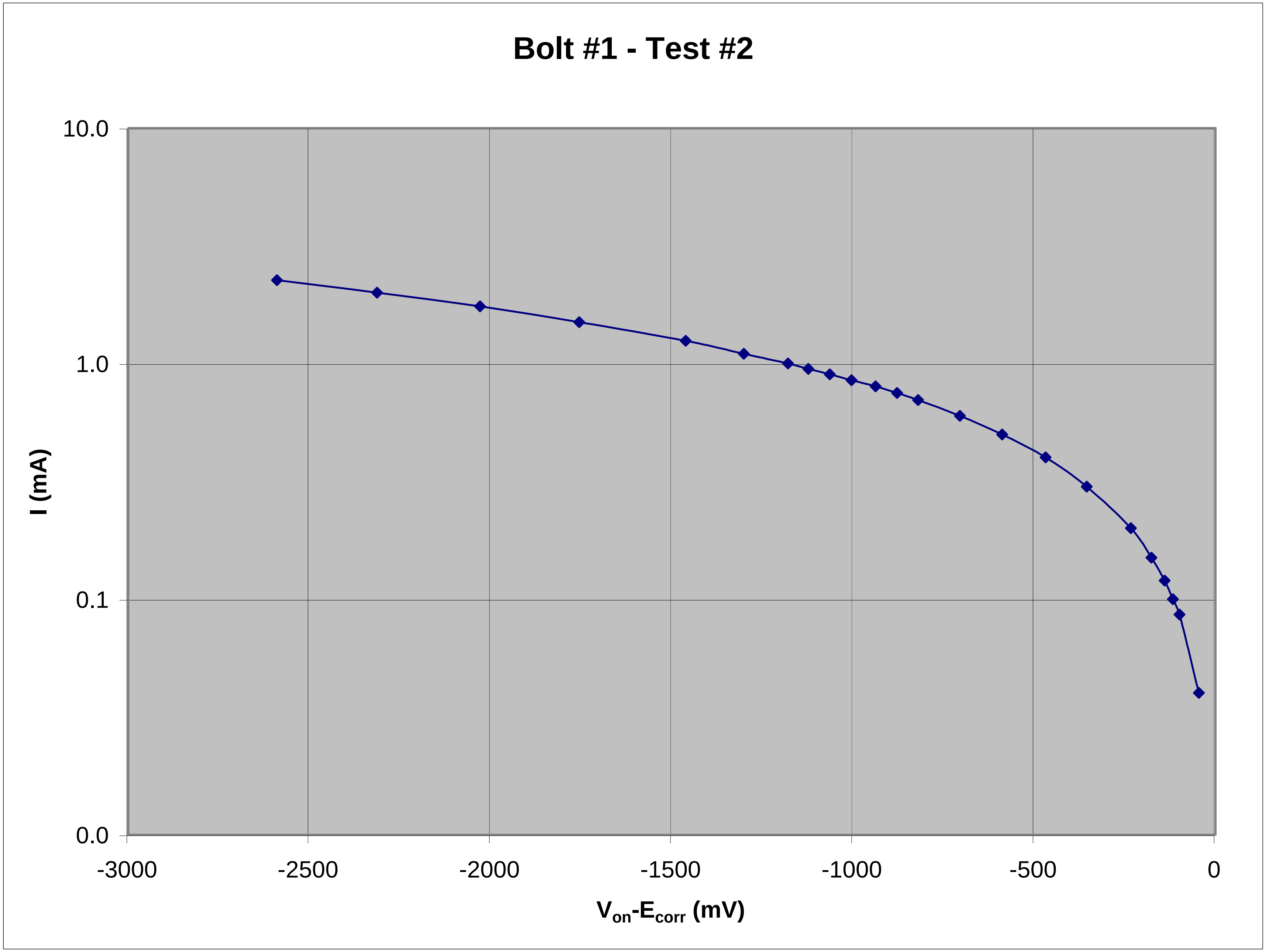
Table V-1. Summary of Polarization Current.

Rock Bolt #	I_P (mA)	Length (ft)	I_p/L (mA/ft)
1	**1.50**	**15**	**0.10**
2	1.50	25	0.06
3	0.85	25	0.03
4	1.75	30	0.06
5	1.50	25	0.06
6	**1.00**	**15**	**0.07**
7	**1.25**	**10**	**0.13**
8	**0.70**	**10**	**0.07**
9	0.50	10	0.05
10	**1.25**	**15**	**0.08**
11	**2.25**	**25**	**0.09**
12	**2.00**	**25**	**0.08**
13	1.75	30	0.06
14	1.75	30	0.06
15	**1.50**	**20**	**0.08**
16	**1.75**	**20**	**0.09**
17	**1.75**	**25**	**0.07**
18	1.25	25	0.05
19	**2.00**	**20**	**0.10**
20	**2.00**	**20**	**0.10**
21	**0.75**	**10**	**0.08**
22	1.50	25	0.06

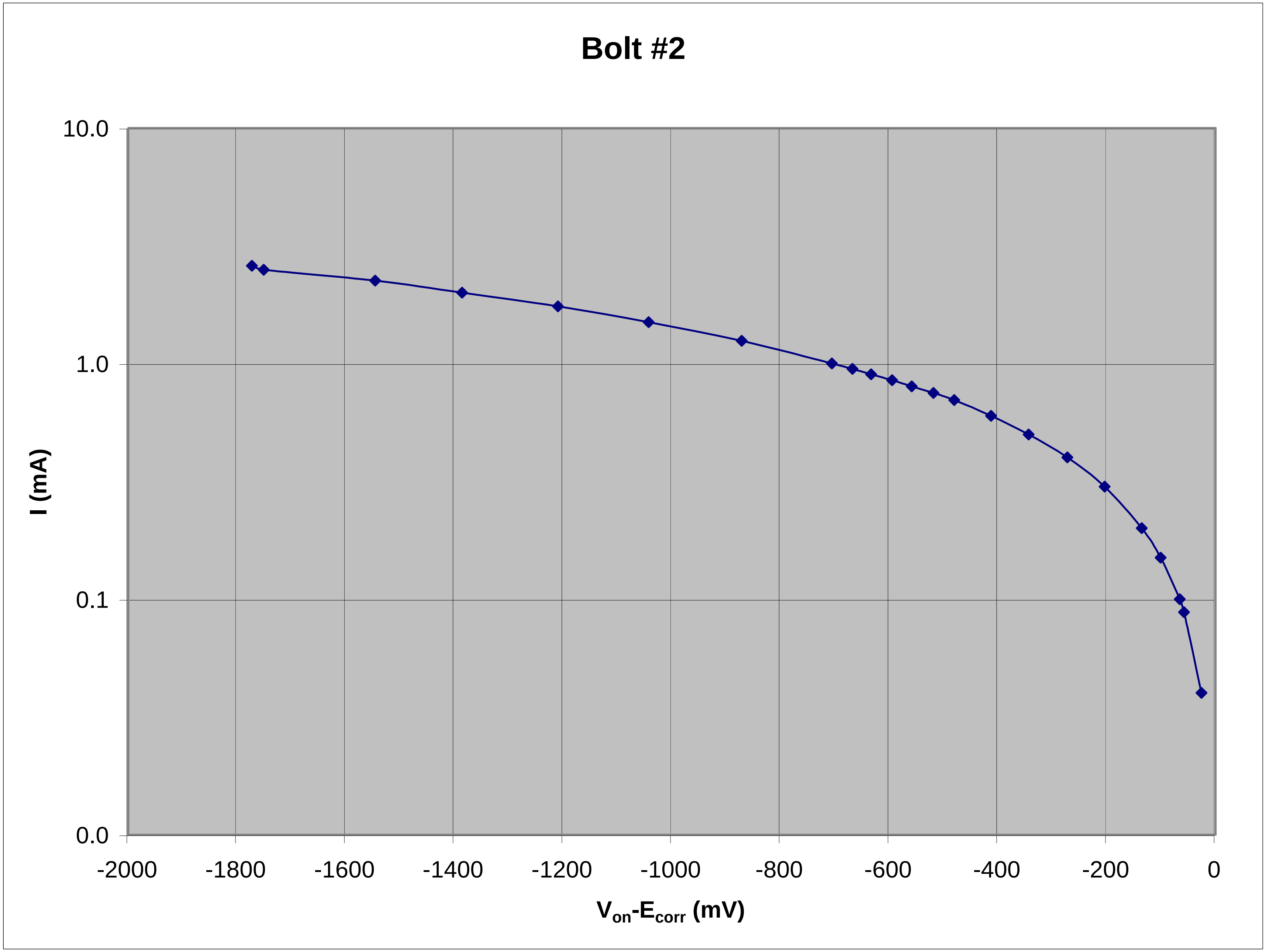
Table V-1. Summary of Polarization Current (cont.)

Tendon #	I_P (mA)	Length (ft)	I_p/L (mA/ft)
1-1	2.00	60	0.03
1-2	2.50	60	0.04
1-3	2.25	60	0.04
1-4	2.25	60	0.04
2-1	2.25	60	0.04
2-2	2.00	60	0.03
2-3	2.00	60	0.03
2-4	1.75	60	0.03
3-1	2.00	60	0.03
3-2	2.00	60	0.03
3-3	1.50	60	0.02
3-4	1.75	60	0.03
4-1	2.25	60	0.04
4-2	1.75	60	0.03
4-3	1.75	60	0.03
4-4	1.75	60	0.03
5-1	2.25	60	0.04
5-2	2.00	60	0.03
5-3	2.75	60	0.05
5-4	2.75	60	0.05

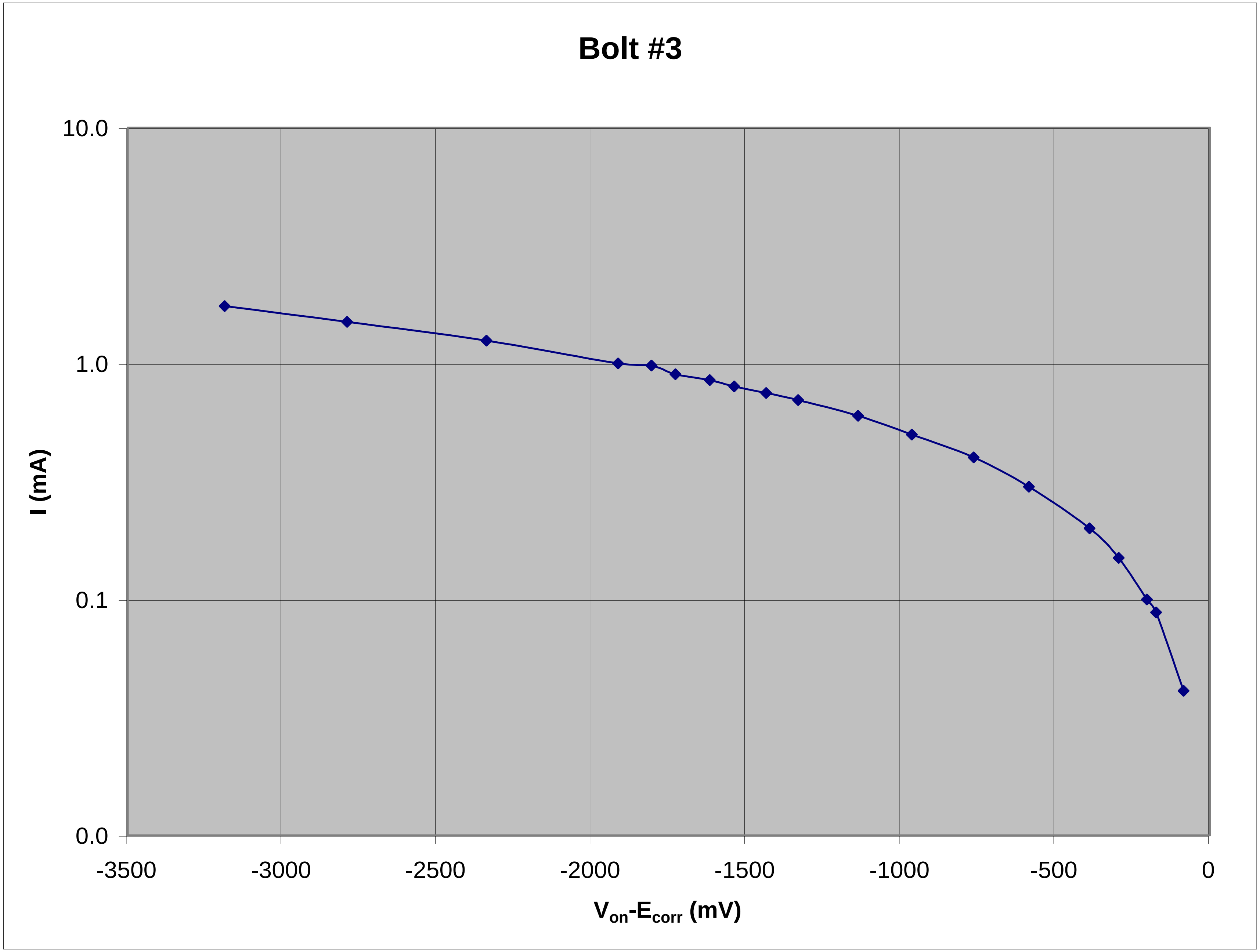

Bolt #1 - Test #1
10.0
1.0
0.1
0.0
I (mV)
-4000
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
V_{on}-E_{corr} (mV)



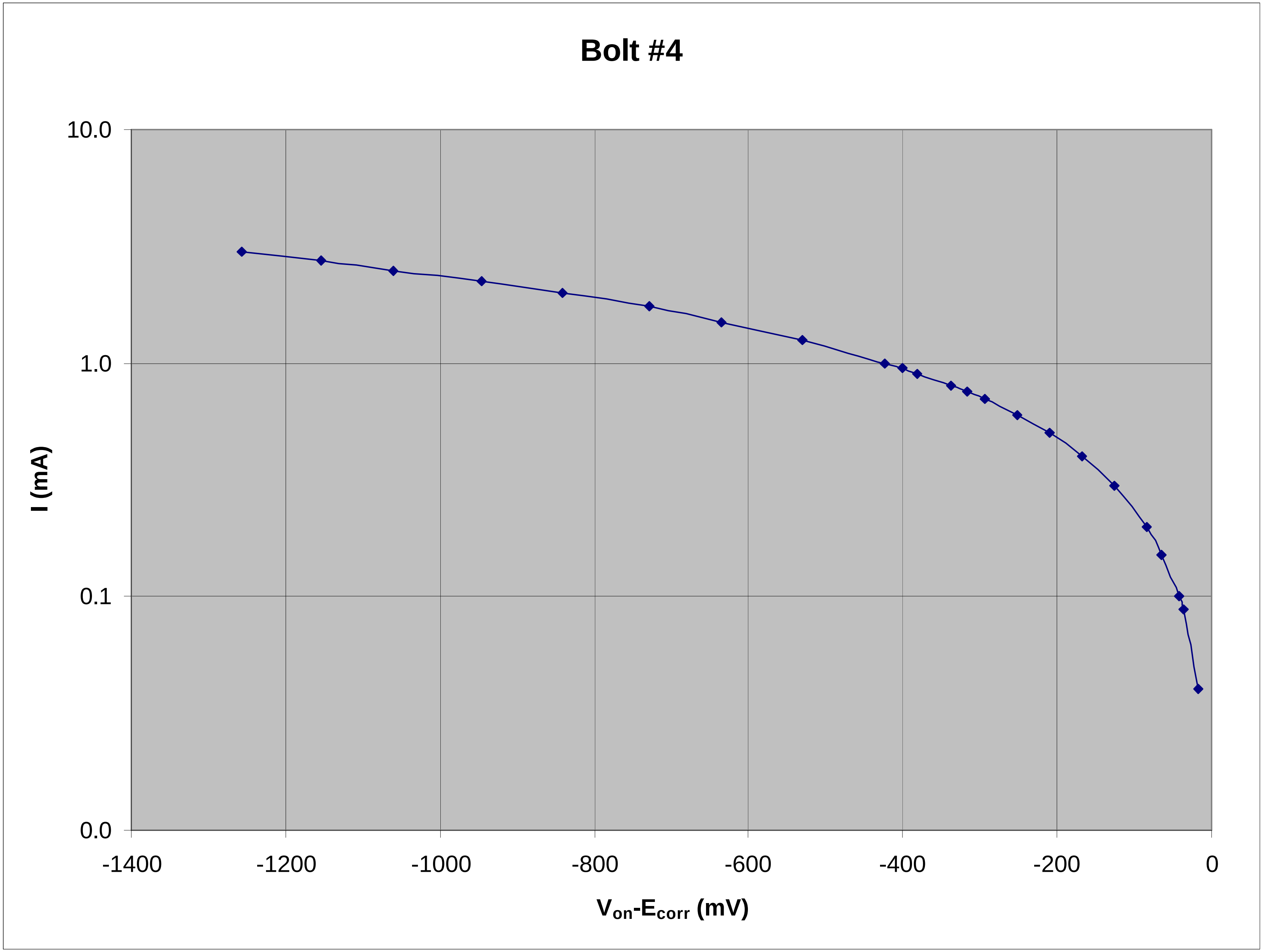
Bolt #1 - Test #2
10.0
1.0
0.1
0.0
I (mA)
-3000
-2500
-2000
-1500
-1000
-500
0
V_{on}-E_{corr} (mV)



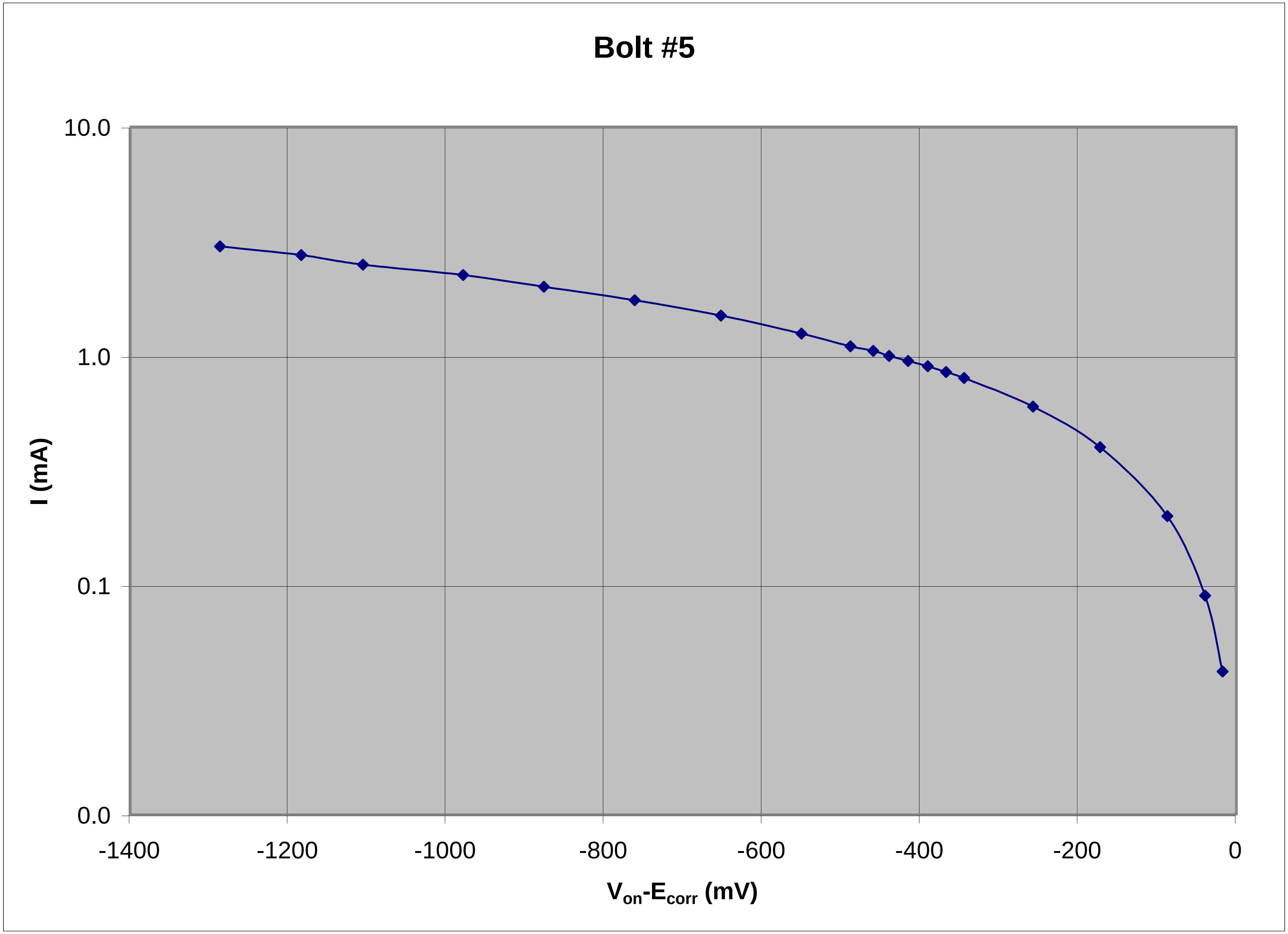
Bolt #2
10.0
1.0
0.1
0.0
I (mA)
-2000
-1800
-1600
-1400
-1200
-1000
-800
-600
-400
-200
0
V_{on}-E_{corr} (mV)



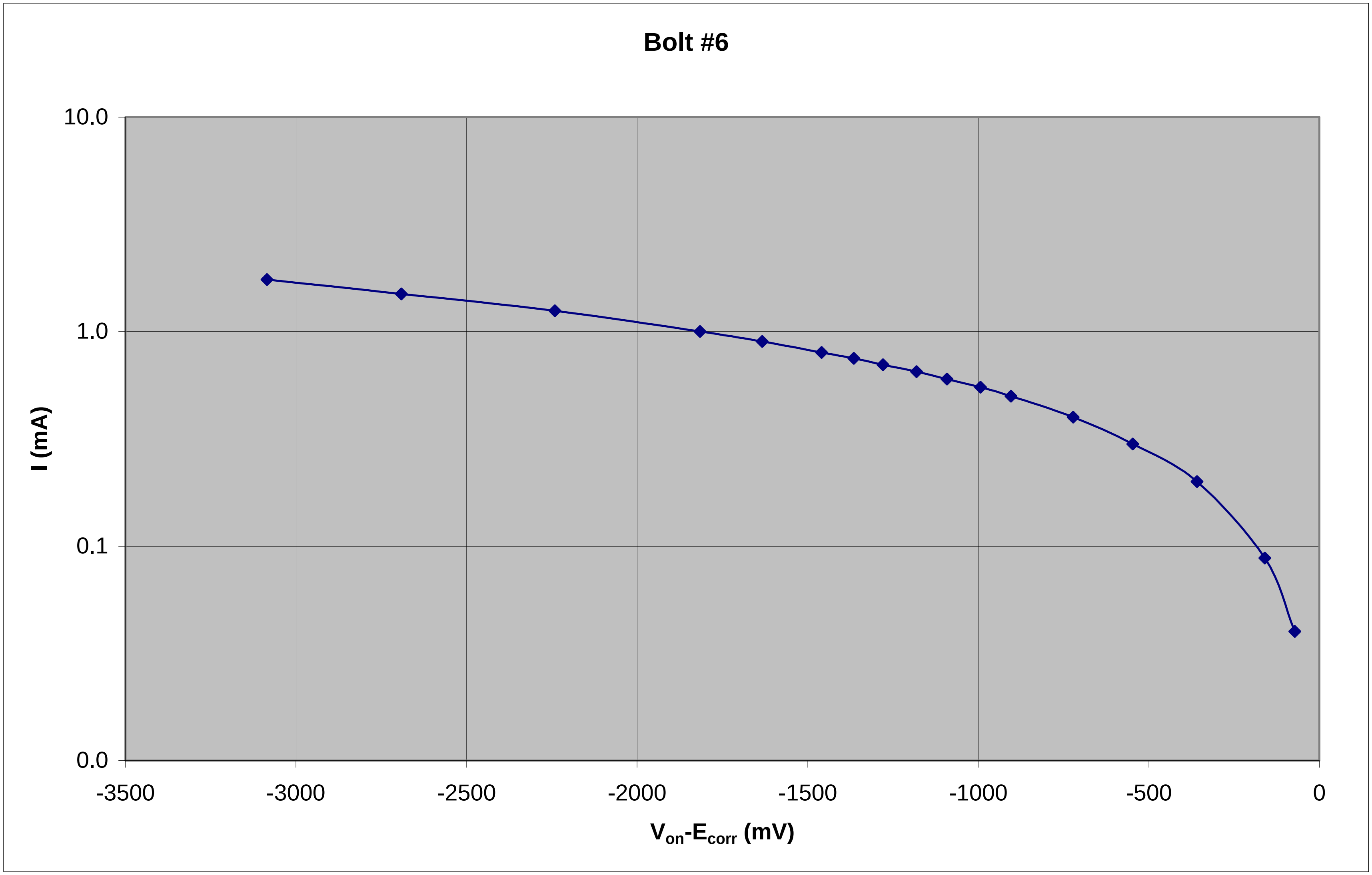
Bolt #3
10.0
1.0
0.1
0.0
I (mA)
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
V_{on}-E_{corr} (mV)



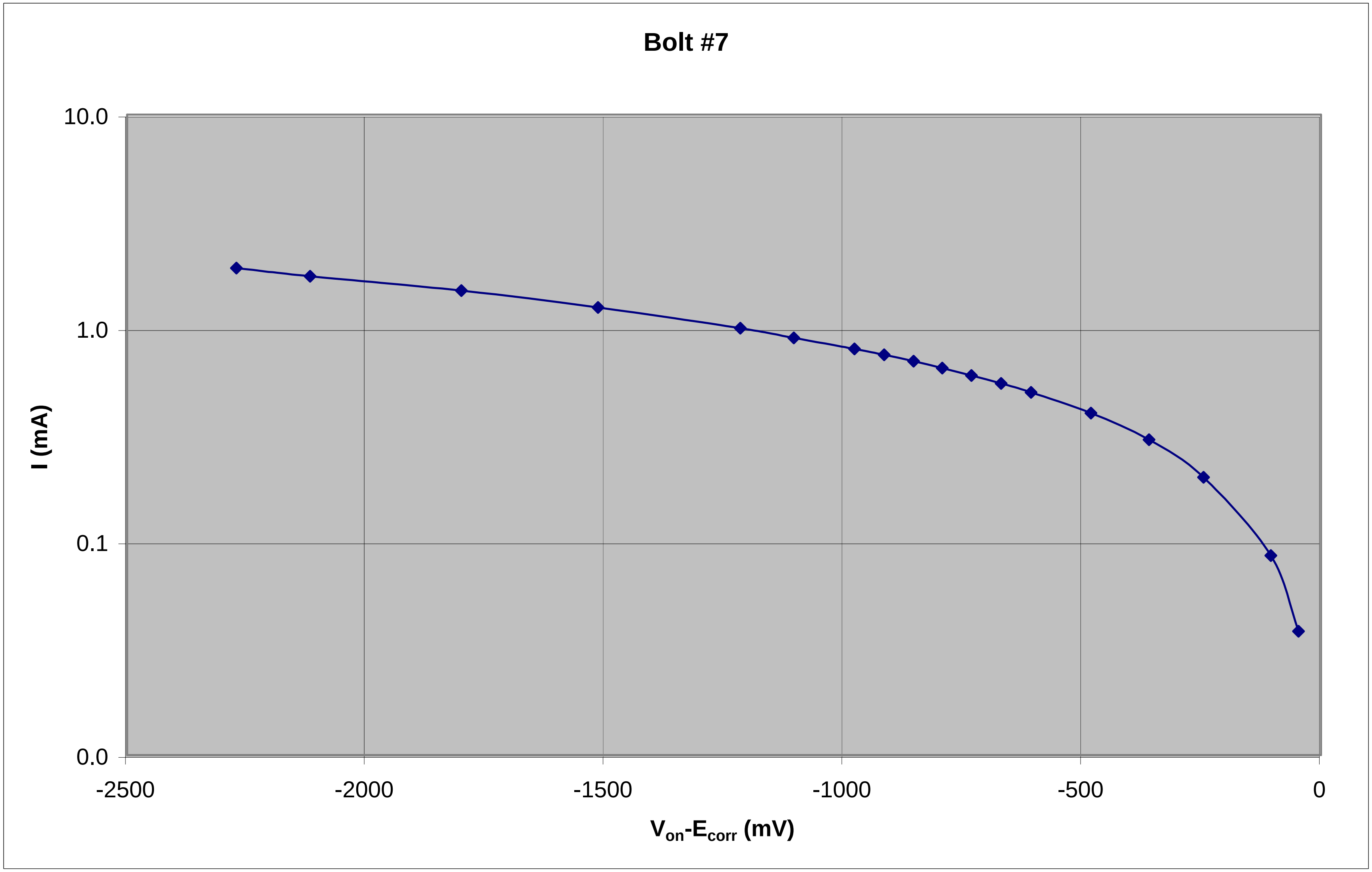
Bolt #4
10.0
1.0
0.1
0.0
I (mA)
-1400
-1200
-1000
-800
-600
-400
-200
0
V_{on}-E_{corr} (mV)



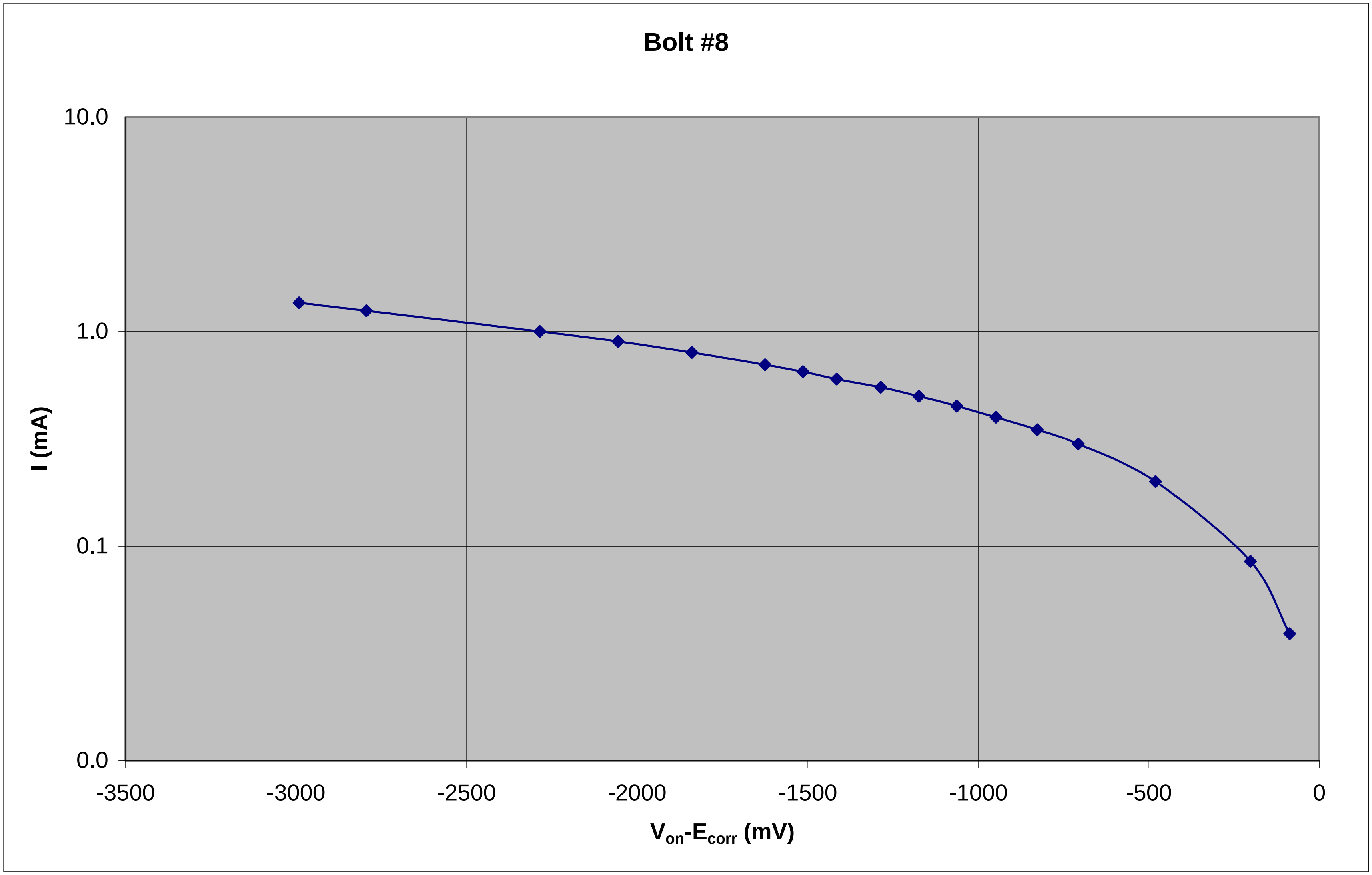
Bolt #5
10.0
1.0
0.1
0.0
I (mA)
-1400
-1200
-1000
-800
-600
-400
-200
0
V_on-E_corr (mV)

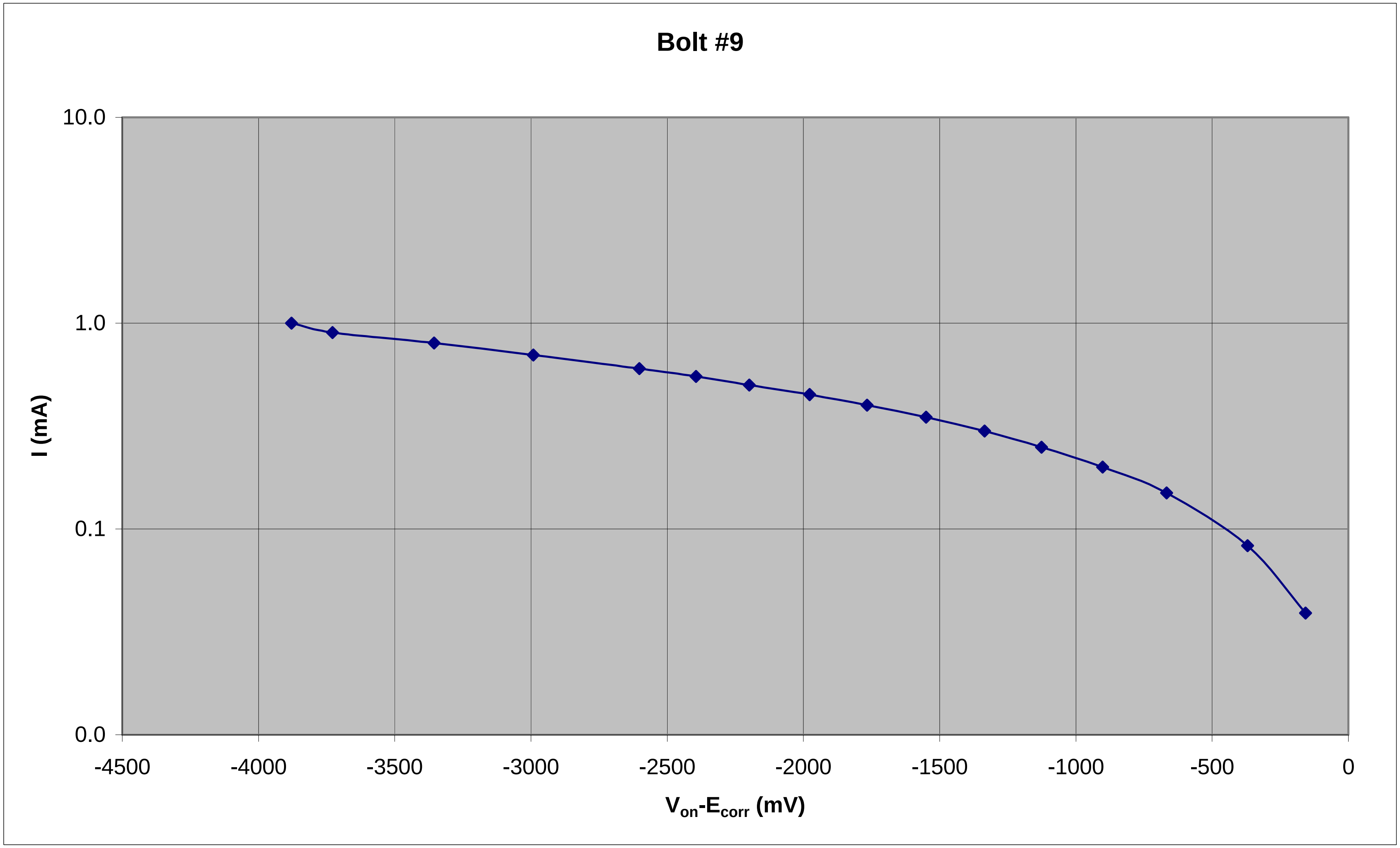


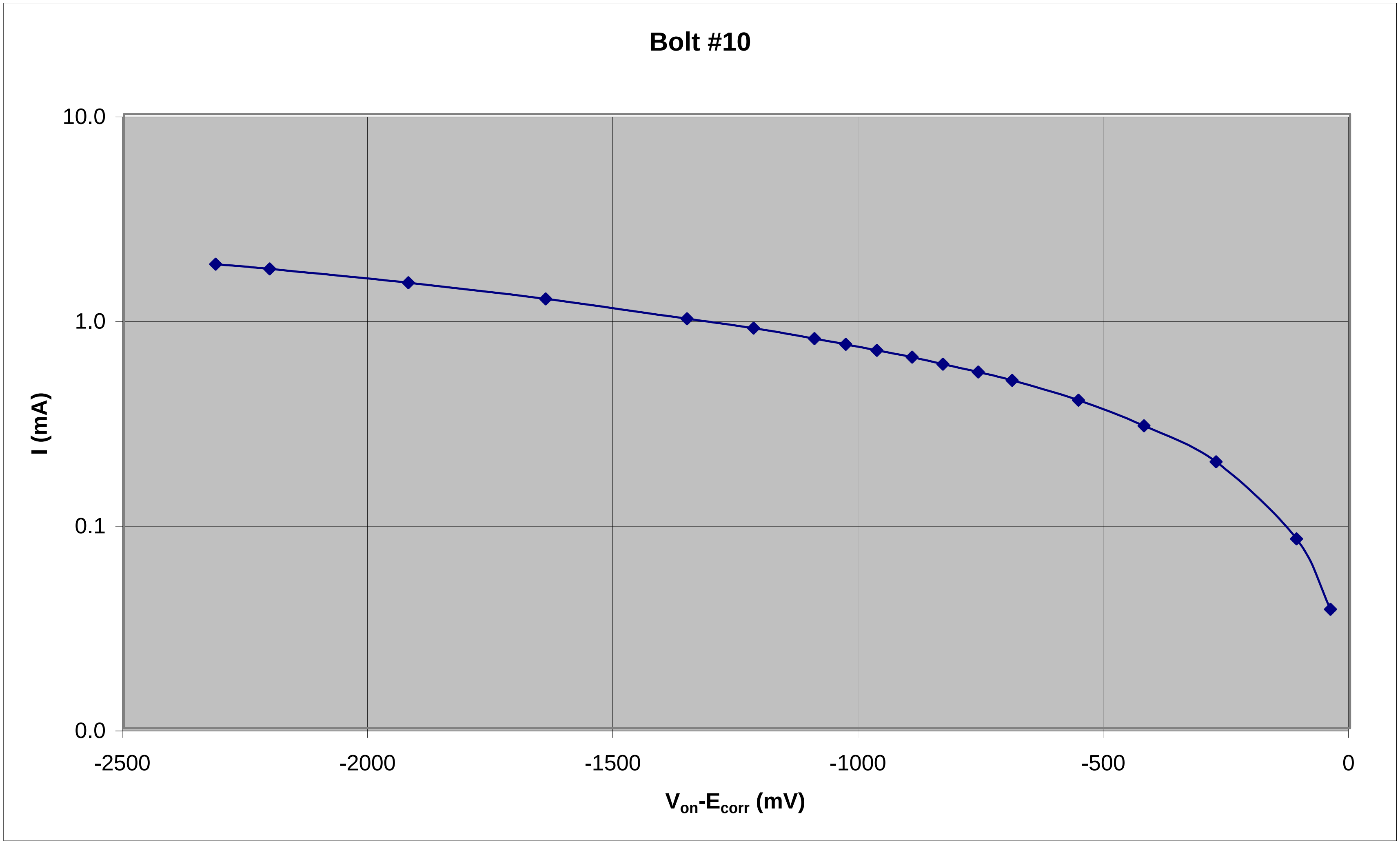
Bolt #6
10.0
1.0
0.1
0.0
I (mA)
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
V_{on}-E_{corr} (mV)



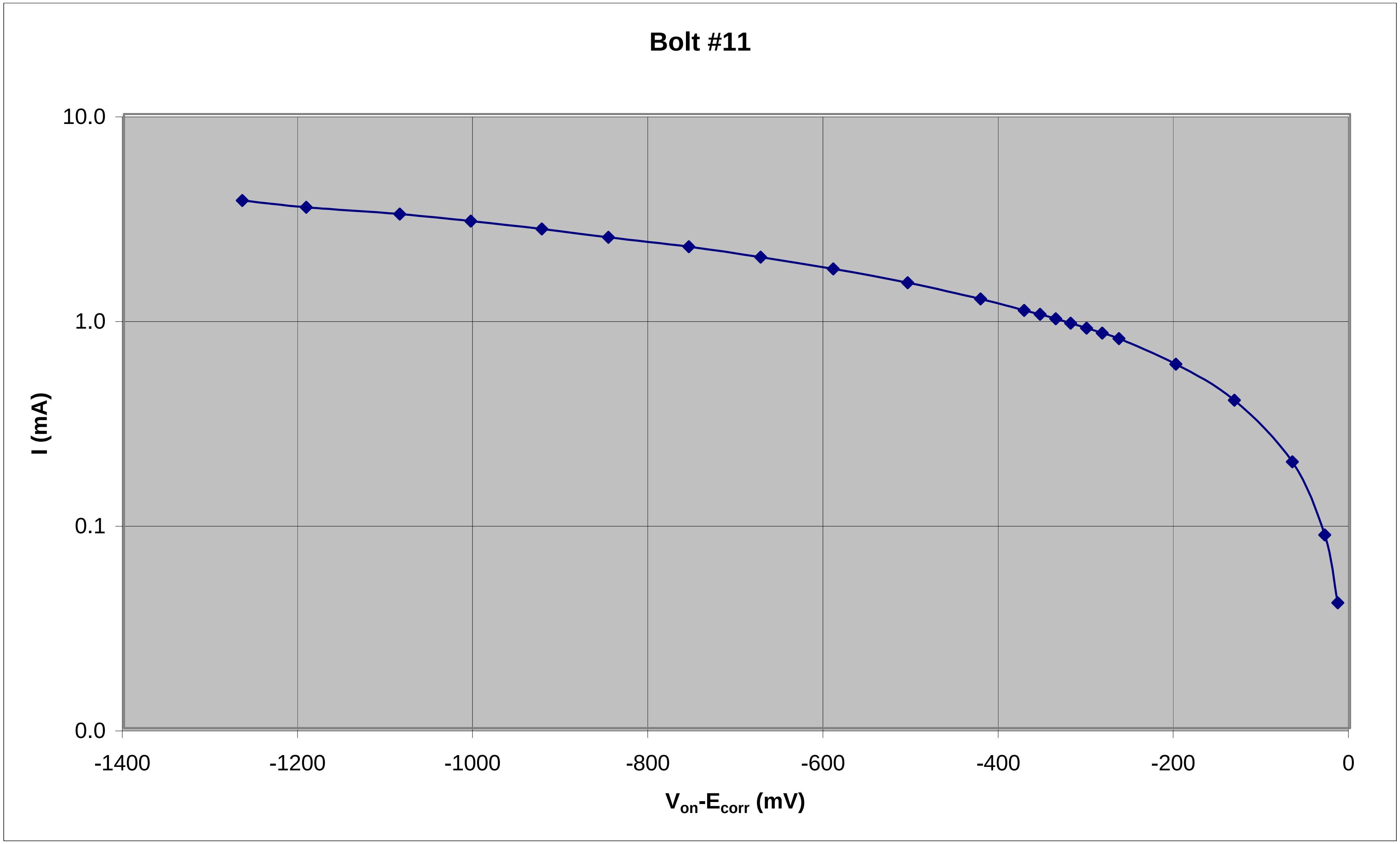
Bolt #7
10.0
1.0
0.1
0.0
I (mA)
-2500
-2000
-1500
-1000
-500
0
V_{on}-E_{corr} (mV)

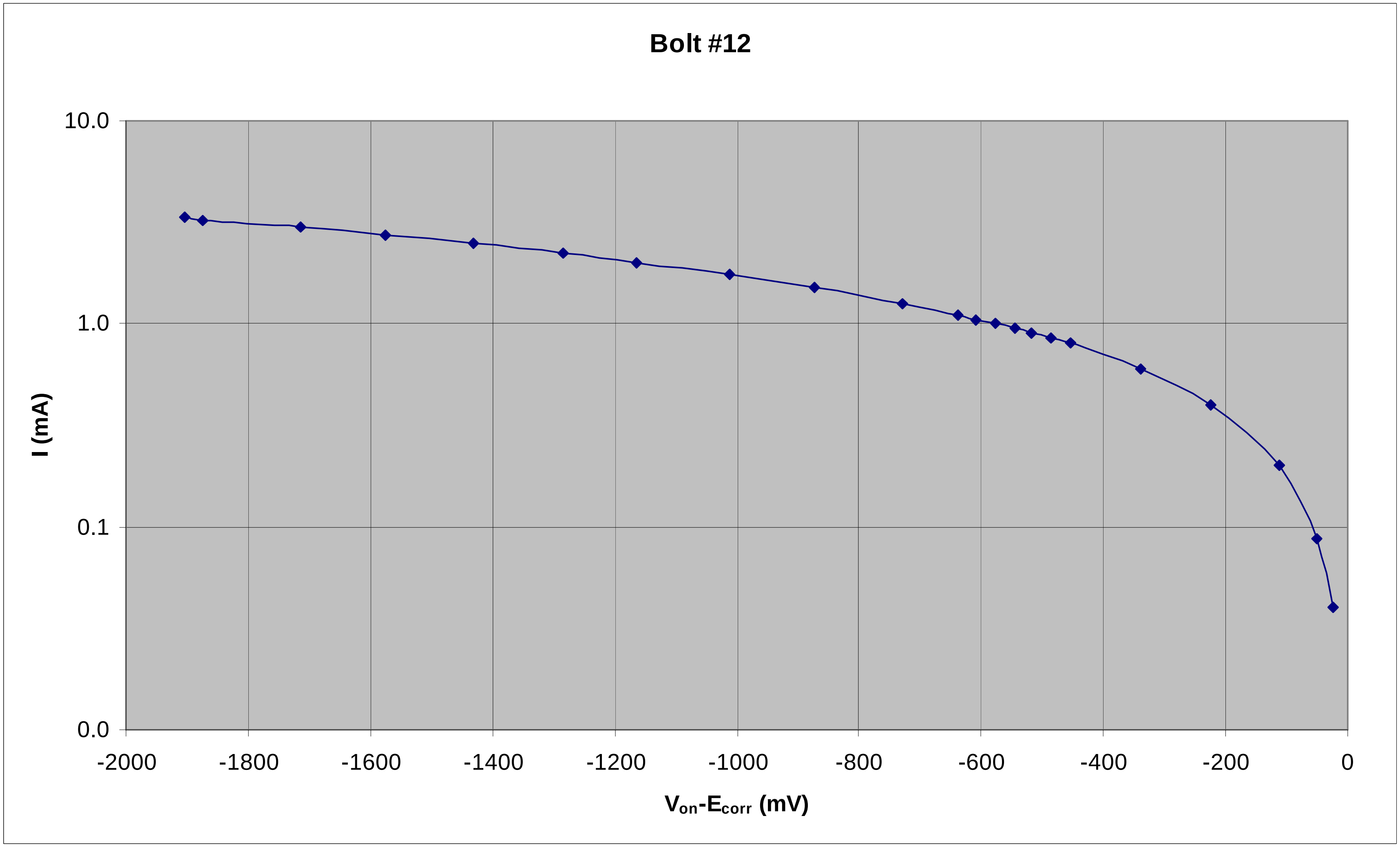

Bolt #8
10.0
1.0
0.1
0.0
I (mA)
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
Von-Ecorr (mV)


Bolt #9
10.0
1.0
0.1
0.0
I (mA)
-4500
-4000
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
V_on-E_corr (mV)

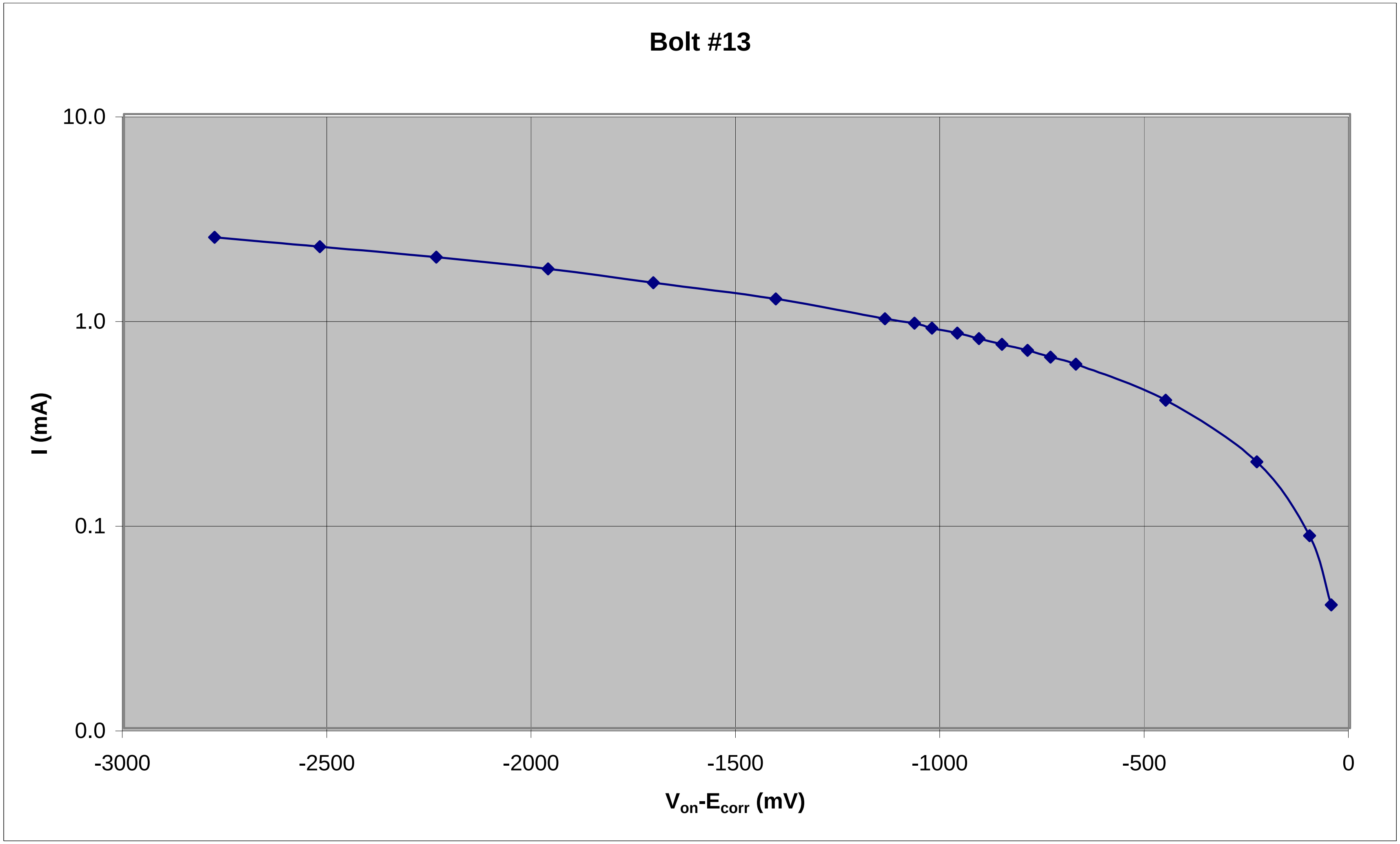


Bolt #10
10.0
1.0
0.1
0.0
I (mA)
-2500
-2000
-1500
-1000
-500
0
V_{on}-E_{corr} (mV)

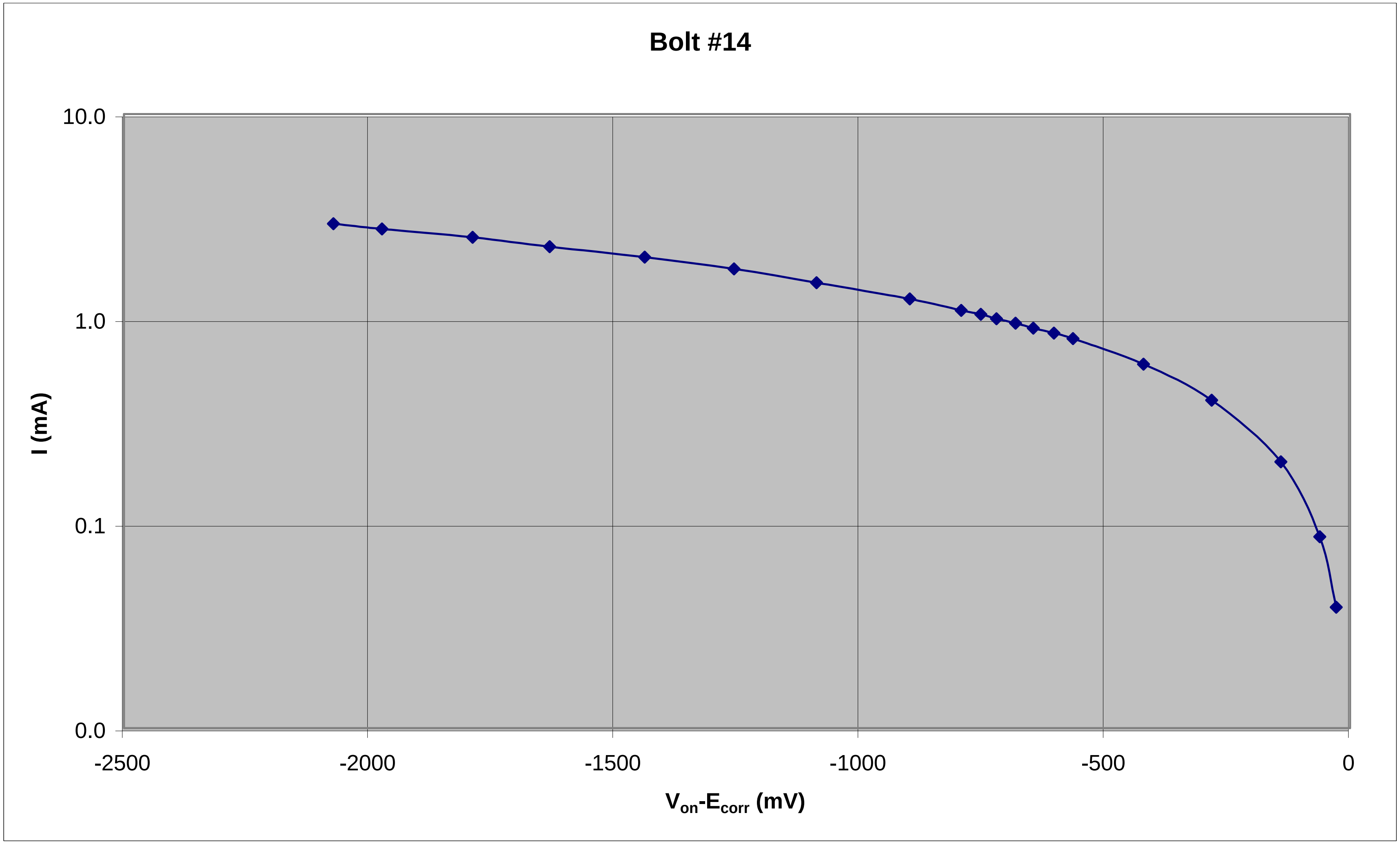

Bolt #11
10.0
1.0
0.1
0.0
I (mA)
-1400
-1200
-1000
-800
-600
-400
-200
0
V_{on}-E_{corr} (mV)

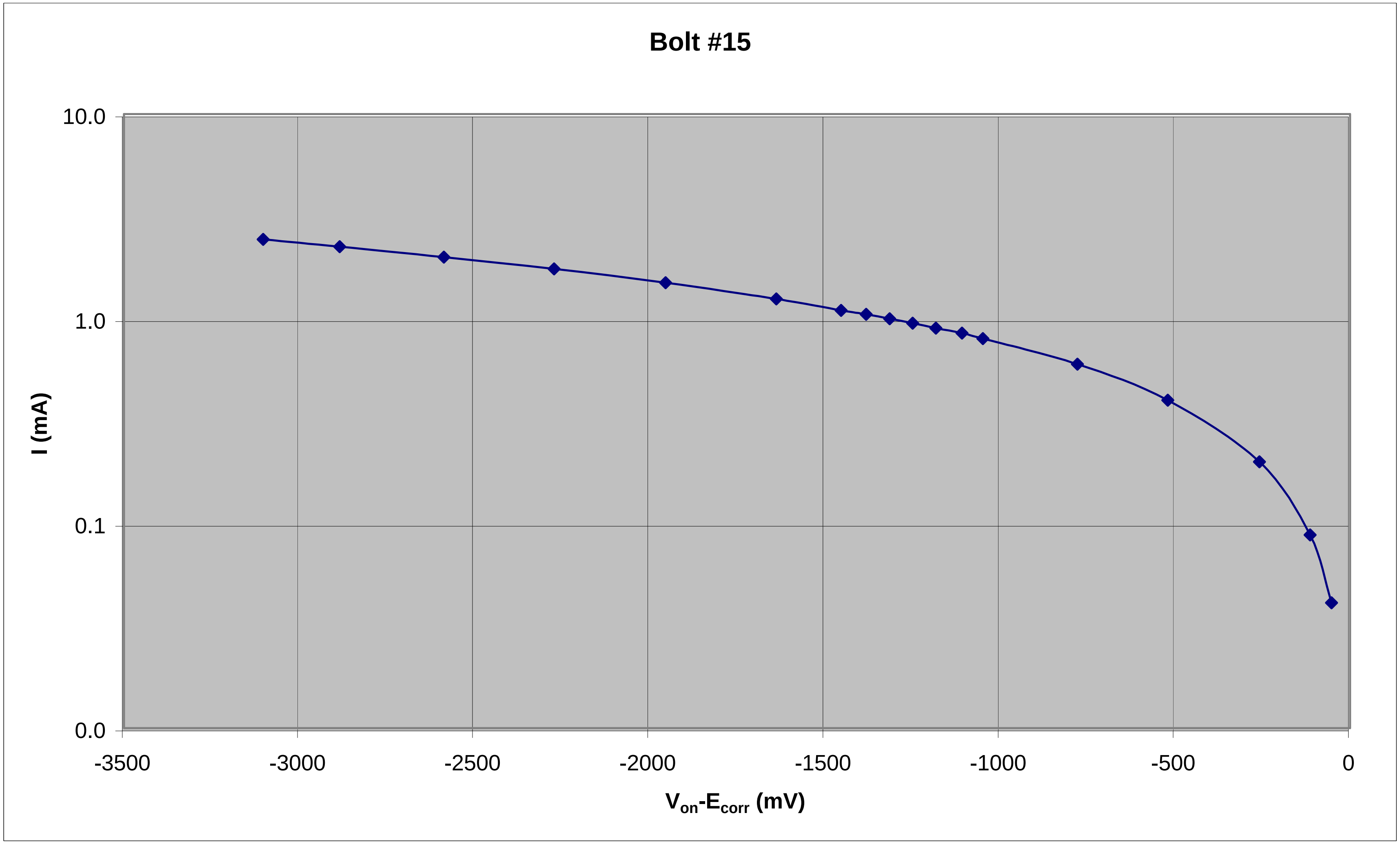


Bolt #12
10.0
1.0
0.1
0.0
I (mA)
-2000
-1800
-1600
-1400
-1200
-1000
-800
-600
-400
-200
0
Von-Ecorr (mV)

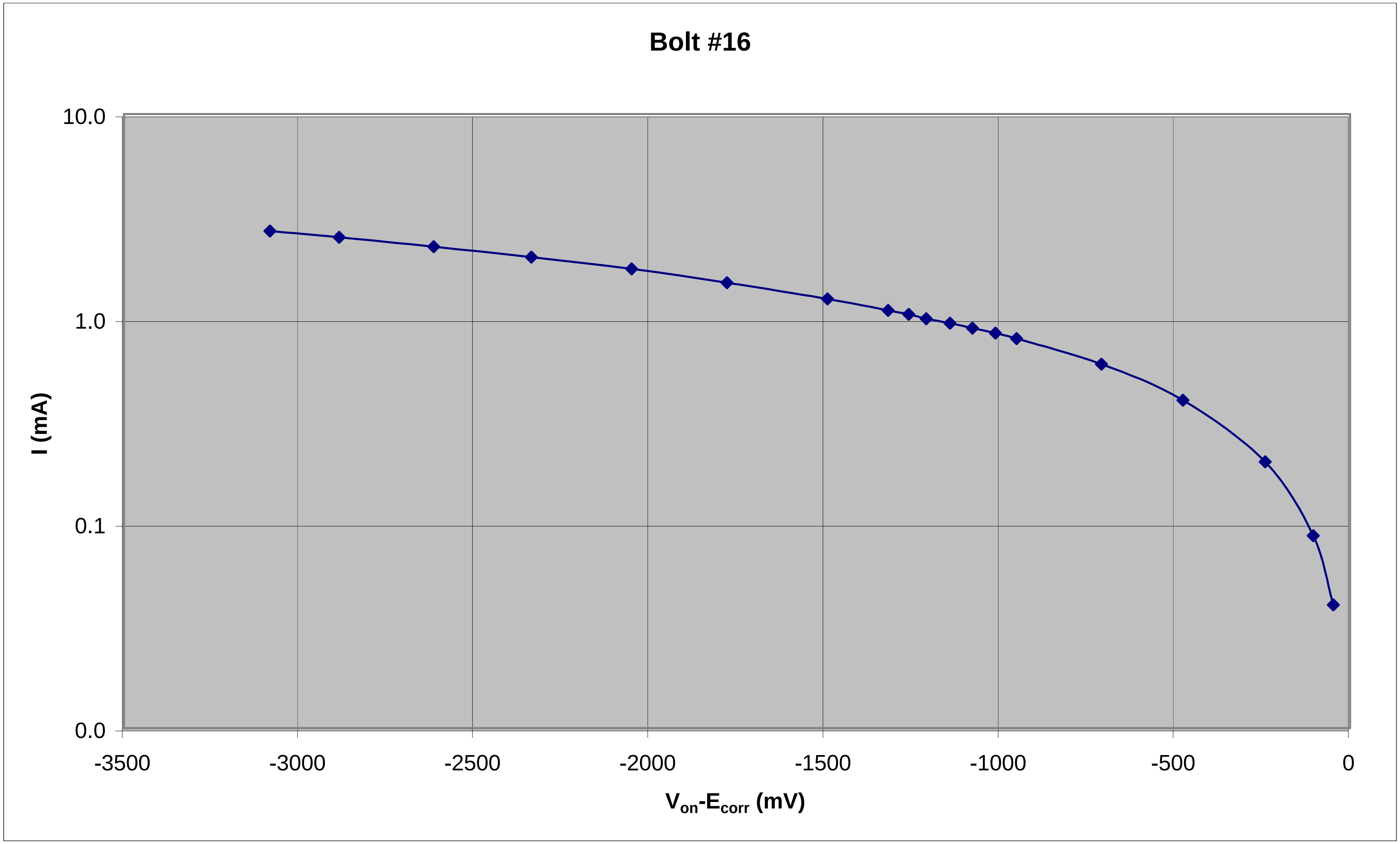


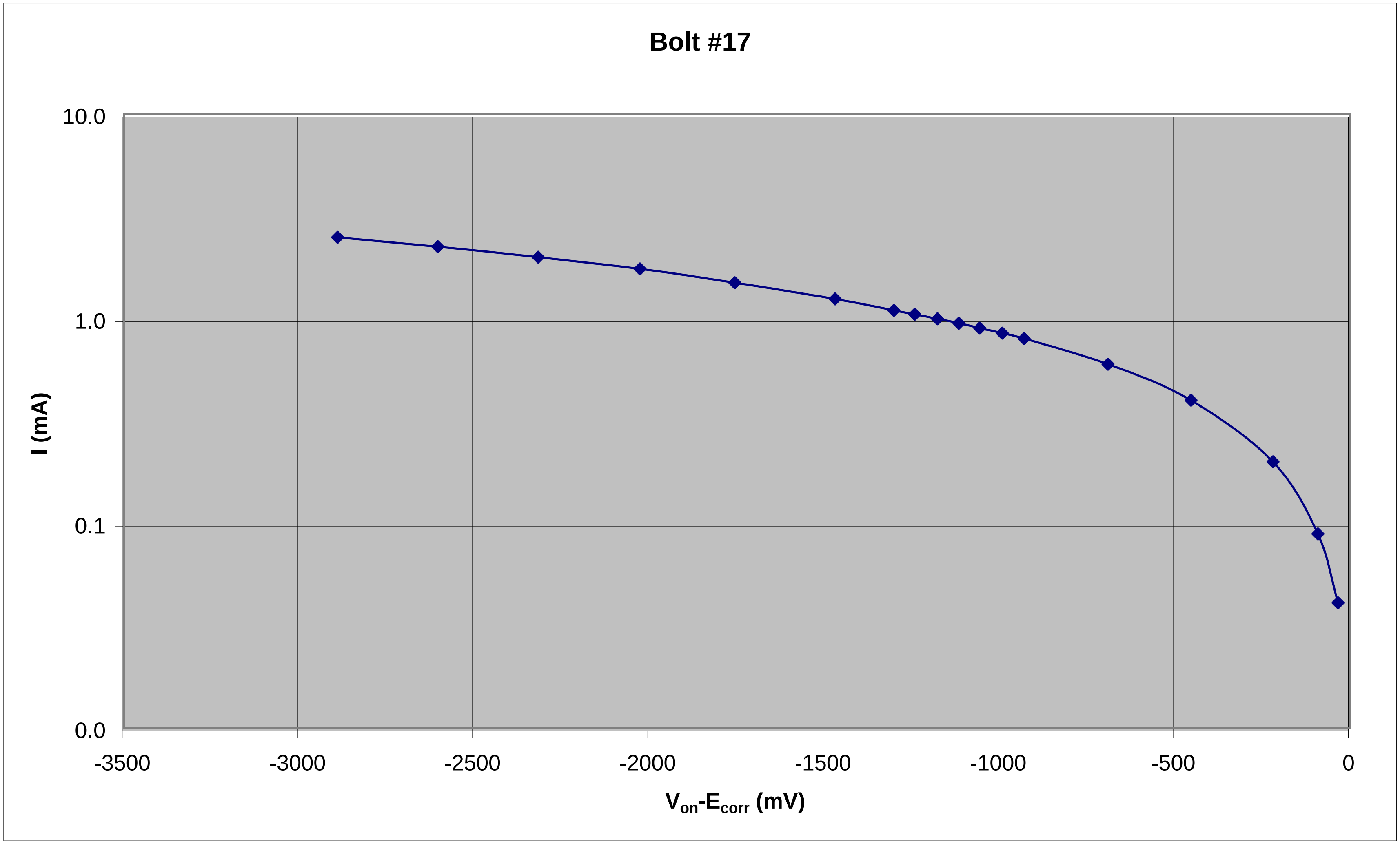
Bolt #13
10.0
1.0
0.1
0.0
I (mA)
-3000
-2500
-2000
-1500
-1000
-500
0
V_on-E_corr (mV)


Bolt #14
10.0
1.0
0.1
0.0
I (mA)
-2500
-2000
-1500
-1000
-500
0
V_{on}-E_{corr} (mV)

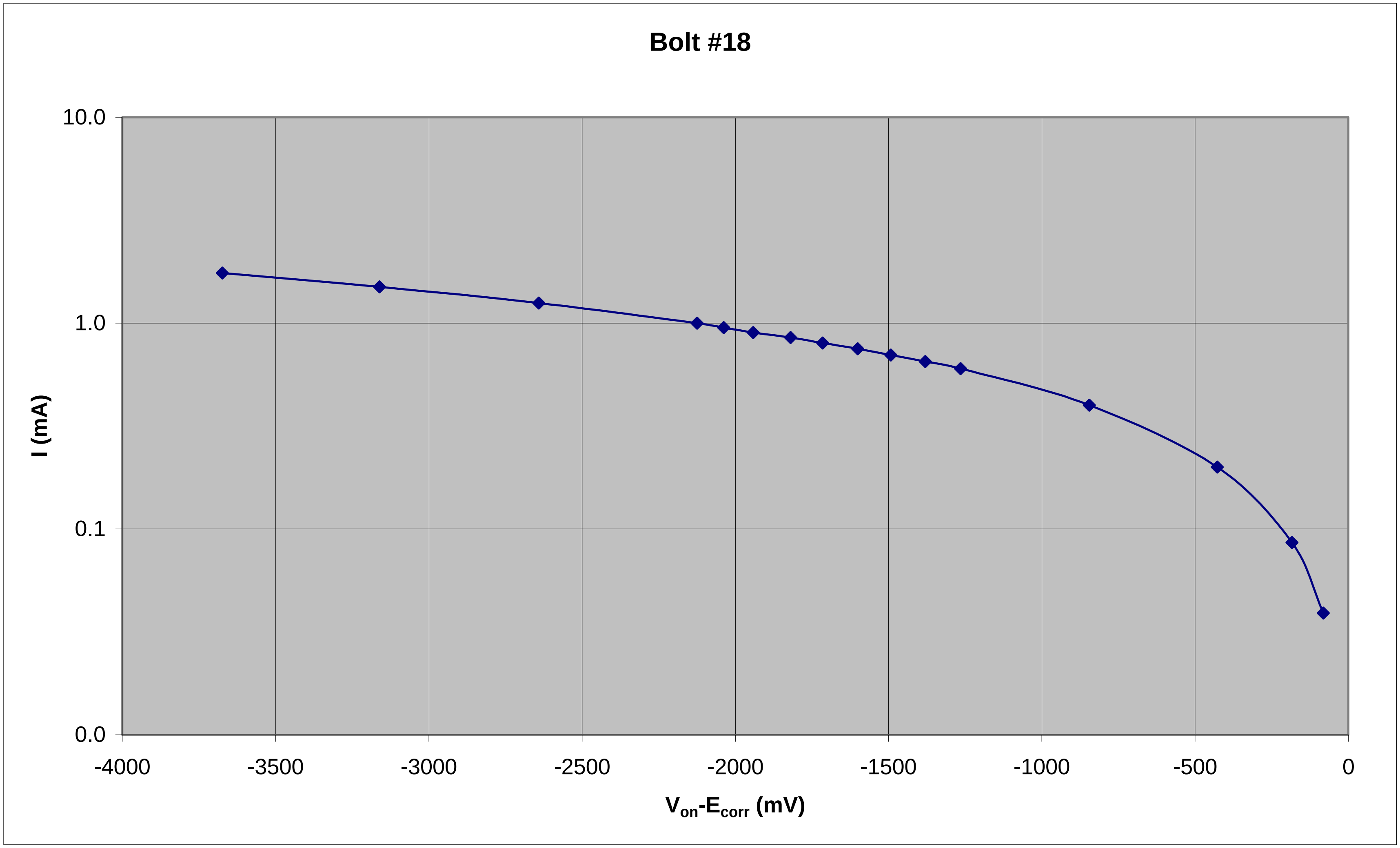


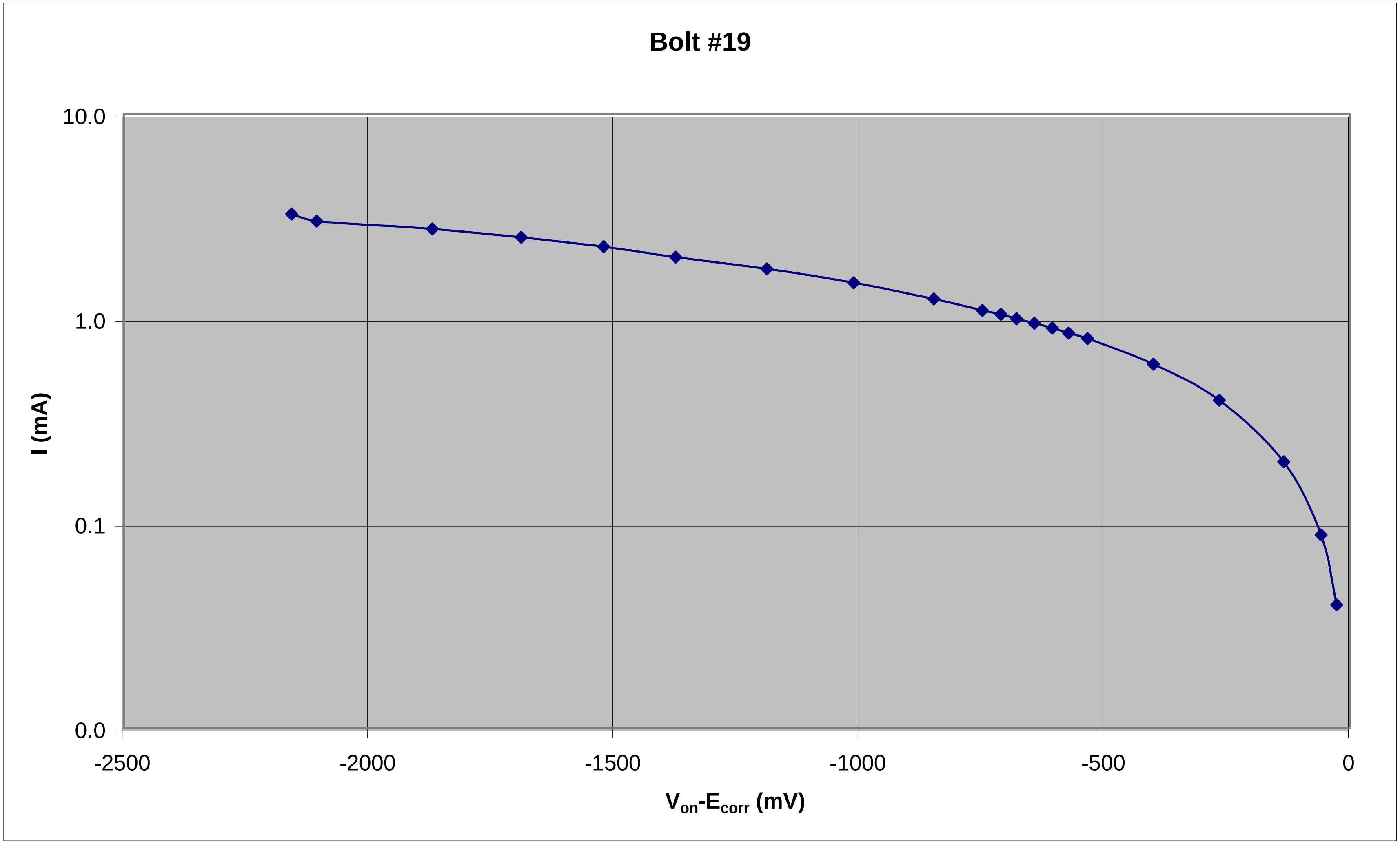
Bolt #15
10.0
1.0
0.1
0.0
I (mA)
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
V_{on}-E_{corr} (mV)


Bolt #16
10.0
1.0
0.1
0.0
I (mA)
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
Von-Ecorr (mV)

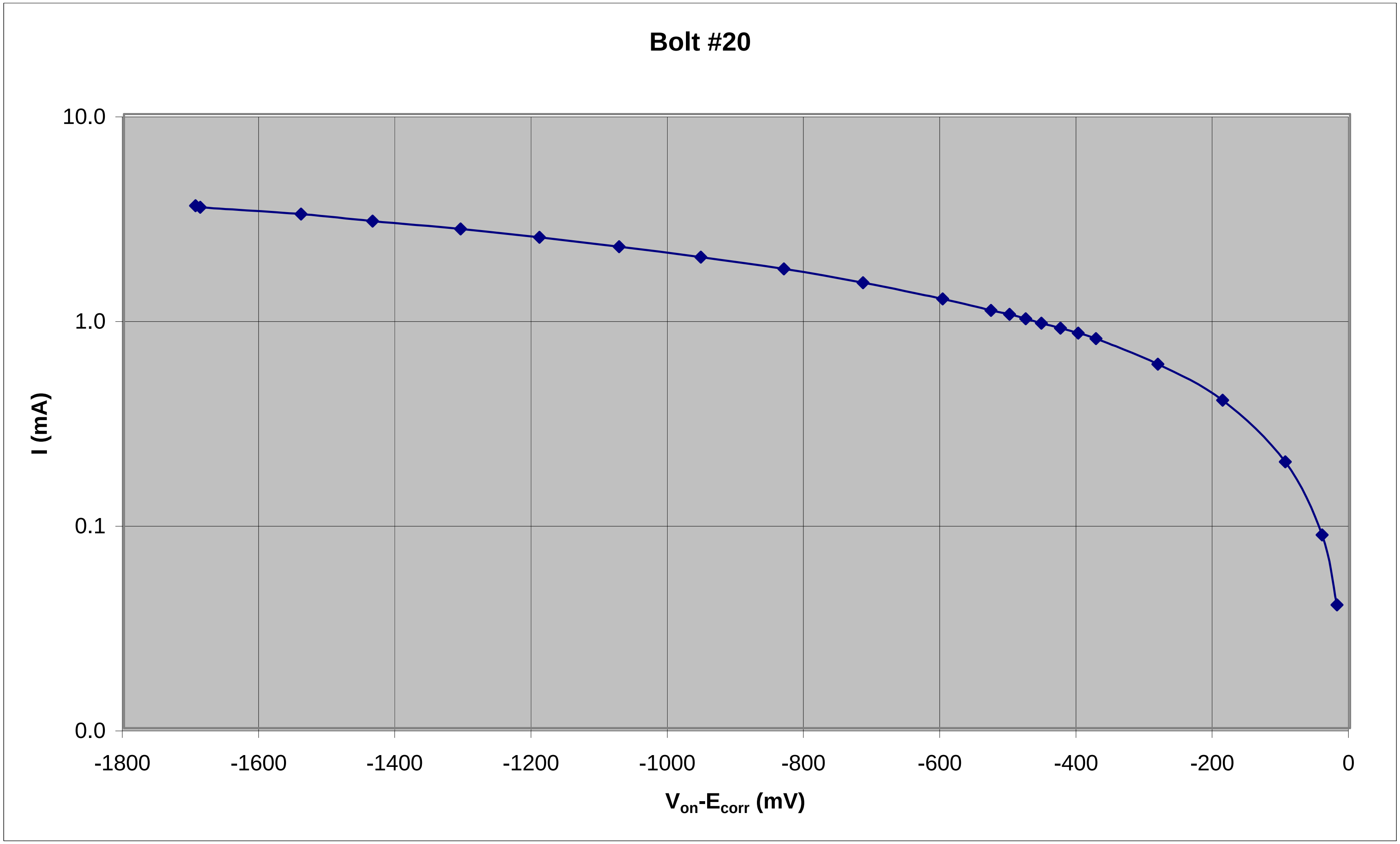


Bolt #17
10.0
1.0
0.1
0.0
I (mA)
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
V_{on}-E_{corr} (mV)

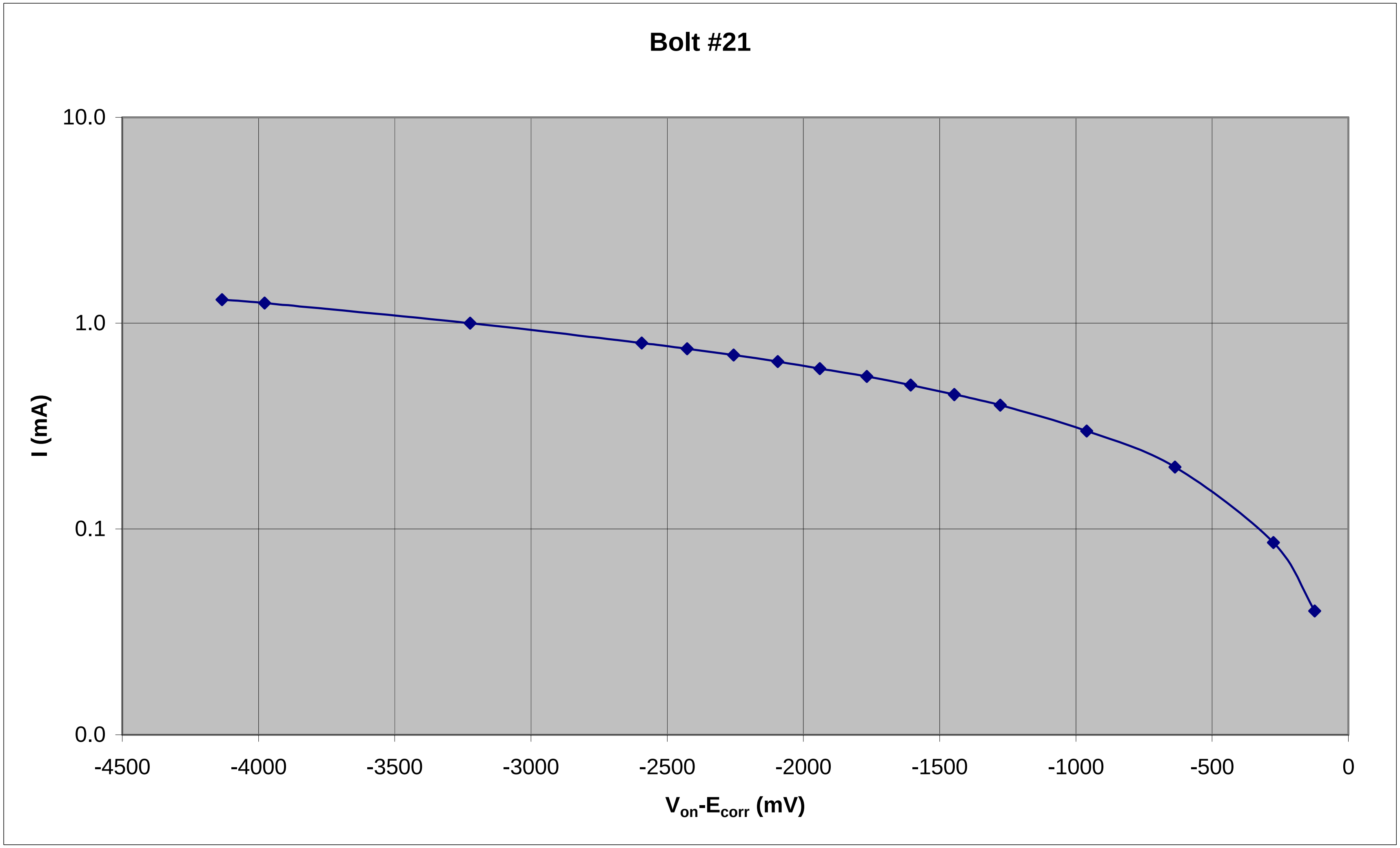

Bolt #18
10.0
1.0
0.1
0.0
I (mA)
-4000
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
V_{on}-E_{corr} (mV)



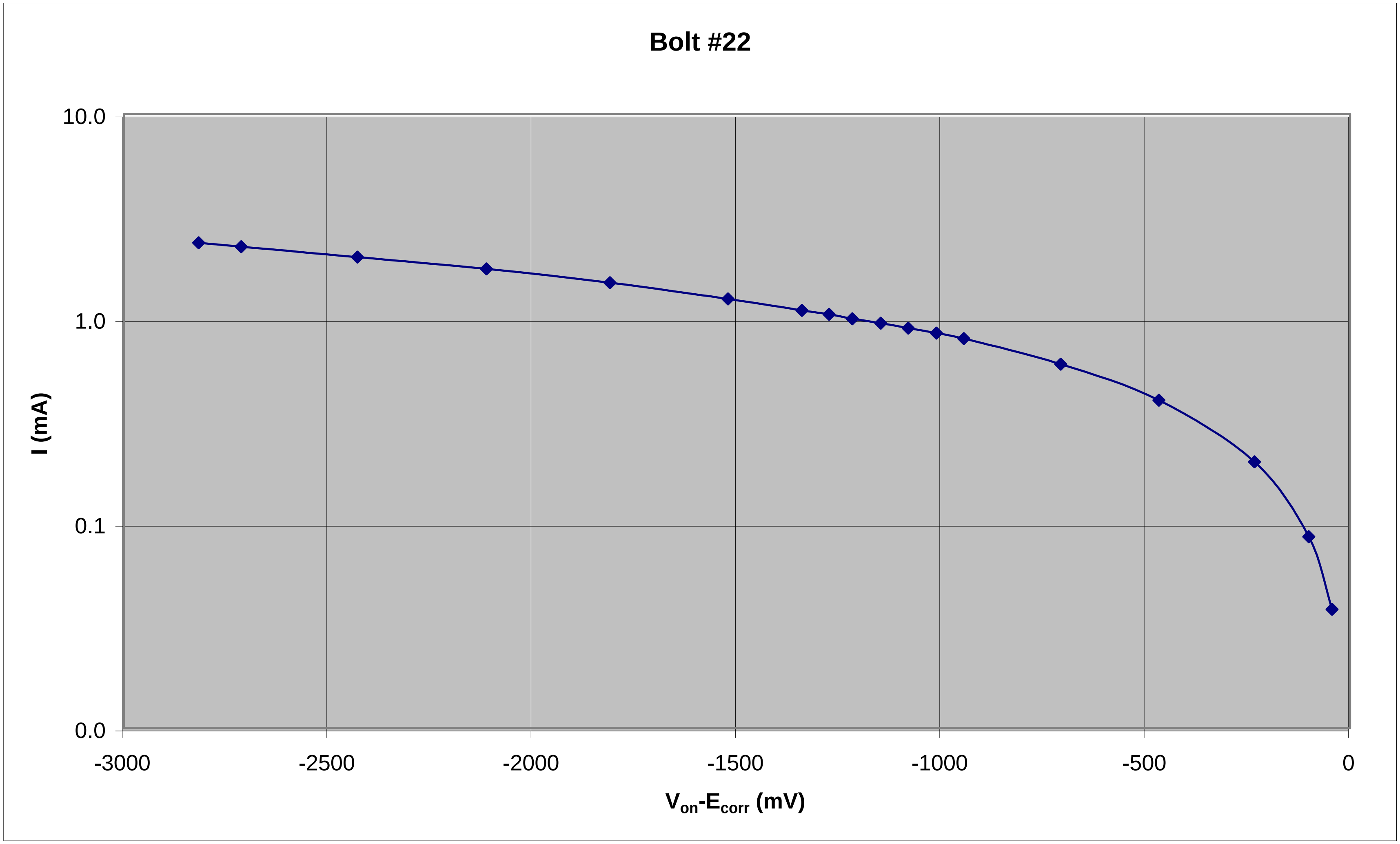
Bolt #19
10.0
1.0
0.1
0.0
I (mA)
-2500
-2000
-1500
-1000
-500
0
V_{on}-E_{corr} (mV)



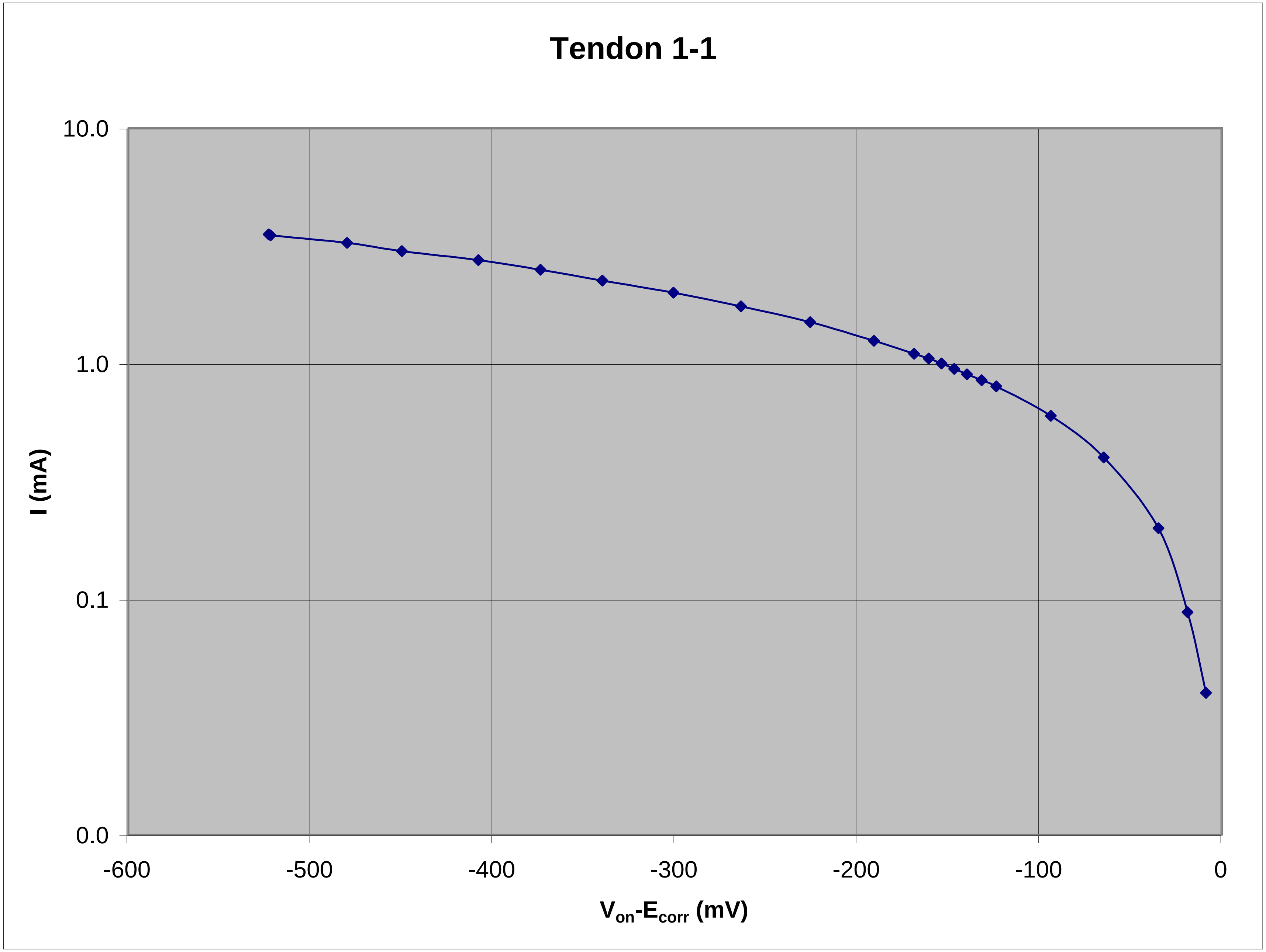
Bolt #20
10.0
1.0
0.1
0.0
I (mA)
-1800
-1600
-1400
-1200
-1000
-800
-600
-400
-200
0
V_{on}-E_{corr} (mV)



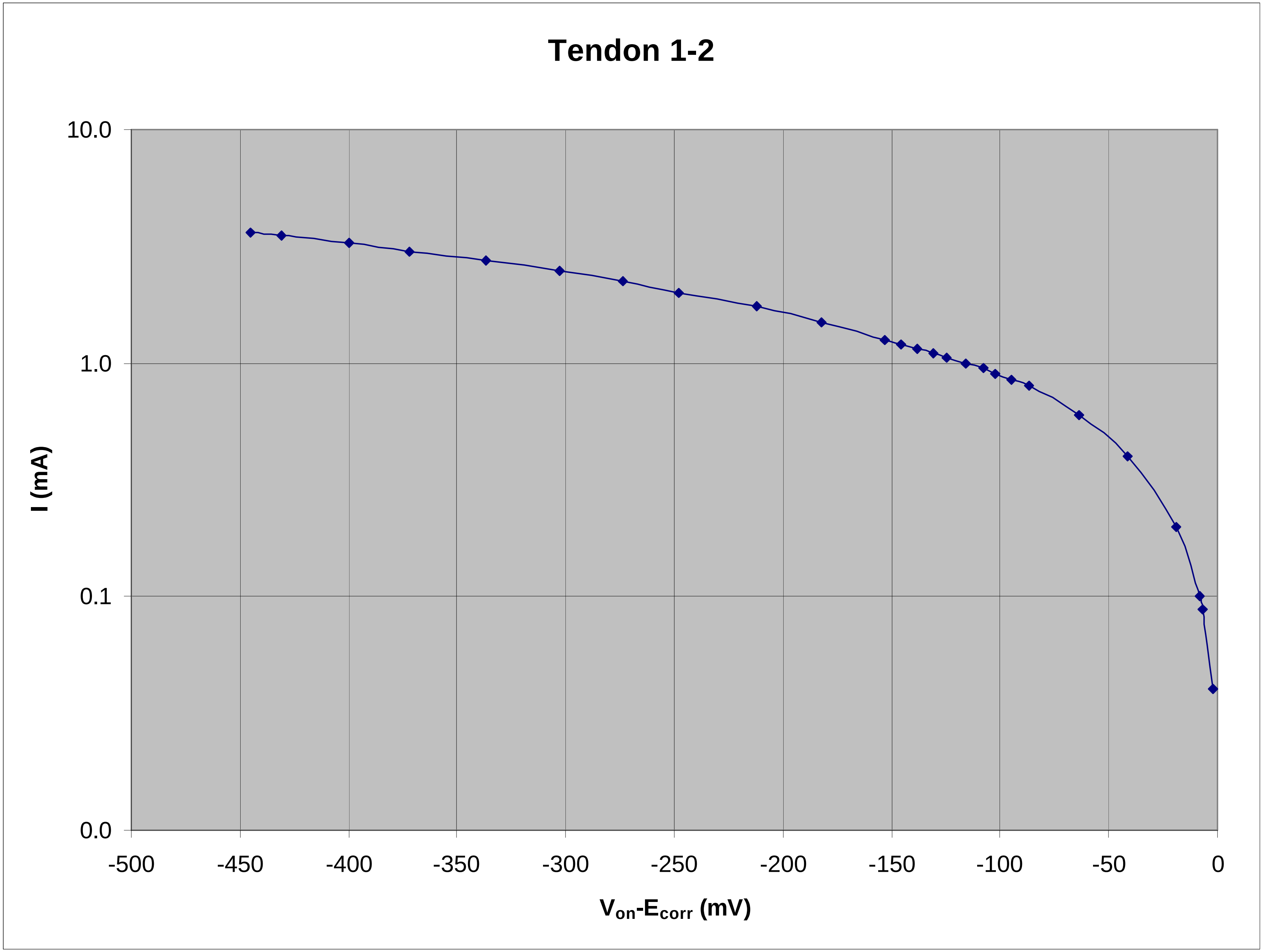
Bolt #21
10.0
1.0
0.1
0.0
I (mA)
-4500
-4000
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
V_{on}-E_{corr} (mV)

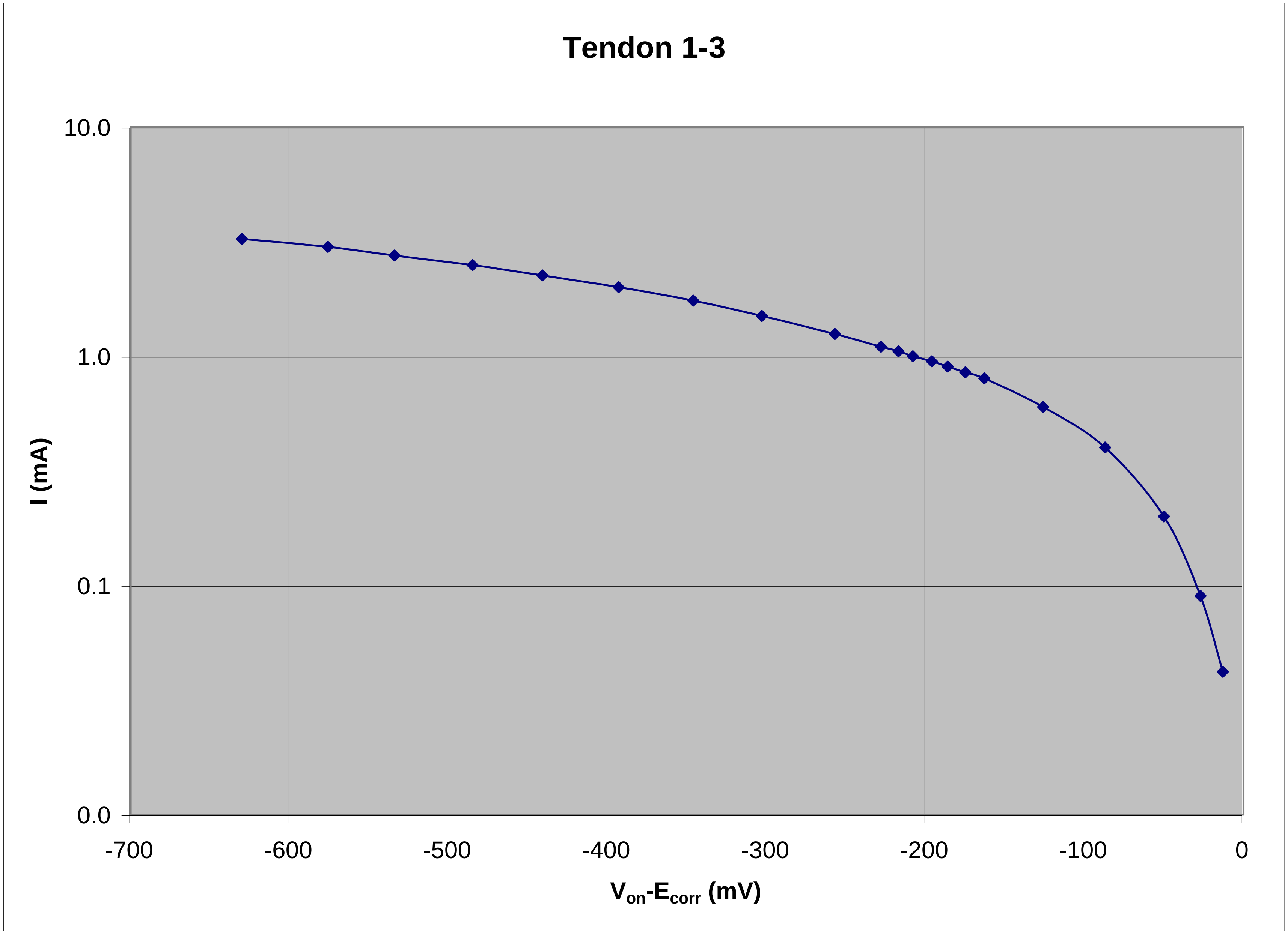


Bolt #22
10.0
1.0
0.1
0.0
I (mA)
-3000
-2500
-2000
-1500
-1000
-500
0
Von-Ecorr (mV)

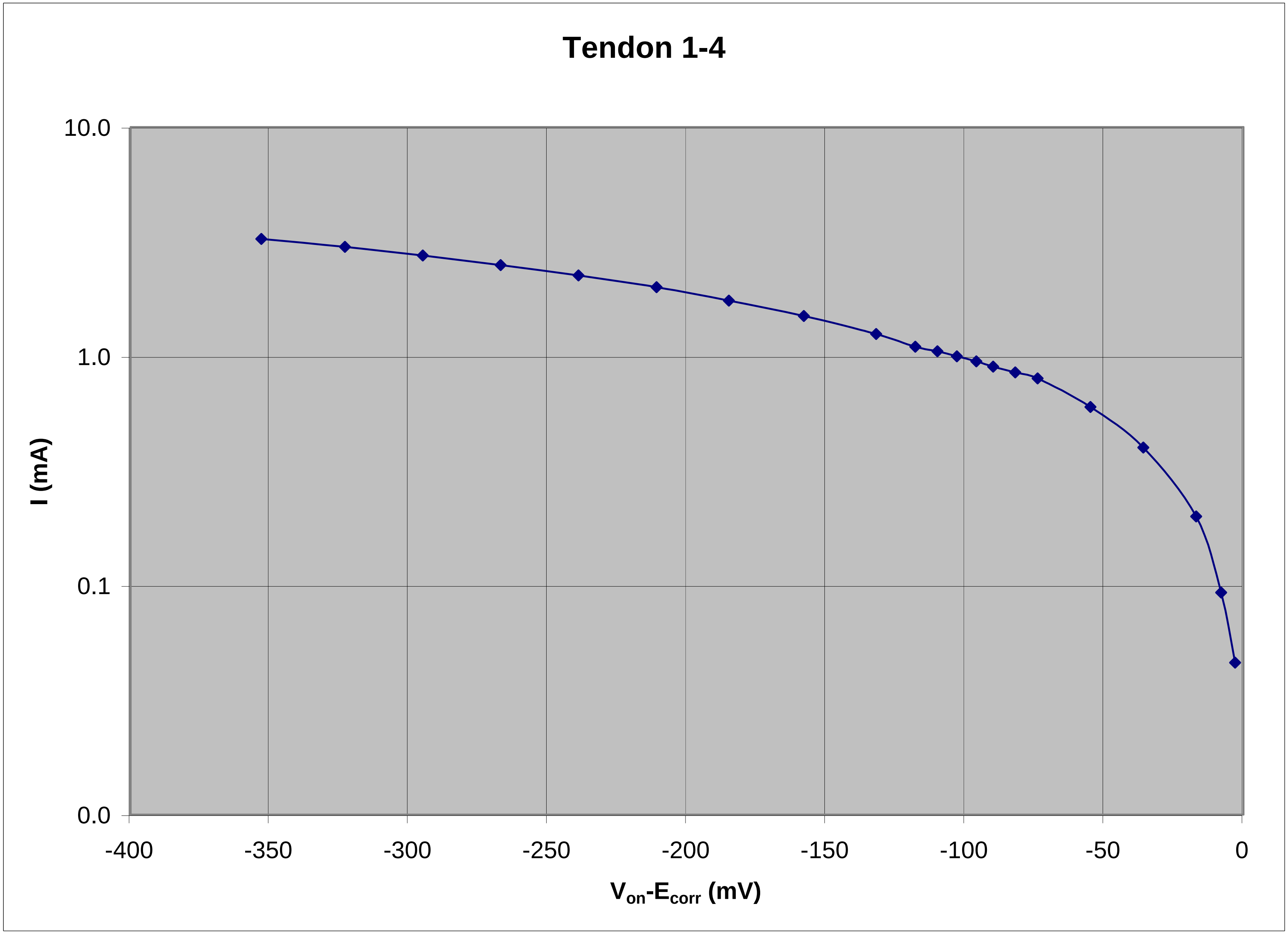


Tendon 1-1
10.0
1.0
0.1
0.0
I (mA)
-600
-500
-400
-300
-200
-100
0
Von-Ecorr (mV)

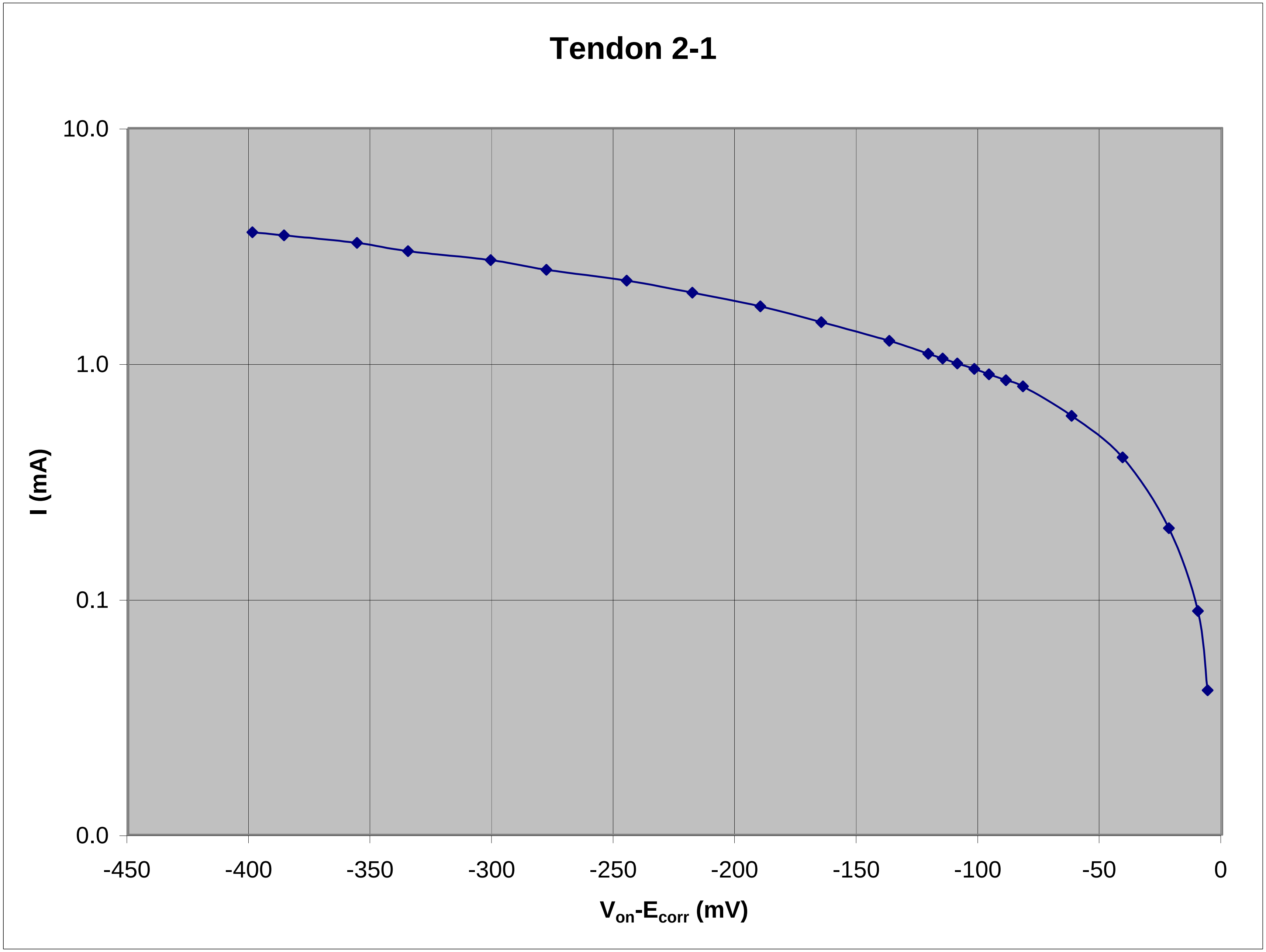

Tendon 1-2
10.0
1.0
0.1
0.0
I (mA)
-500
-450
-400
-350
-300
-250
-200
-150
-100
-50
0
V_{on}-E_{corr} (mV)

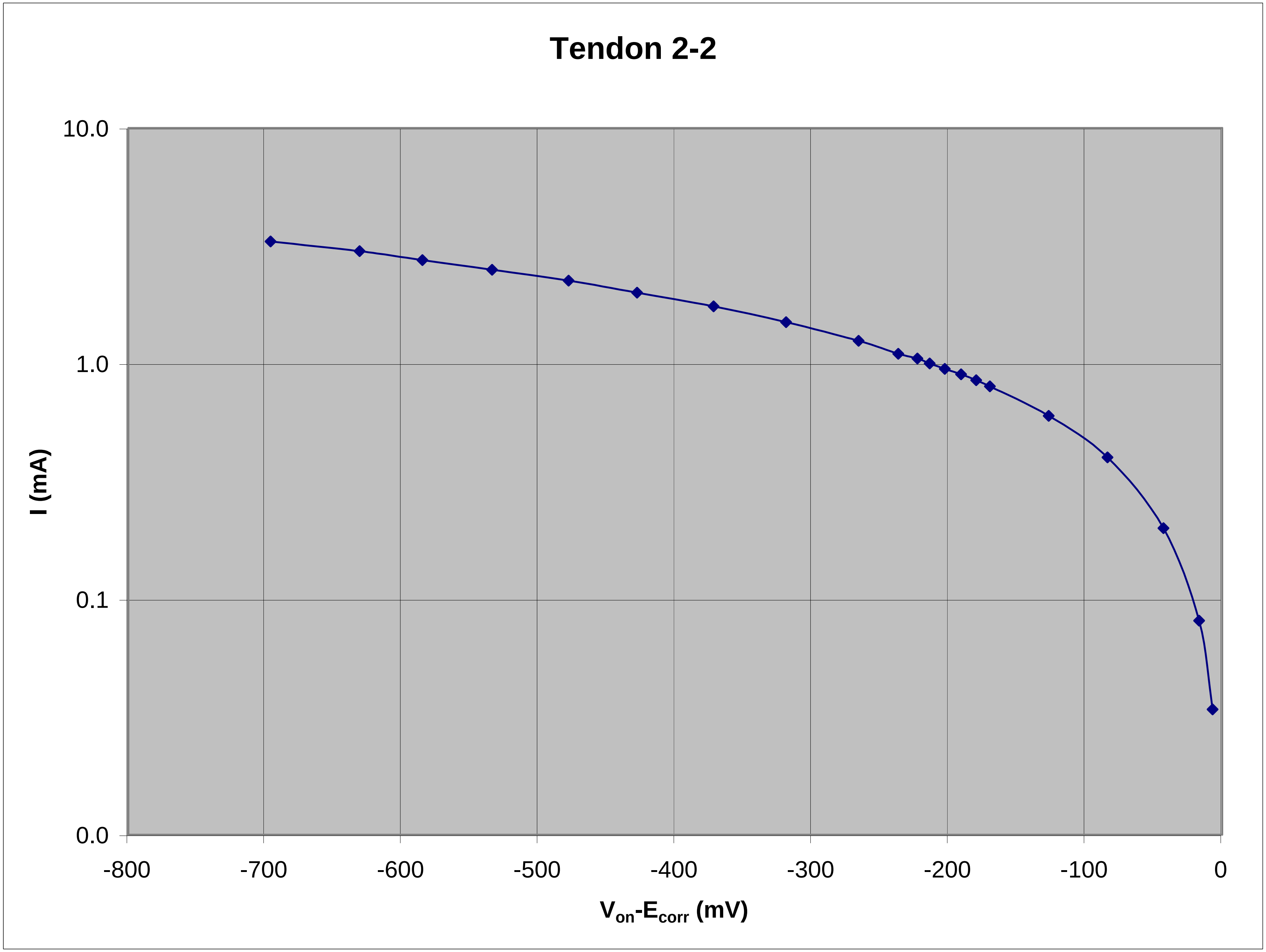


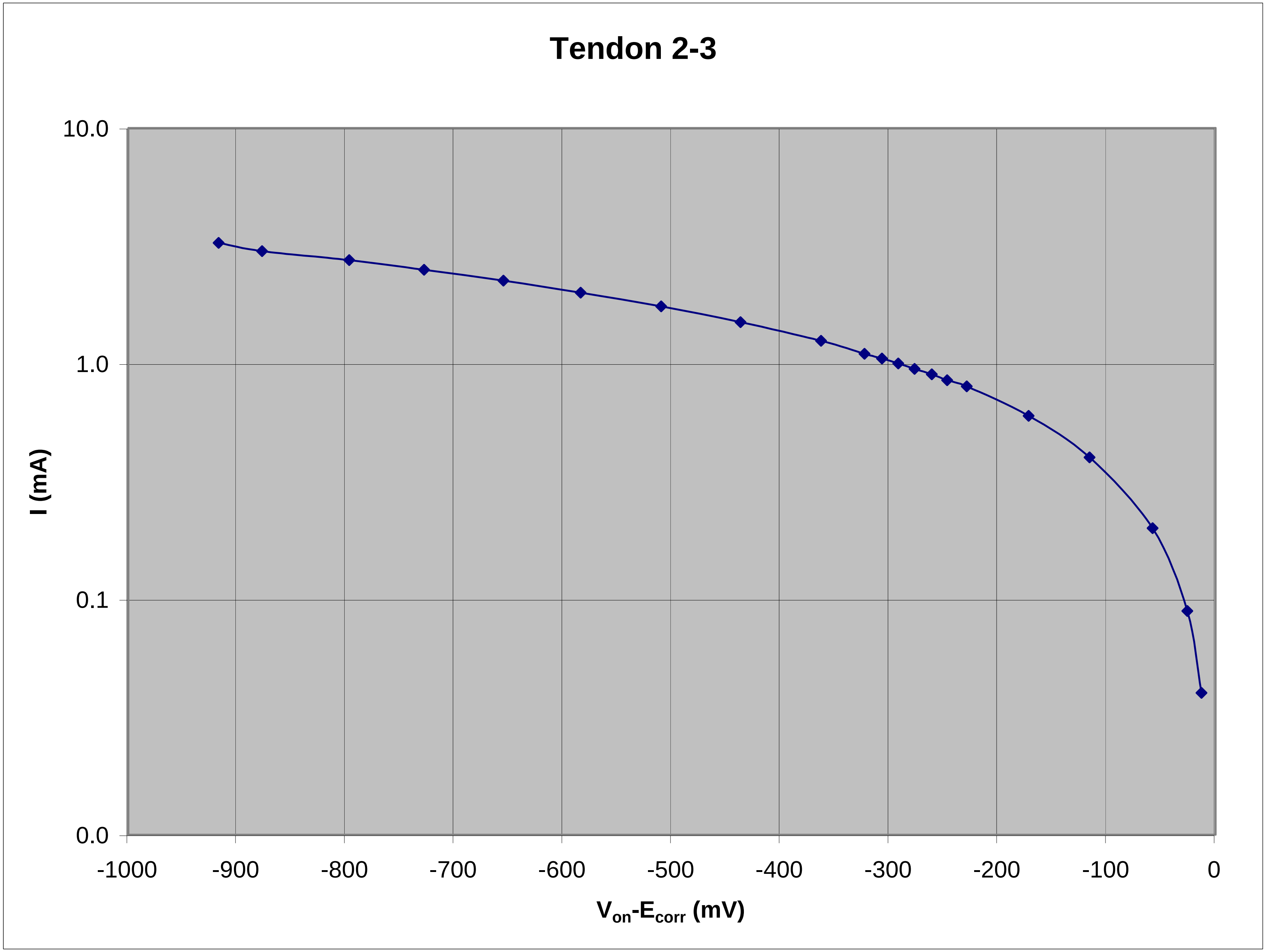
Tendon 1-3
10.0
1.0
0.1
0.0
I (mA)
-700
-600
-500
-400
-300
-200
-100
0
V_{on}-E_{corr} (mV)



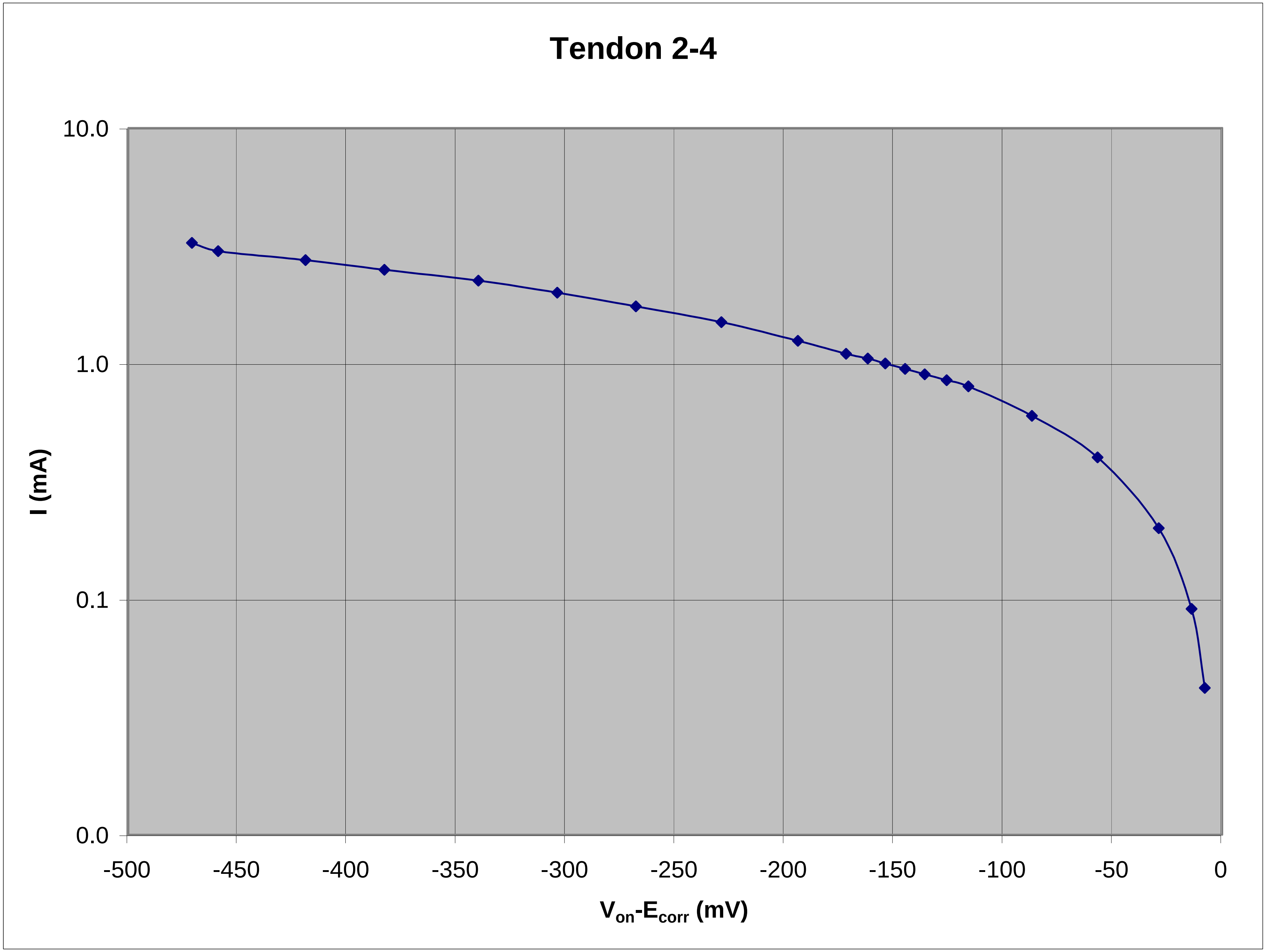
Tendon 1-4
10.0
1.0
0.1
0.0
I (mA)
-400
-350
-300
-250
-200
-150
-100
-50
0
V_{on}-E_{corr} (mV)


Tendon 2-1
10.0
1.0
0.1
0.0
I (mA)
-450
-400
-350
-300
-250
-200
-150
-100
-50
0
V_on-E_corr (mV)

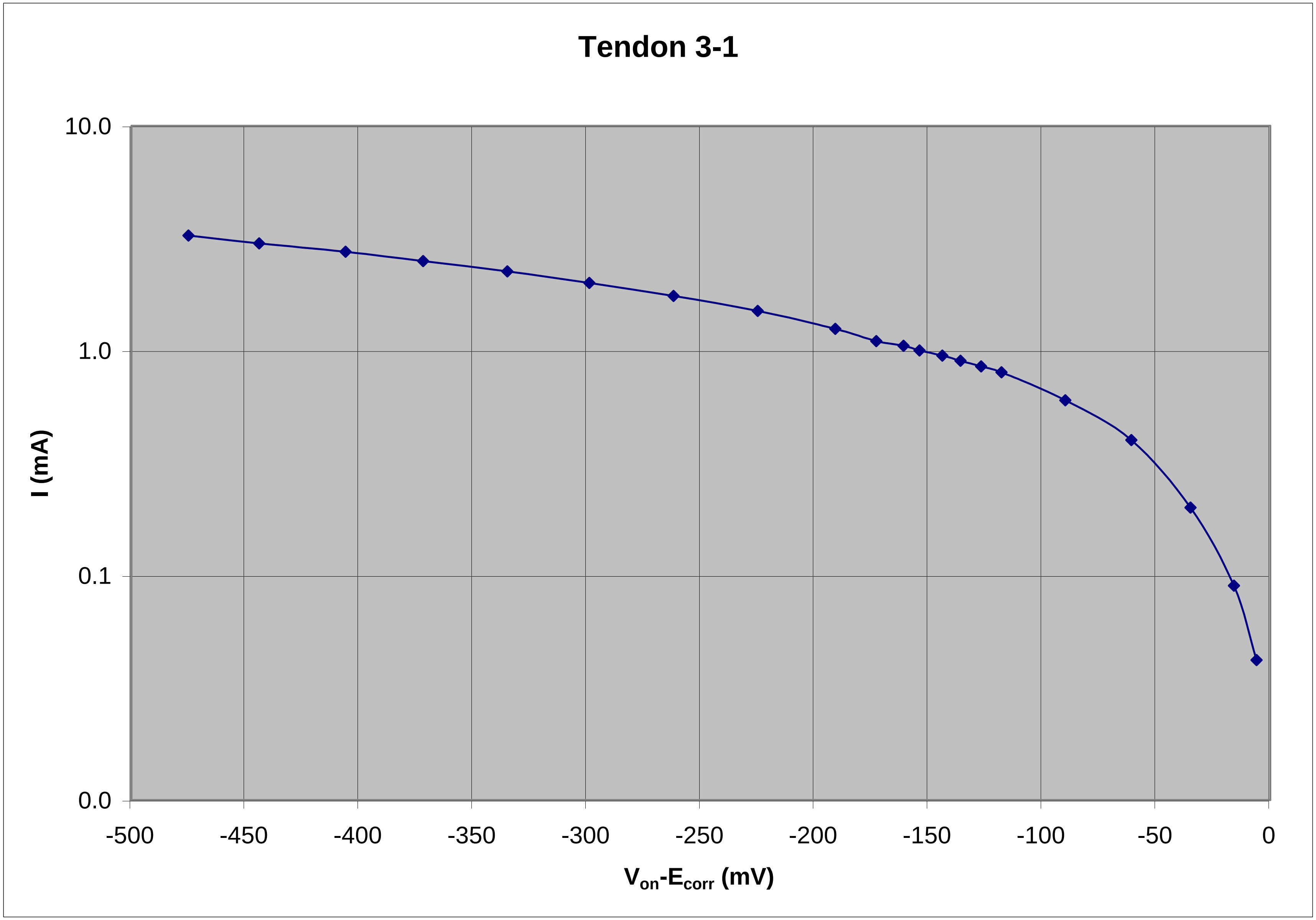

Tendon 2-2
10.0
1.0
0.1
0.0
I (mA)
-800
-700
-600
-500
-400
-300
-200
-100
0
V_{on}-E_{corr} (mV)



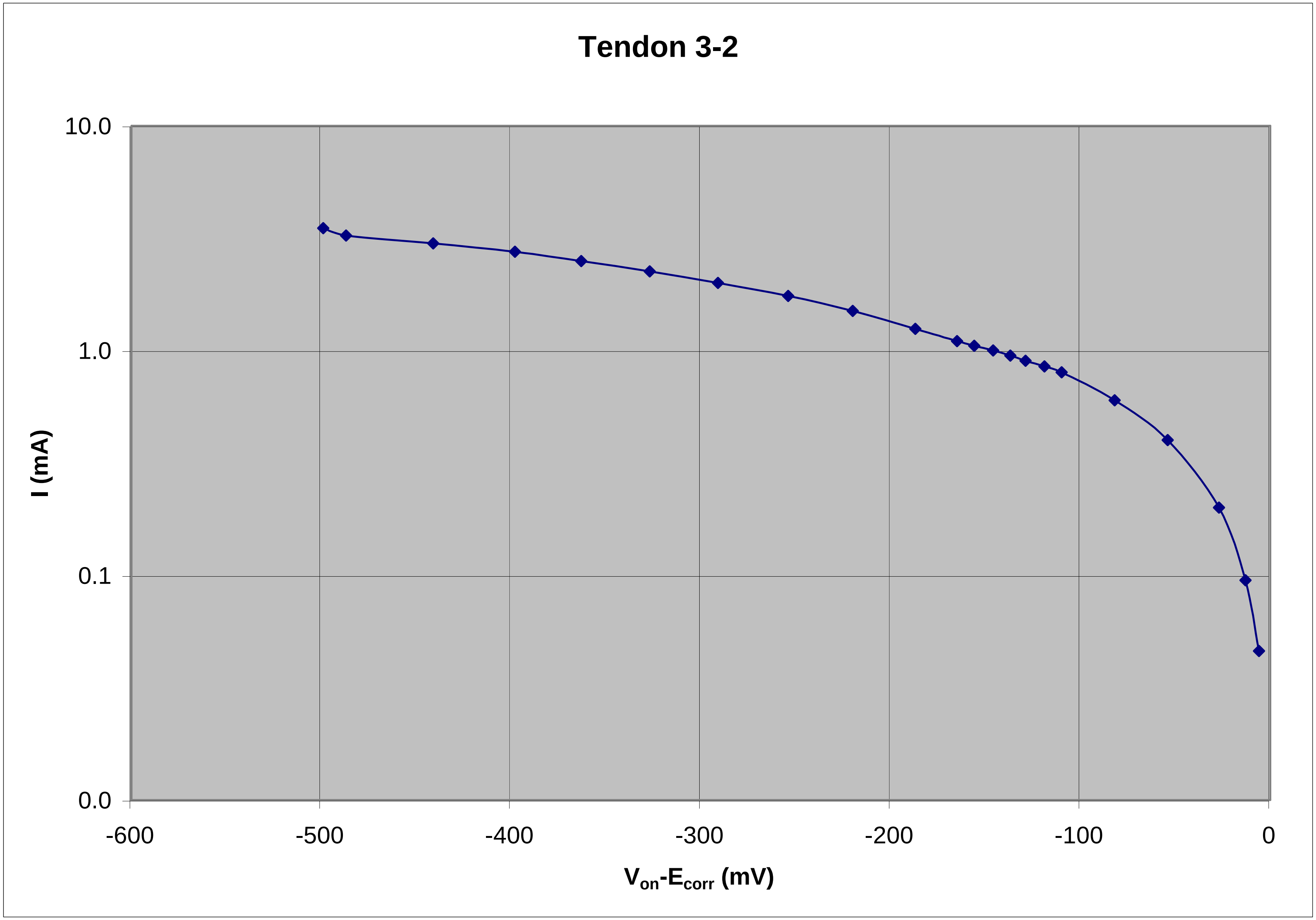
Tendon 2-3
10.0
1.0
0.1
0.0
I (mA)
-1000
-900
-800
-700
-600
-500
-400
-300
-200
-100
0
V_{on}-E_{corr} (mV)



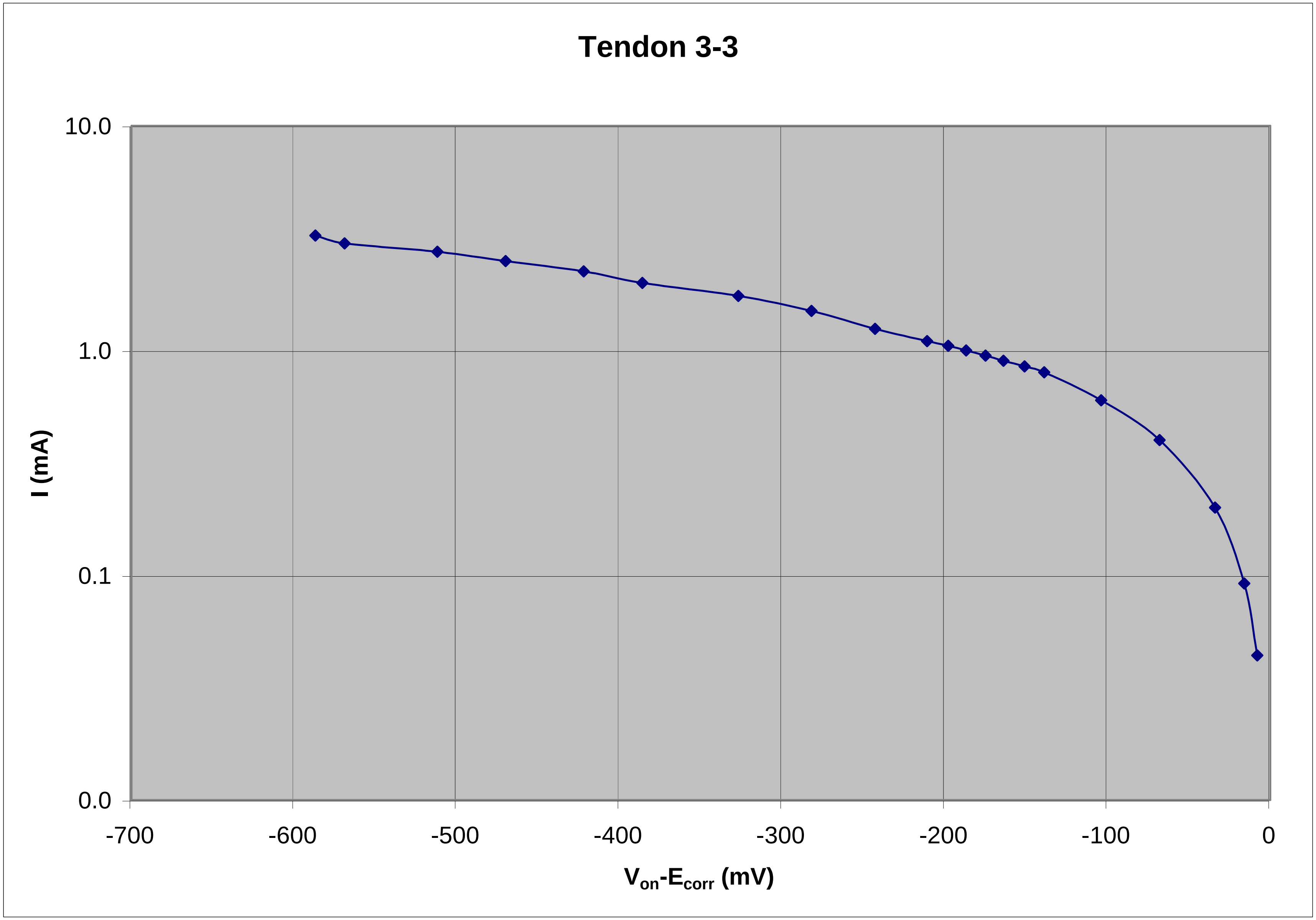
Tendon 2-4
10.0
1.0
0.1
0.0
I (mA)
-500
-450
-400
-350
-300
-250
-200
-150
-100
-50
0
V_{on}-E_{corr} (mV)



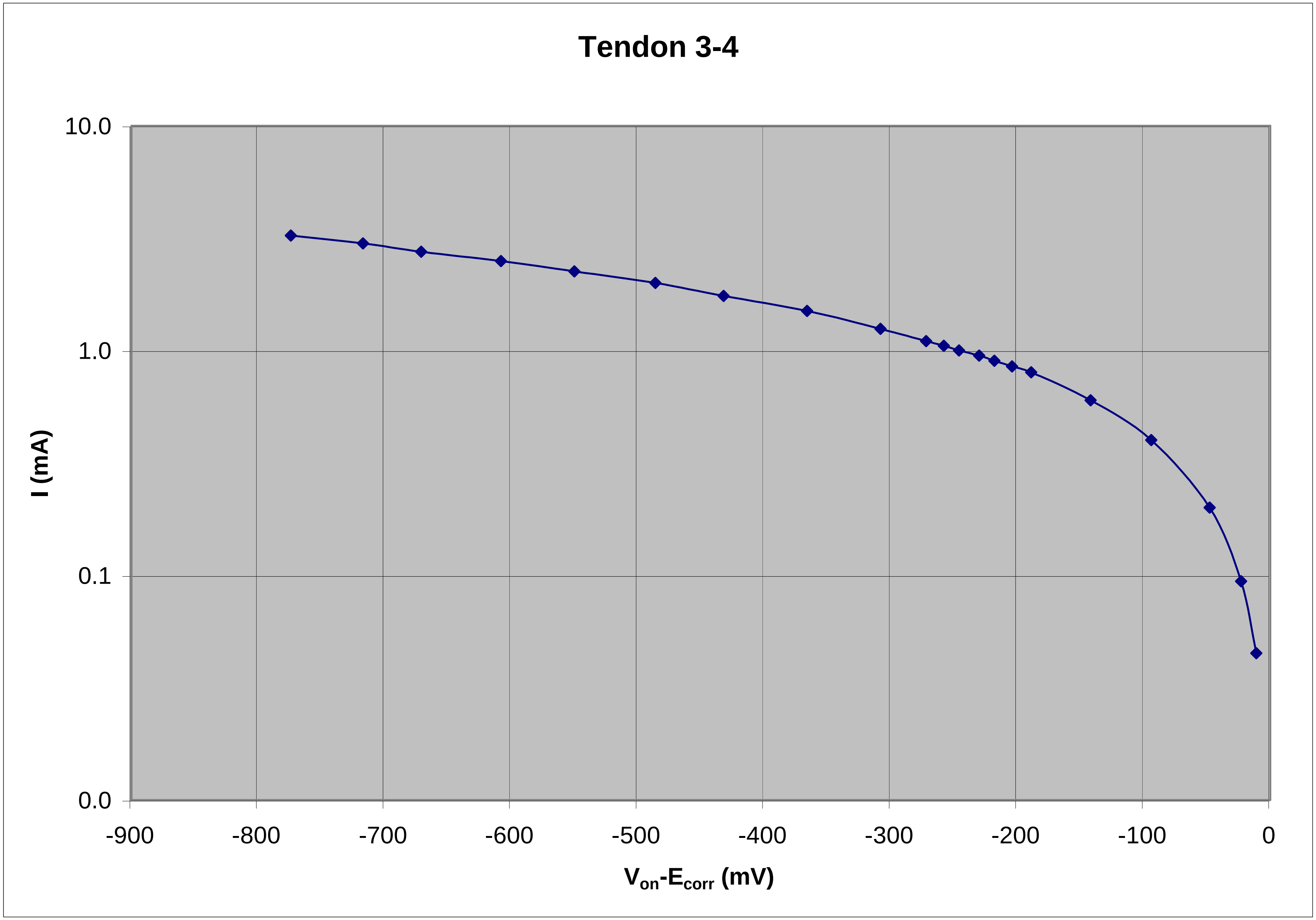
Tendon 3-1
10.0
1.0
0.1
0.0
I (mA)
-500
-450
-400
-350
-300
-250
-200
-150
-100
-50
0
V_{on}-E_{corr} (mV)



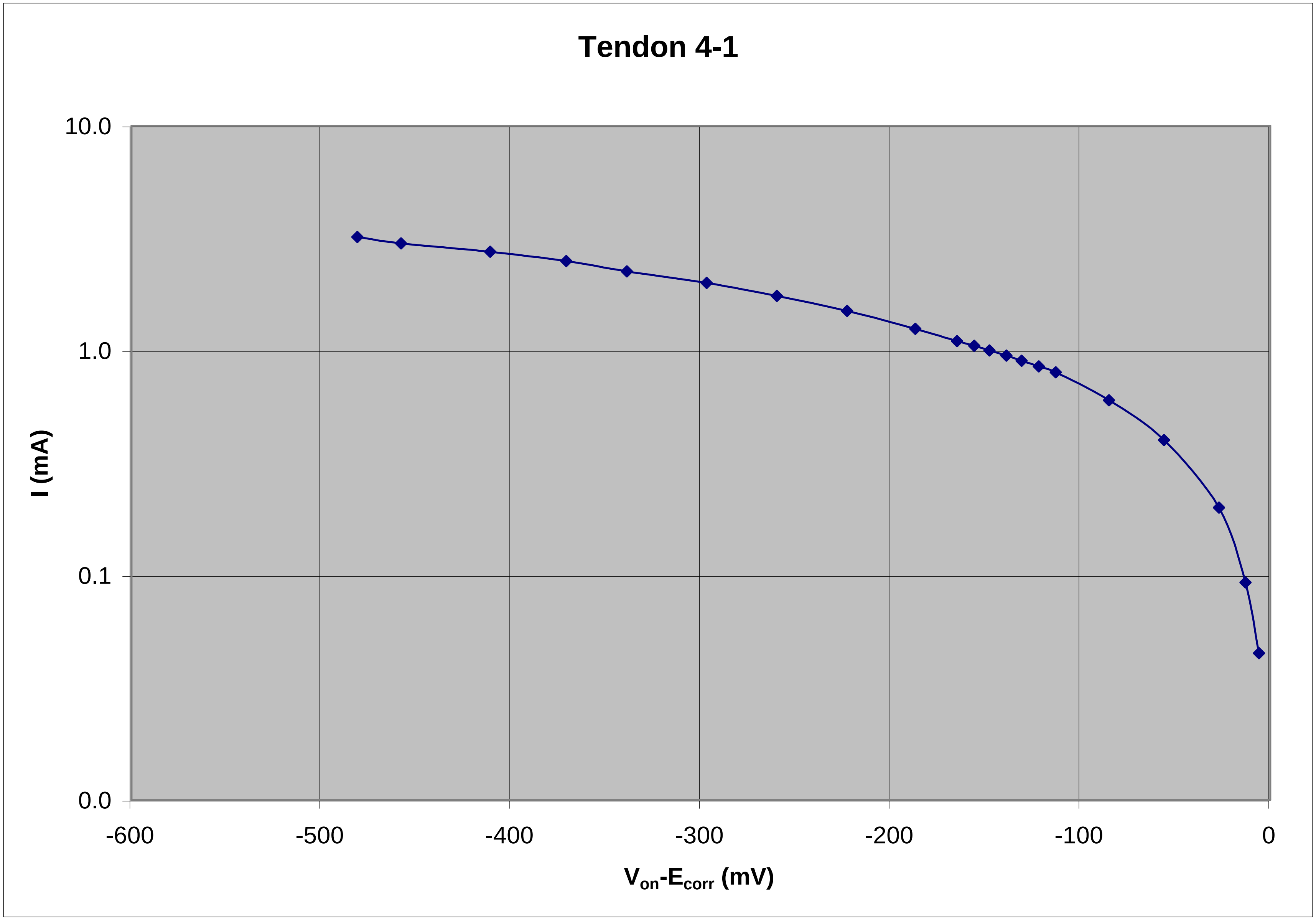
Tendon 3-2
10.0
1.0
0.1
0.0
I (mA)
-600
-500
-400
-300
-200
-100
0
V_{on}-E_{corr} (mV)



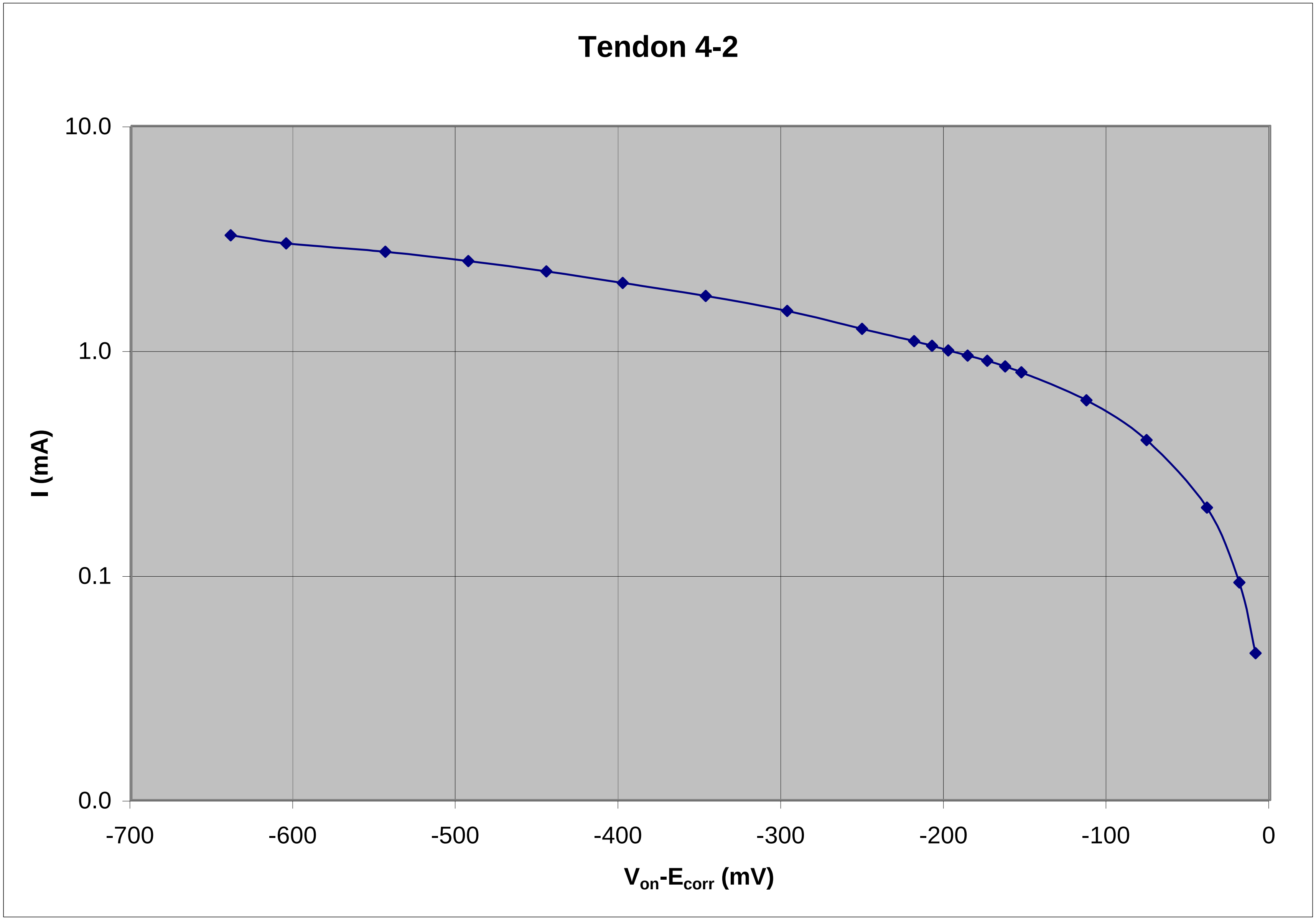
Tendon 3-3
10.0
1.0
0.1
0.0
I (mA)
-700
-600
-500
-400
-300
-200
-100
0
V_{on}-E_{corr} (mV)



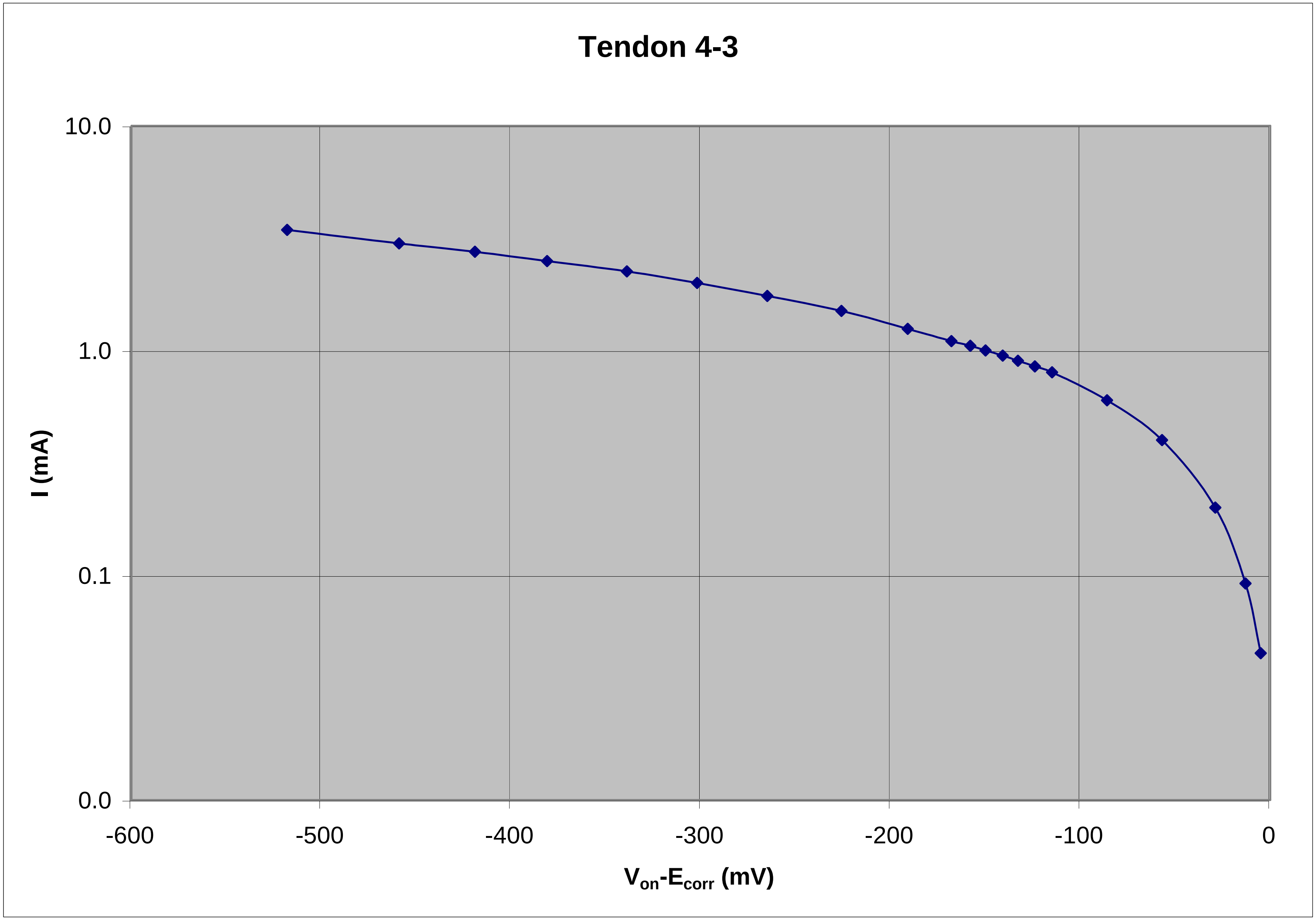
Tendon 3-4
10.0
1.0
0.1
0.0
I (mA)
-900
-800
-700
-600
-500
-400
-300
-200
-100
0
V_{on}-E_{corr} (mV)



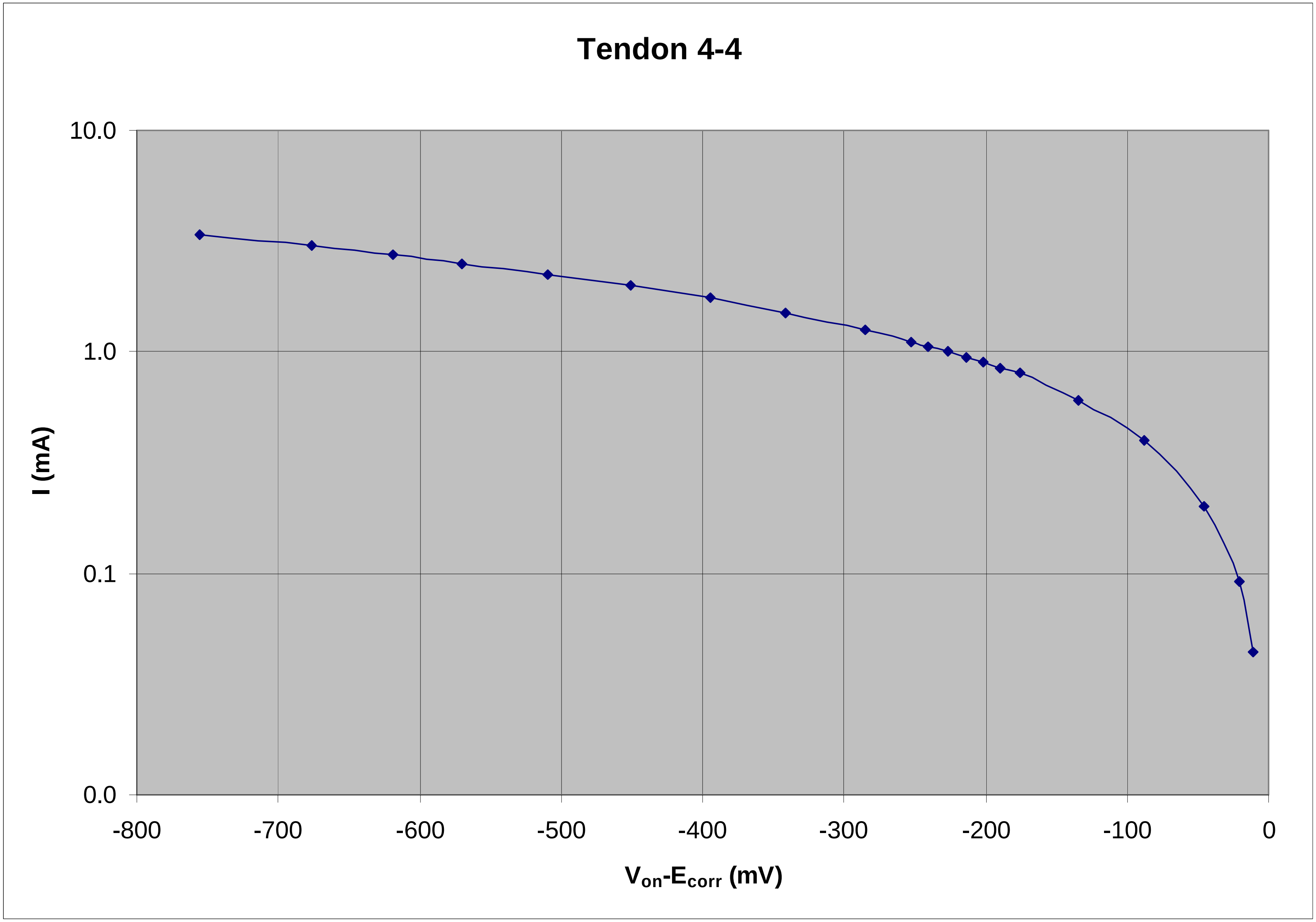
Tendon 4-1
10.0
1.0
0.1
0.0
I (mA)
-600
-500
-400
-300
-200
-100
0
V_{on}-E_{corr} (mV)



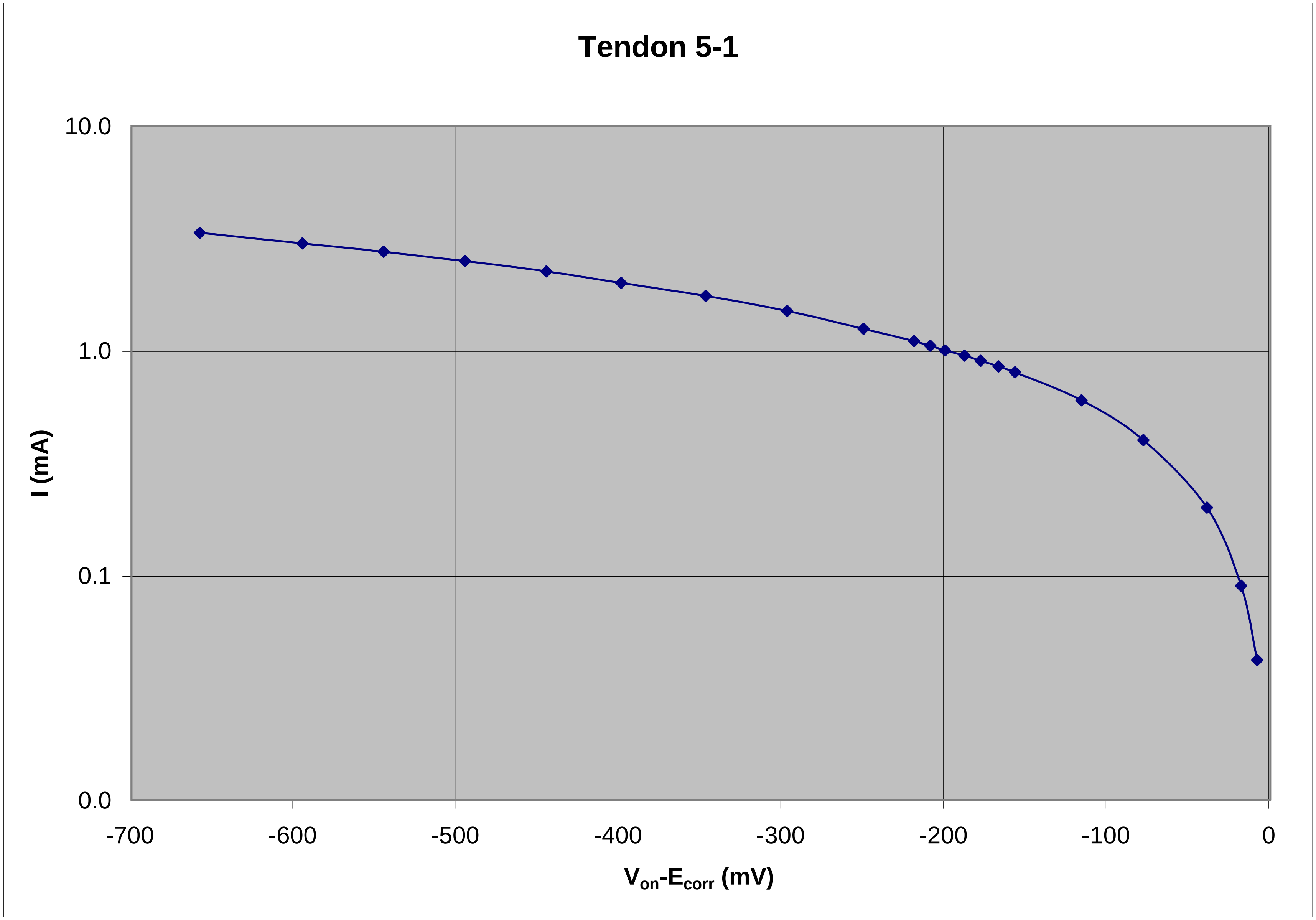
Tendon 4-2
10.0
1.0
0.1
0.0
I (mA)
-700
-600
-500
-400
-300
-200
-100
0
V_{on}-E_{corr} (mV)

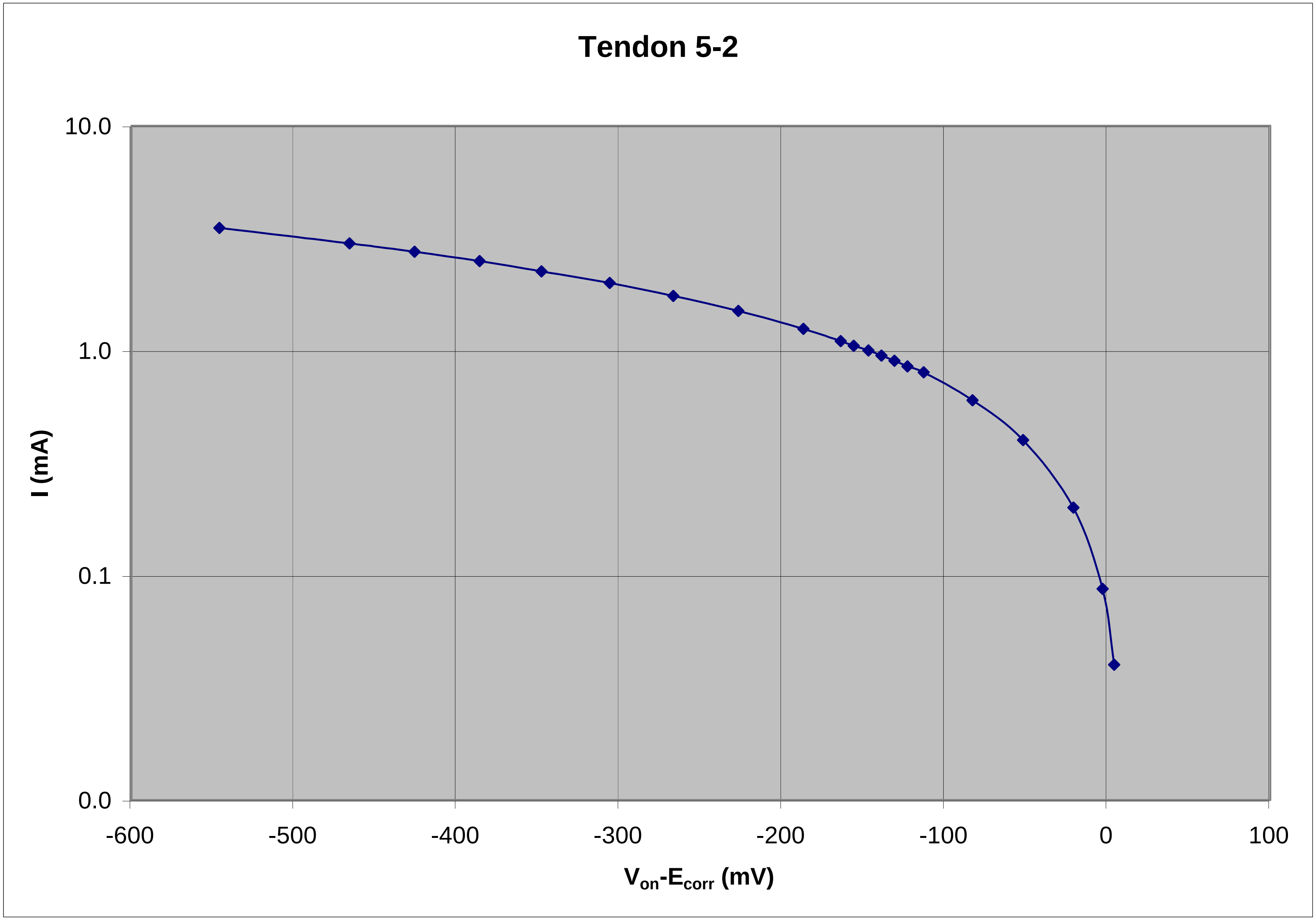


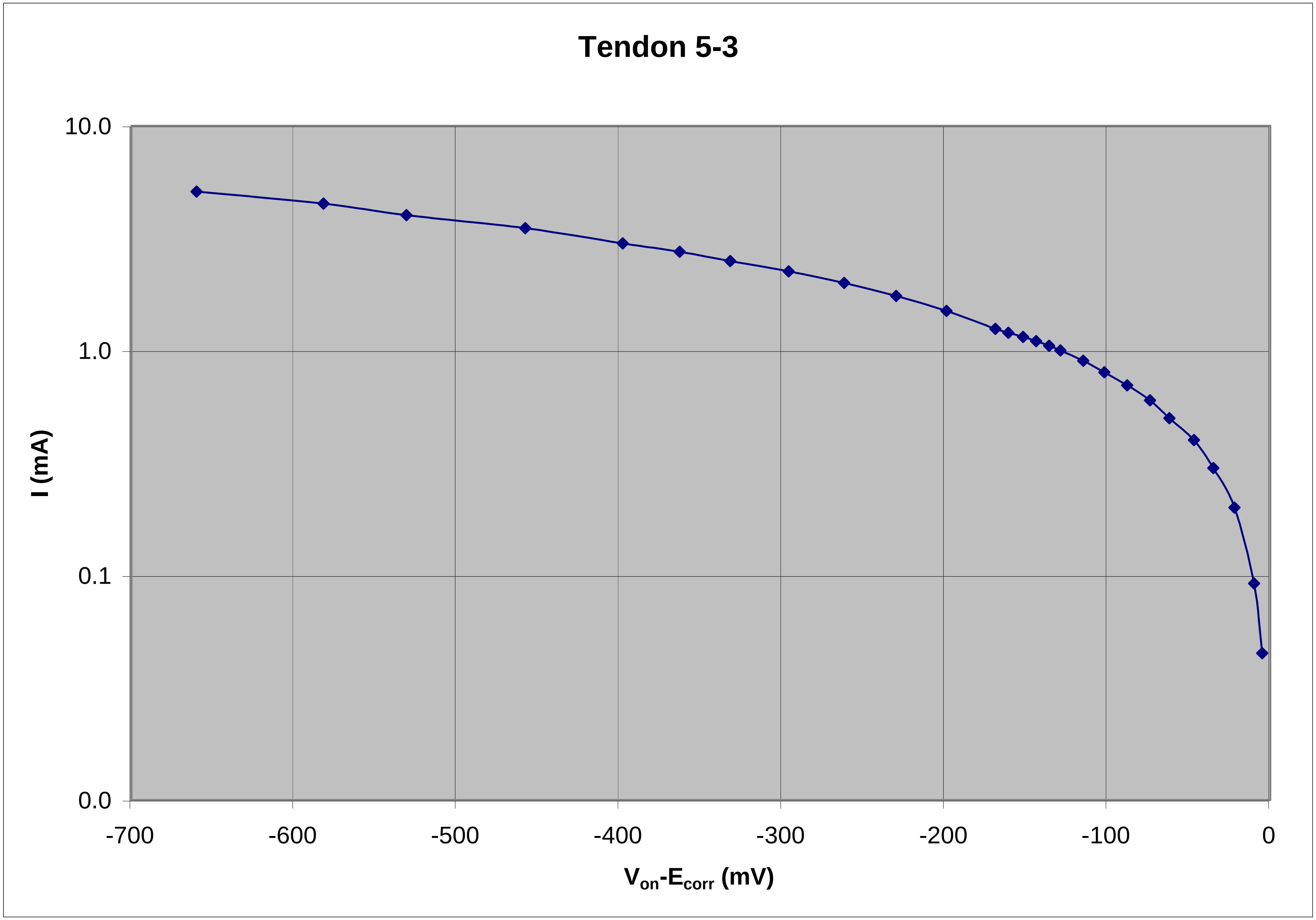
Tendon 4-3
10.0
1.0
0.1
0.0
I (mA)
-600
-500
-400
-300
-200
-100
0
V_{on}-E_{corr} (mV)



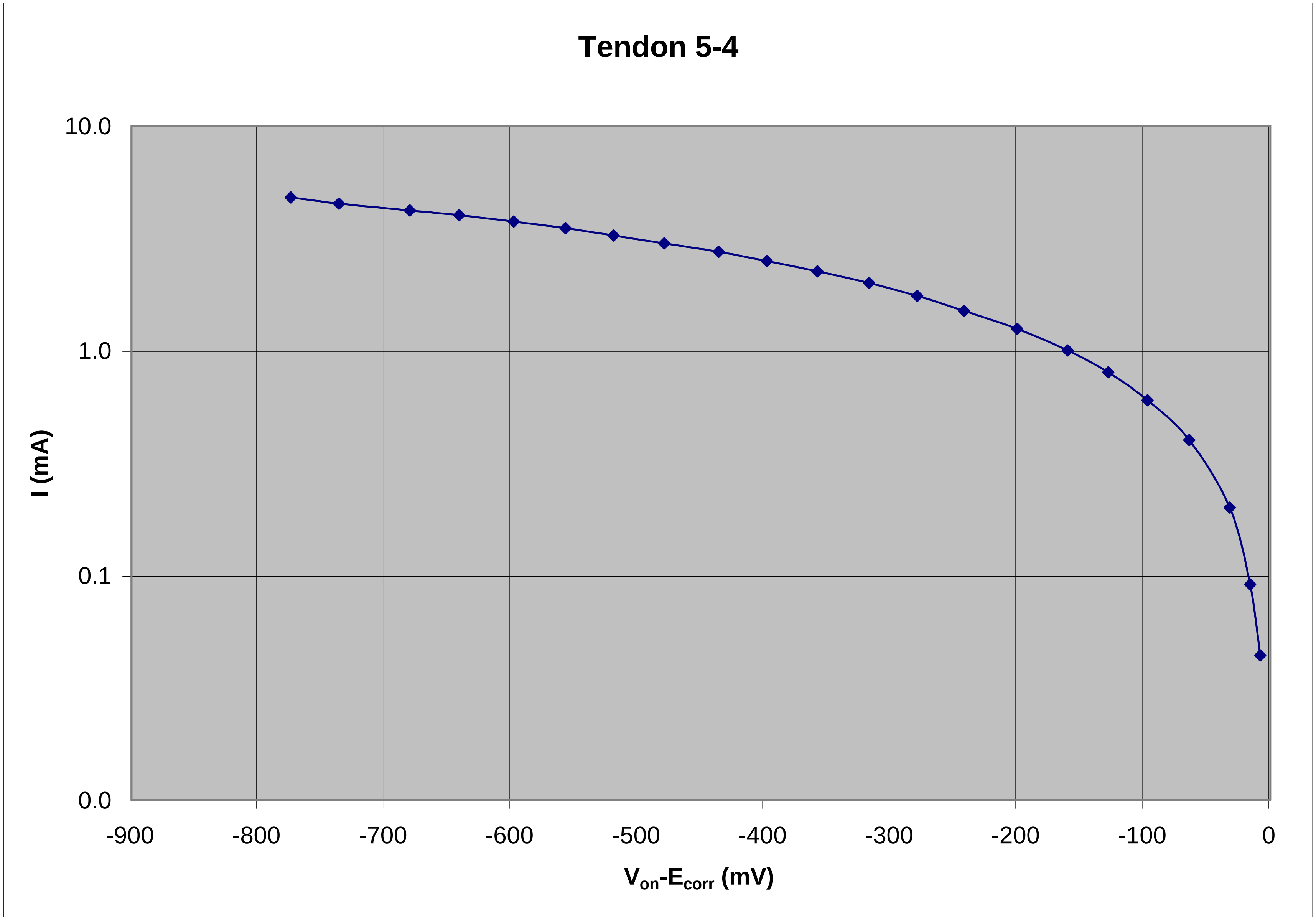
Tendon 4-4
10.0
1.0
0.1
0.0
I (mA)
-800
-700
-600
-500
-400
-300
-200
-100
0
Von-Ecorr (mV)


Tendon 5-1
10.0
1.0
0.1
0.0
I (mA)
-700
-600
-500
-400
-300
-200
-100
0
V_{on}-E_{corr} (mV)


Tendon 5-2
I (mA)
10.0
1.0
0.1
0.0
-600
-500
-400
-300
-200
-100
0
100
V_on-E_corr (mV)

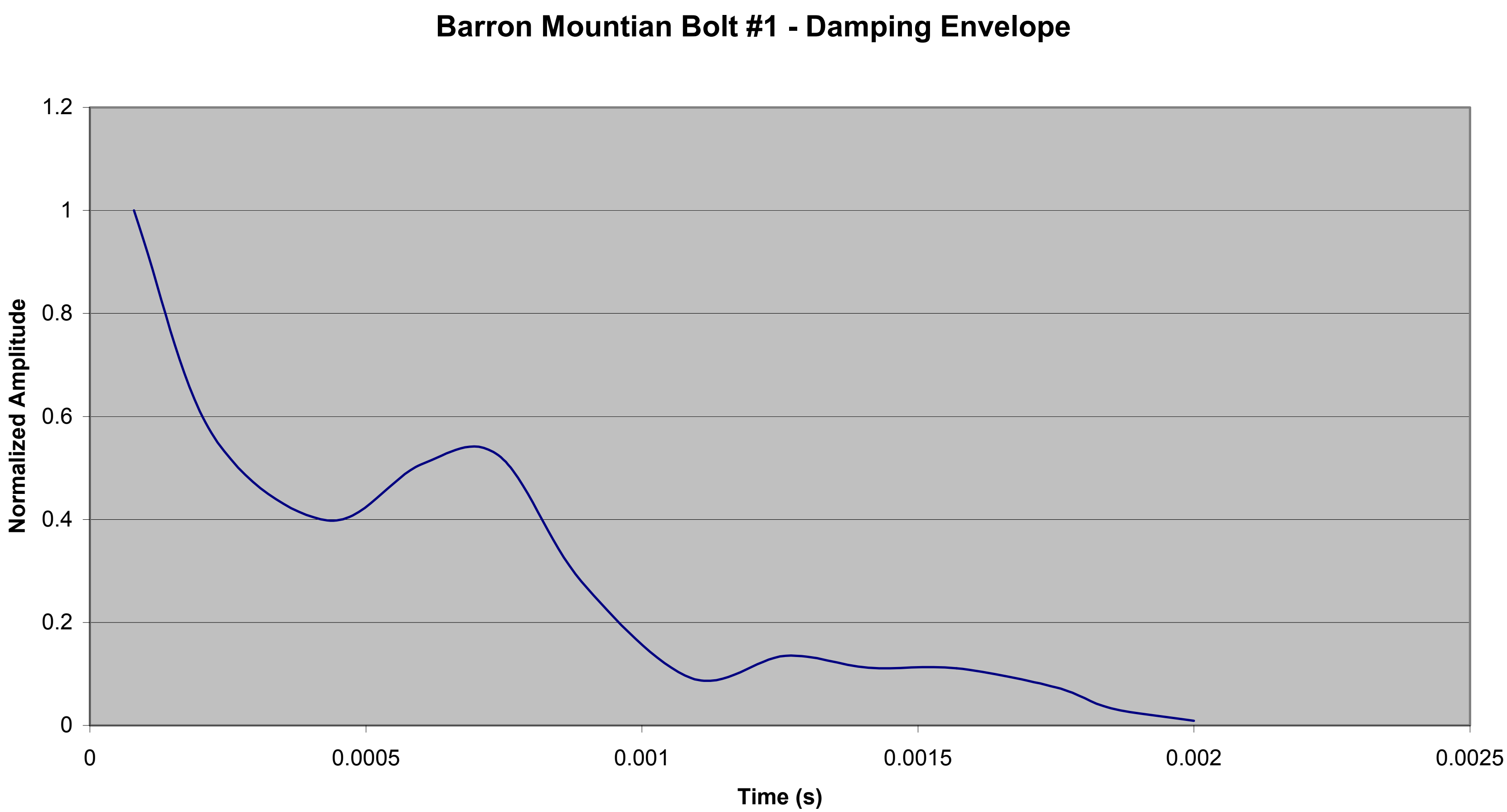


Tendon 5-3
10.0
1.0
0.1
0.0
I (mA)
-700
-600
-500
-400
-300
-200
-100
0
V_{on}-E_{corr} (mV)

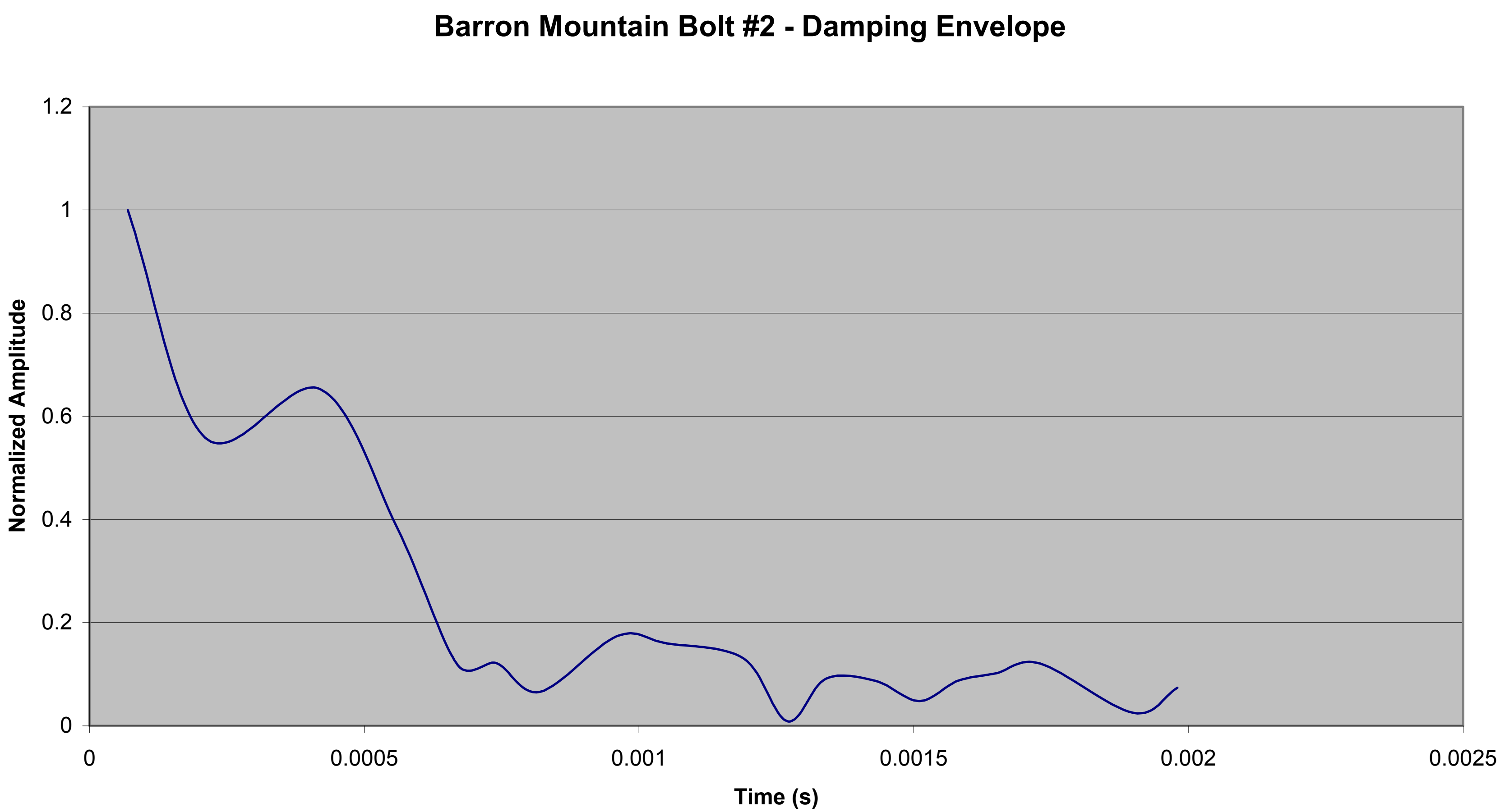

Tendon 5-4
10.0
1.0
0.1
0.0
I (mA)
-900
-800
-700
-600
-500
-400
-300
-200
-100
0
V_{on}-E_{corr} (mV)

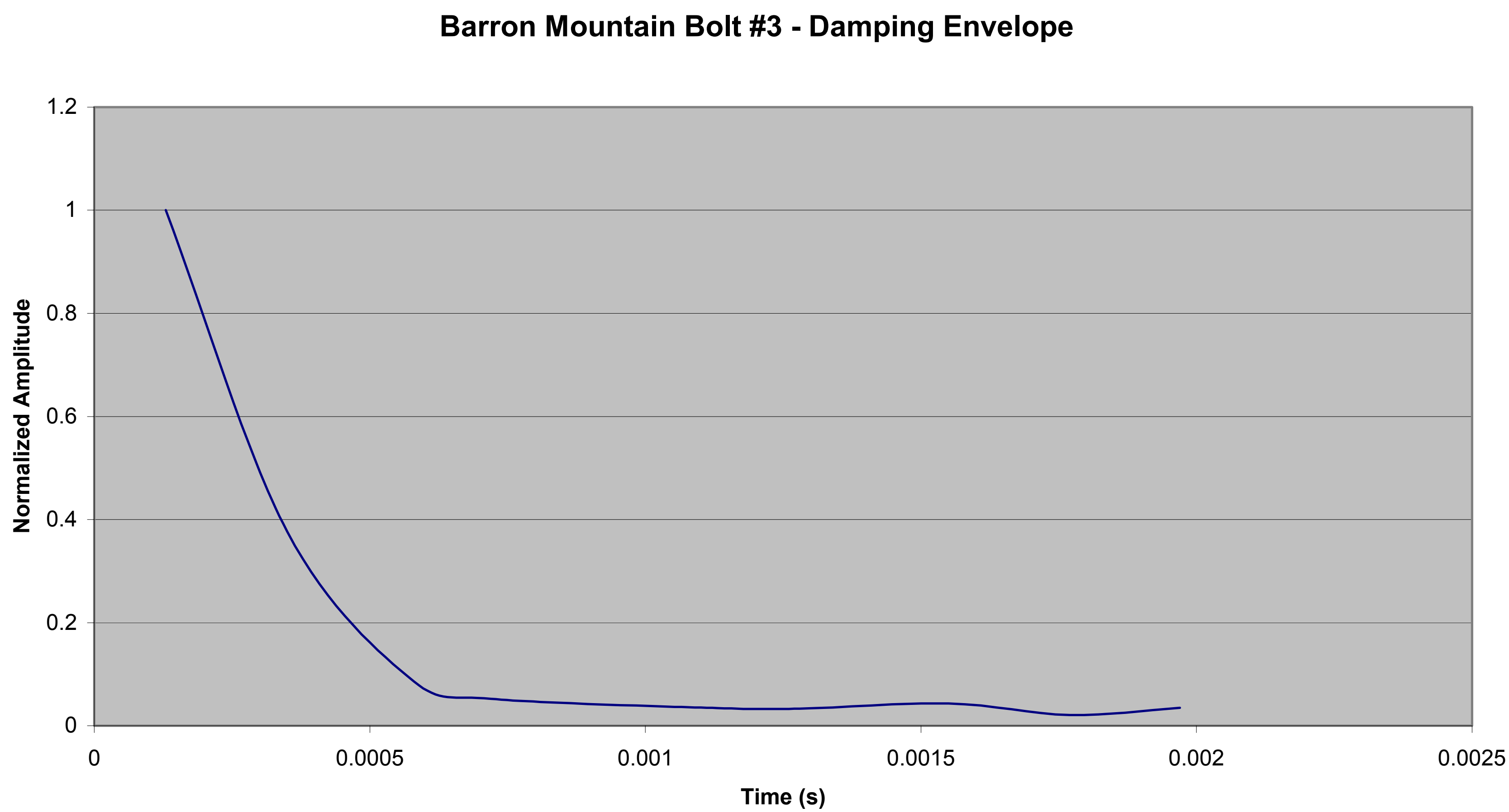
APPENDIX VI

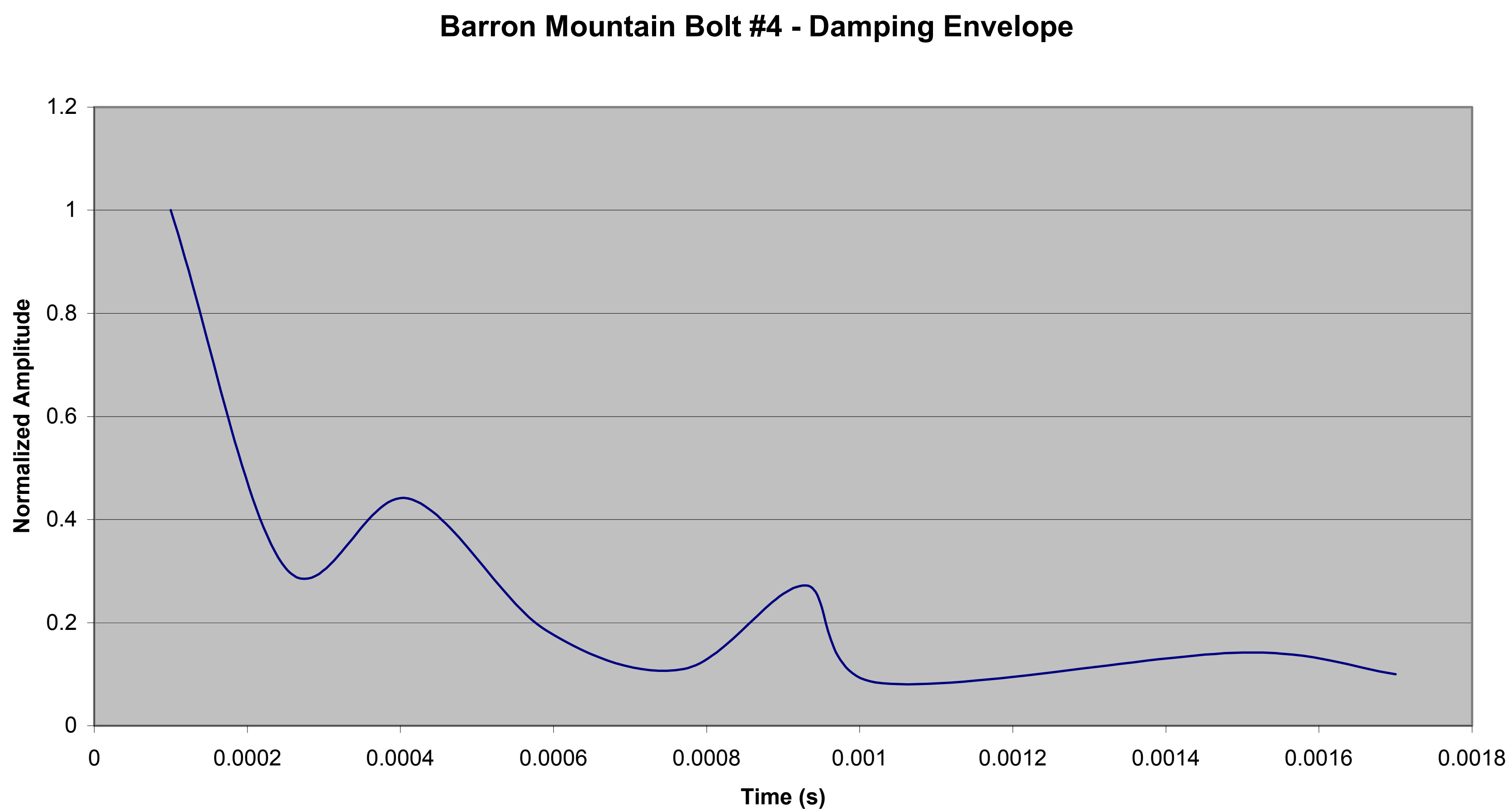
DAMPING ENVELOPES FOR ROCK BOLTS

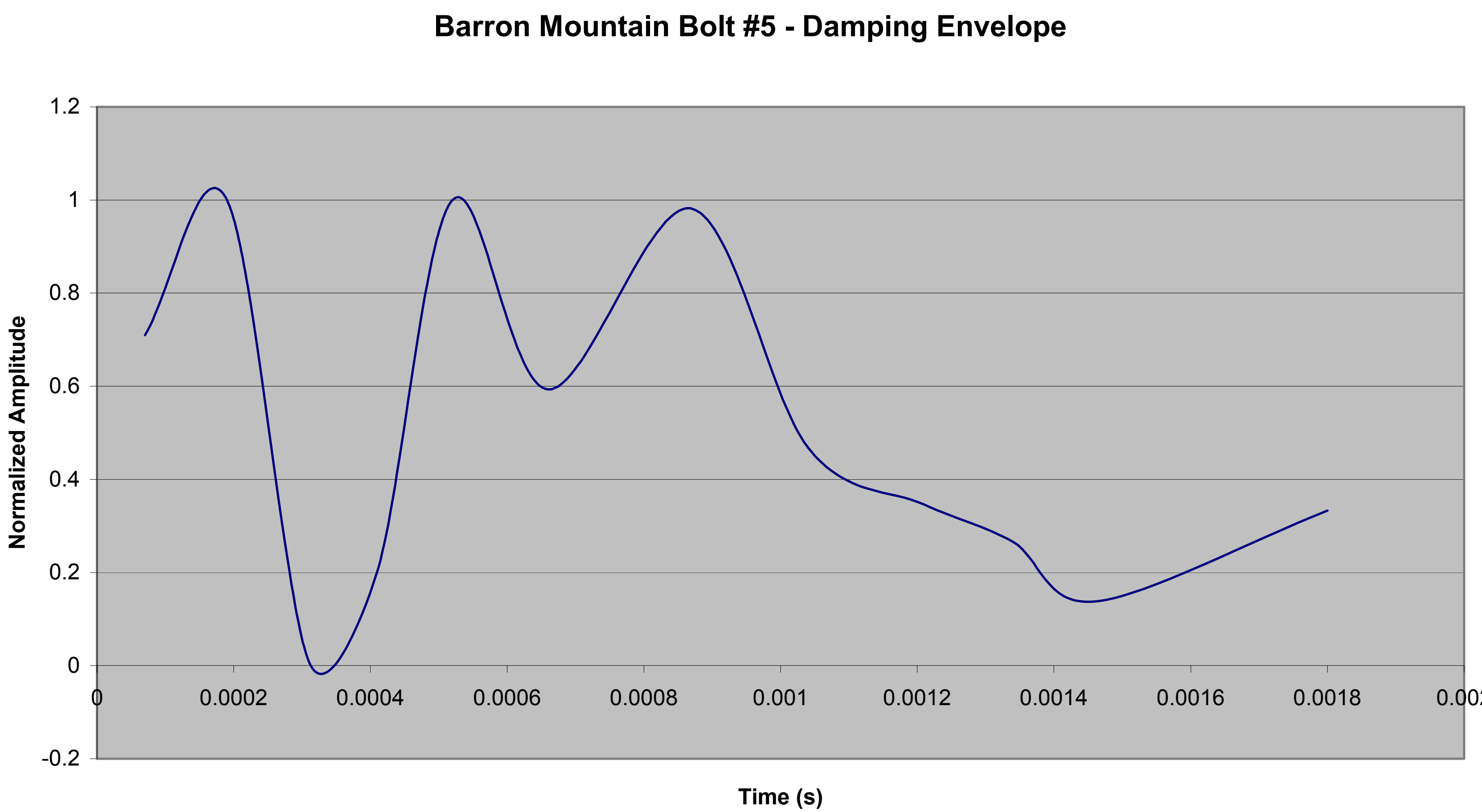


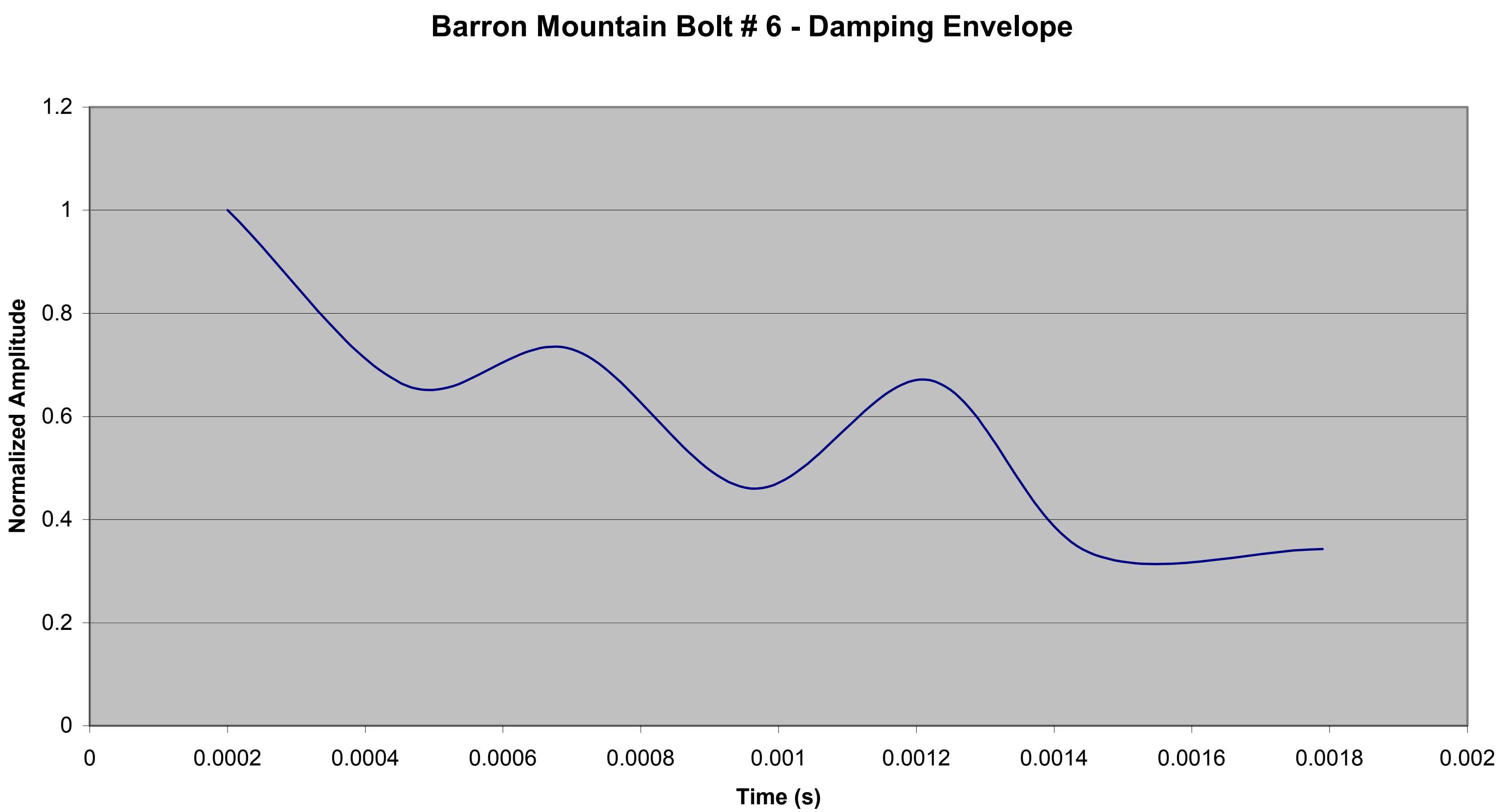
Barron Mountian Bolt #1 - Damping Envelope
Normalized Amplitude
1.2
1
0.8
0.6
0.4
0.2
0
0
0.0005
0.001
0.0015
0.002
0.0025
Time (s)

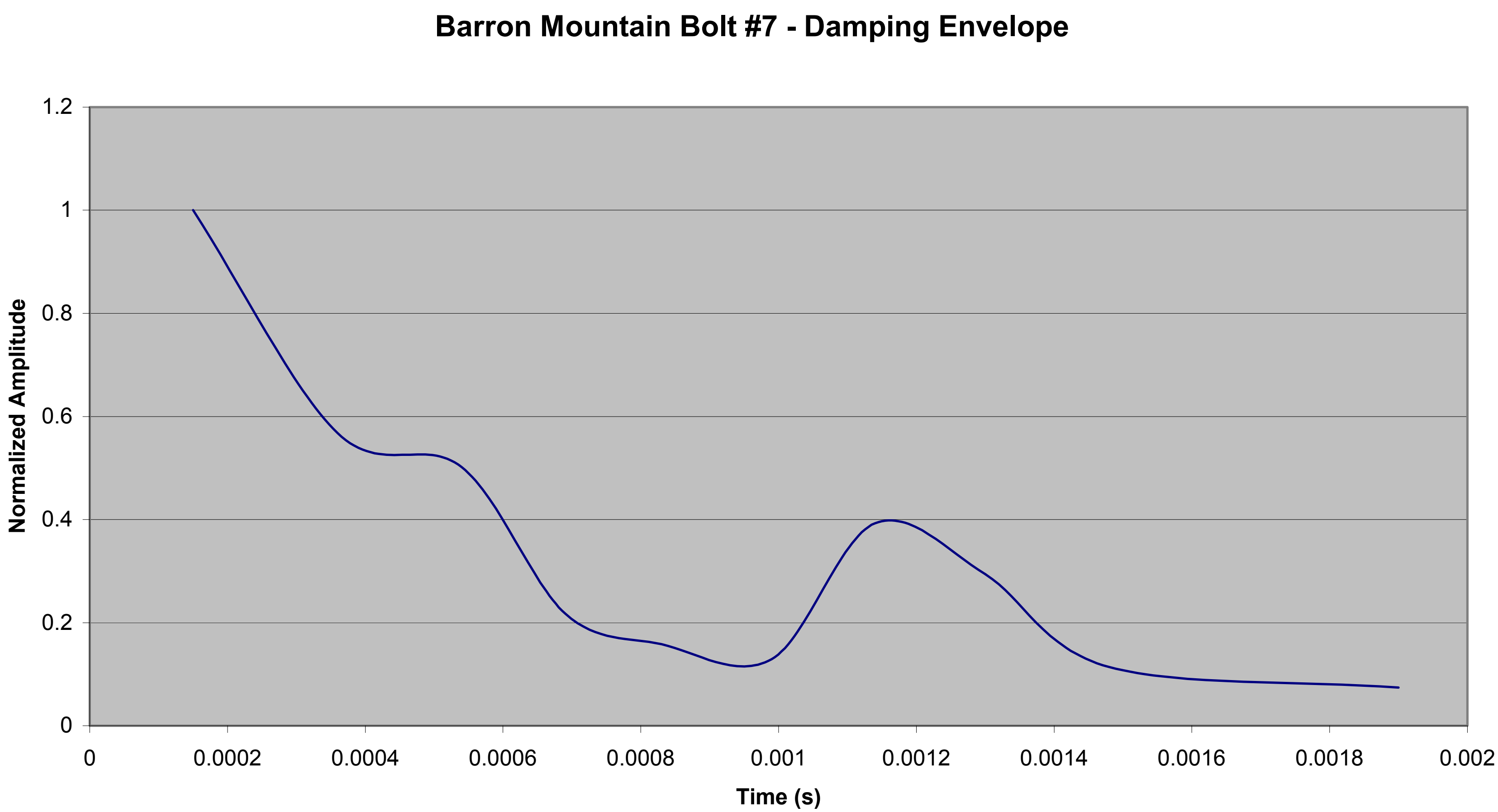

Barron Mountain Bolt #2 - Damping Envelope
Normalized Amplitude
1.2
1
0.8
0.6
0.4
0.2
0
0
0.0005
0.001
0.0015
0.002
0.0025
Time (s)


Barron Mountain Bolt #3 - Damping Envelope
Normalized Amplitude
1.2
1
0.8
0.6
0.4
0.2
0
0
0.0005
0.001
0.0015
0.002
0.0025
Time (s)

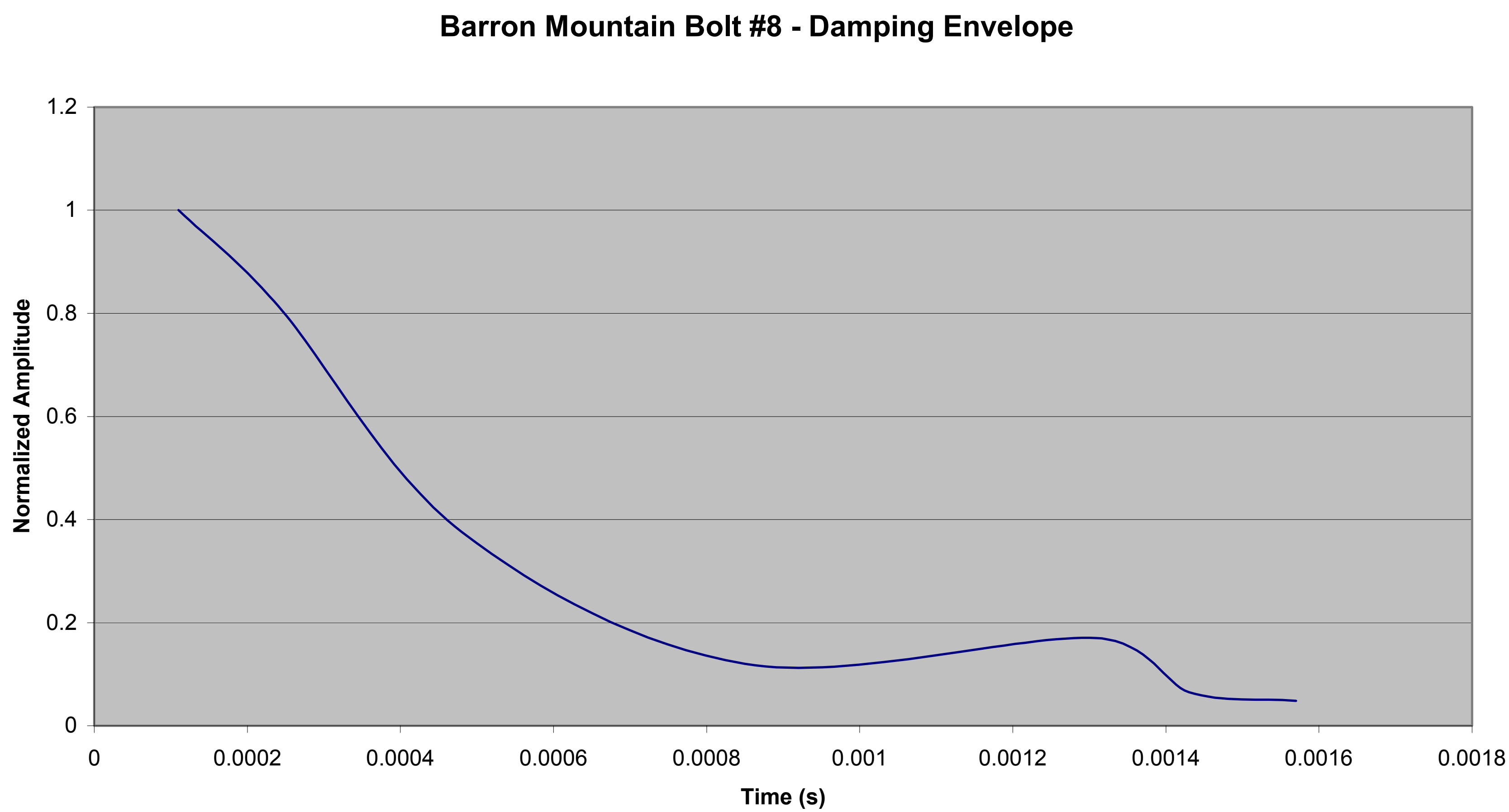

Barron Mountain Bolt #4 - Damping Envelope
Normalized Amplitude
1.2
1
0.8
0.6
0.4
0.2
0
0.0002
0.0004
0.0006
0.0008
0.001
0.0012
0.0014
0.0016
0.0018
Time (s)

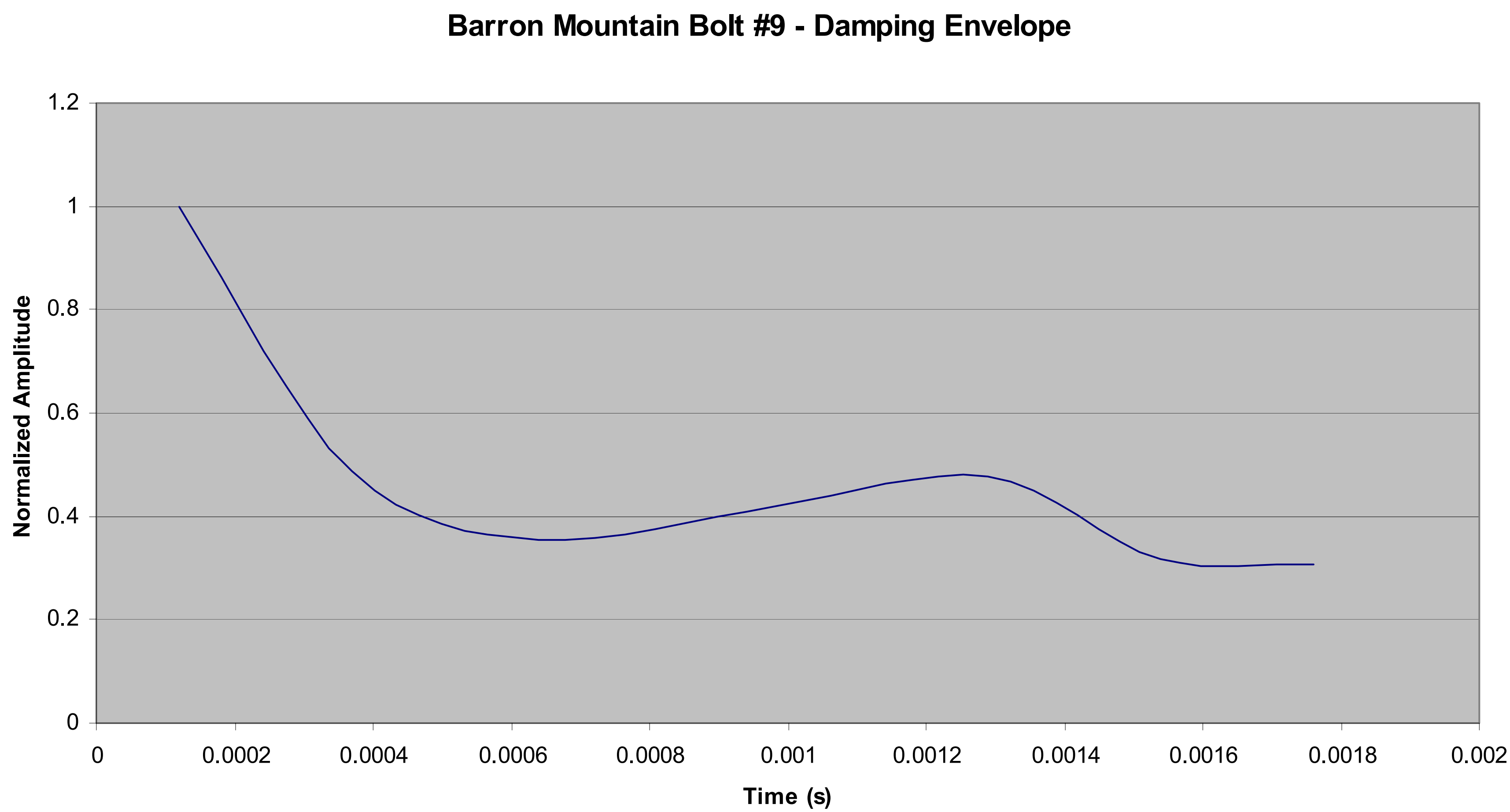

Barron Mountain Bolt #5 - Damping Envelope
Normalized Amplitude
1.2
1
0.8
0.6
0.4
0.2
0
-0.2
0
0.0002
0.0004
0.0006
0.0008
0.001
0.0012
0.0014
0.0016
0.0018
0.002
Time (s)

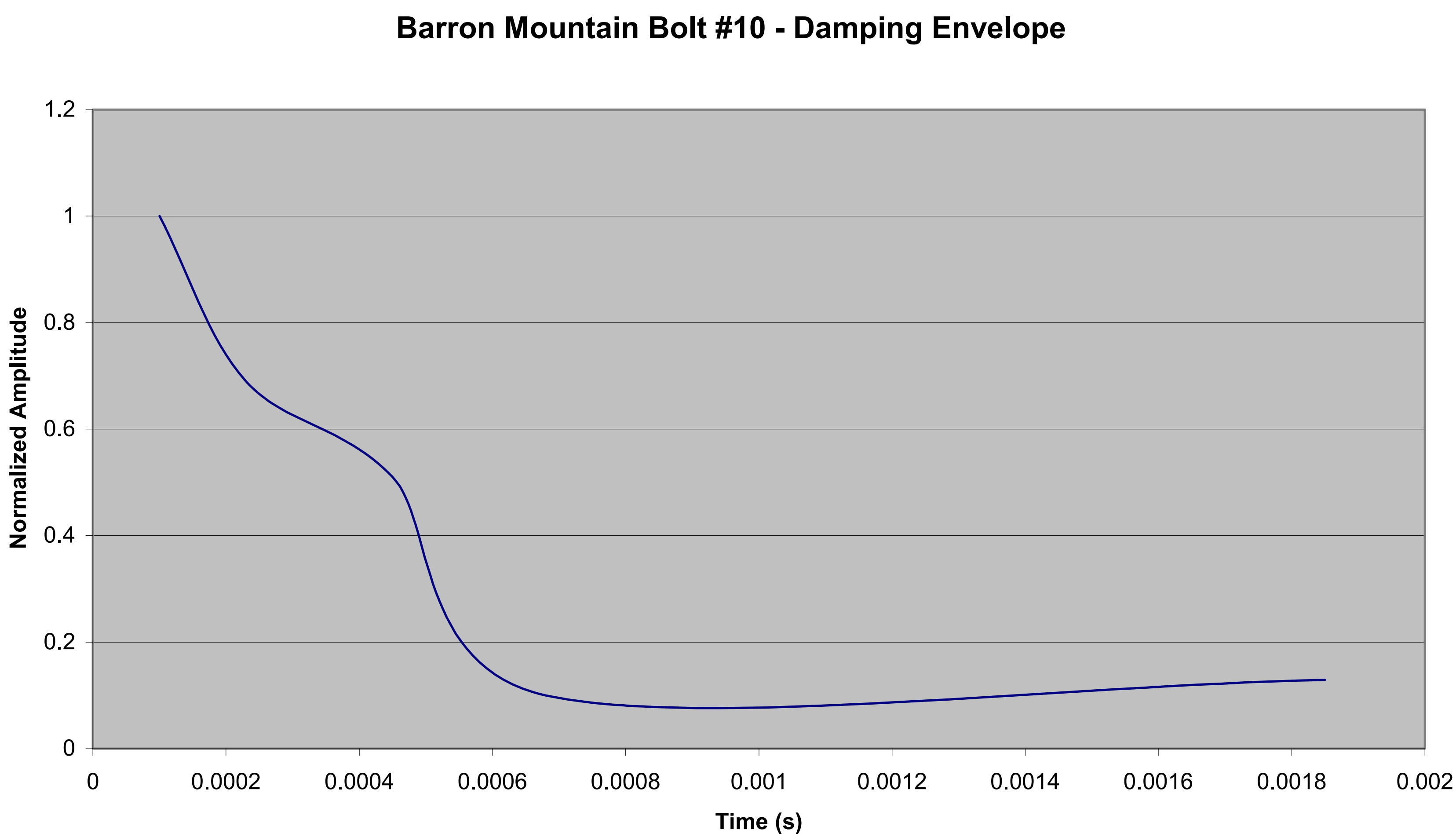

Barron Mountain Bolt # 6 - Damping Envelope
Normalized Amplitude
0
0.2
0.4
0.6
0.8
1
1.2
0
0.0002
0.0004
0.0006
0.0008
0.001
0.0012
0.0014
0.0016
0.0018
0.002
Time (s)



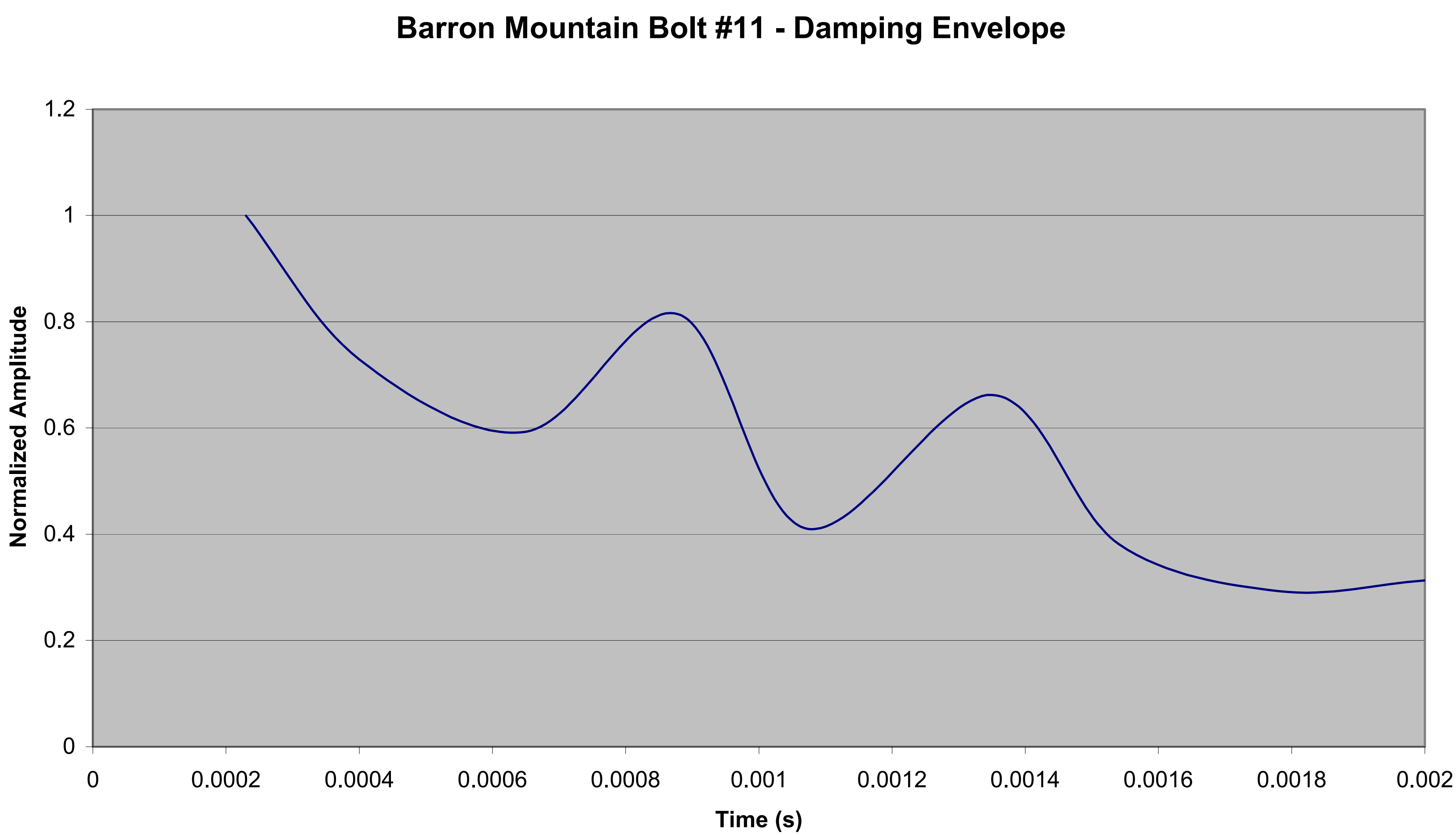
Barron Mountain Bolt #7 - Damping Envelope
Normalized Amplitude
1.2
1
0.8
0.6
0.4
0.2
0
0
0.0002
0.0004
0.0006
0.0008
0.001
0.0012
0.0014
0.0016
0.0018
0.002
Time (s)


Barron Mountain Bolt #8 - Damping Envelope
Normalized Amplitude
1.2
1
0.8
0.6
0.4
0.2
0
0
0.0002
0.0004
0.0006
0.0008
0.001
0.0012
0.0014
0.0016
0.0018
Time (s)

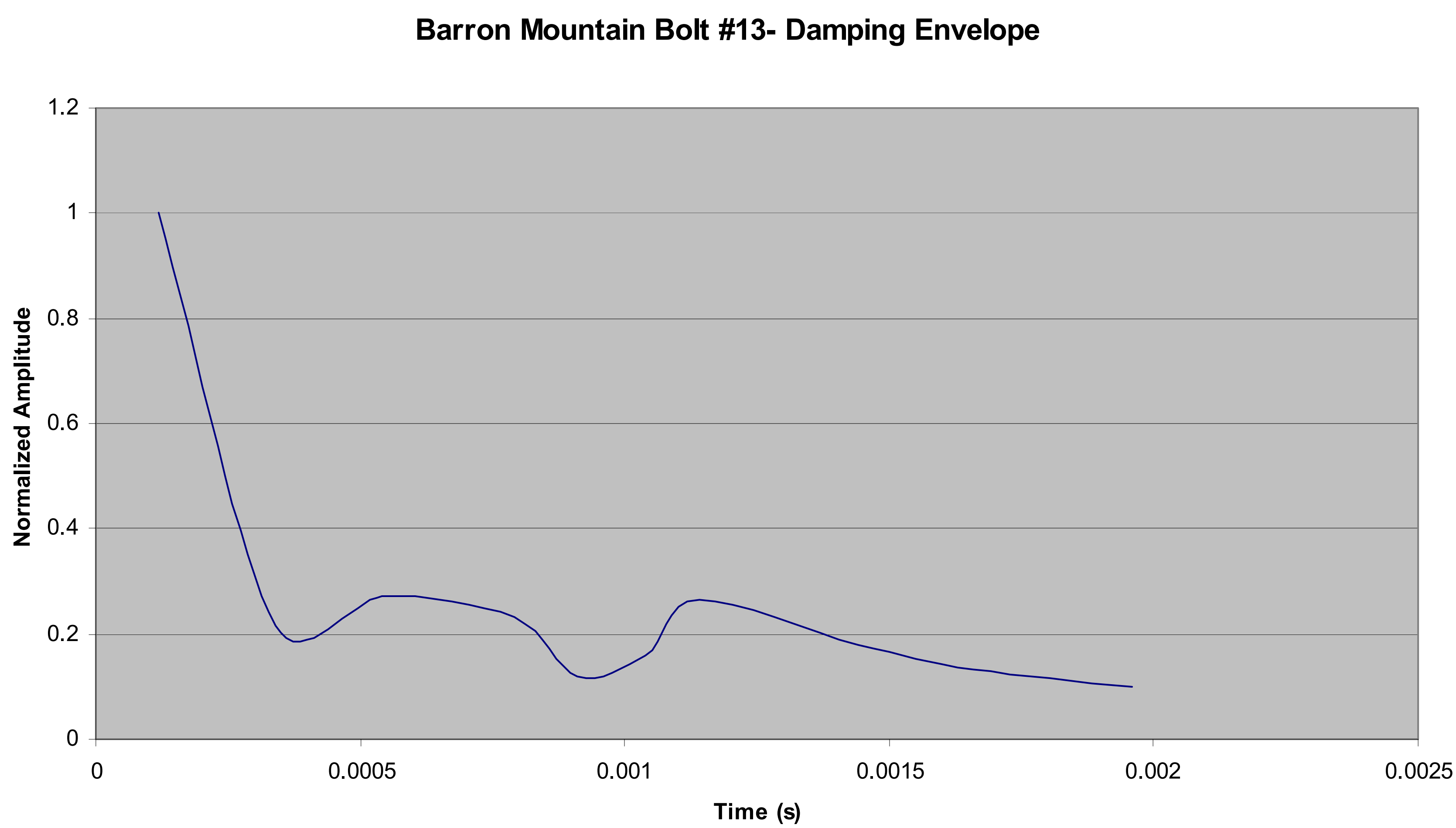

Barron Mountain Bolt #9 - Damping Envelope
Normalized Amplitude
Time (s)
0
0.2
0.4
0.6
0.8
1
1.2
0
0.0002
0.0004
0.0006
0.0008
0.001
0.0012
0.0014
0.0016
0.0018
0.002

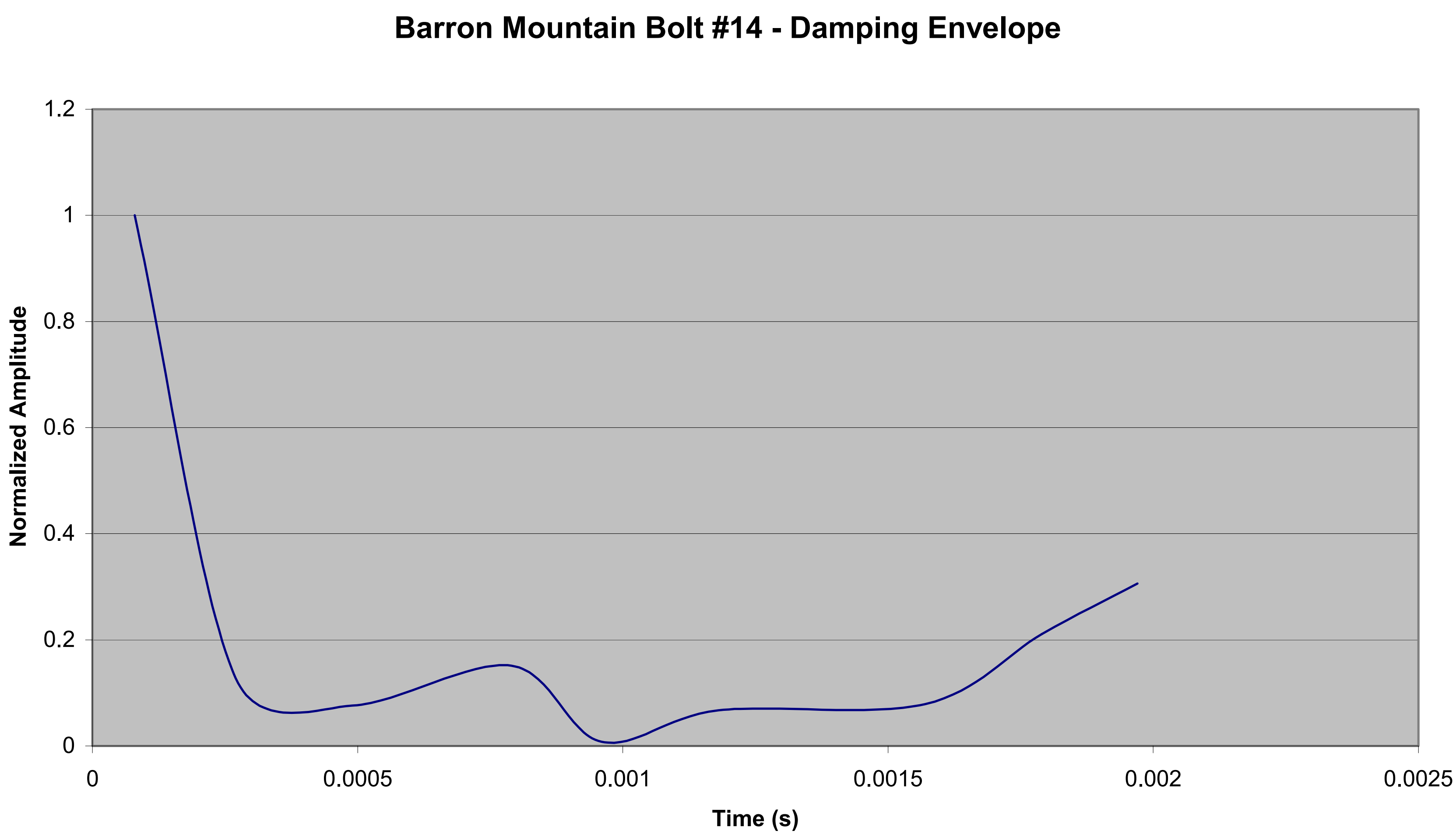


Barron Mountain Bolt #10 - Damping Envelope
Normalized Amplitude
1.2
1
0.8
0.6
0.4
0.2
0
0.0002
0.0004
0.0006
0.0008
0.001
0.0012
0.0014
0.0016
0.0018
0.002
Time (s)

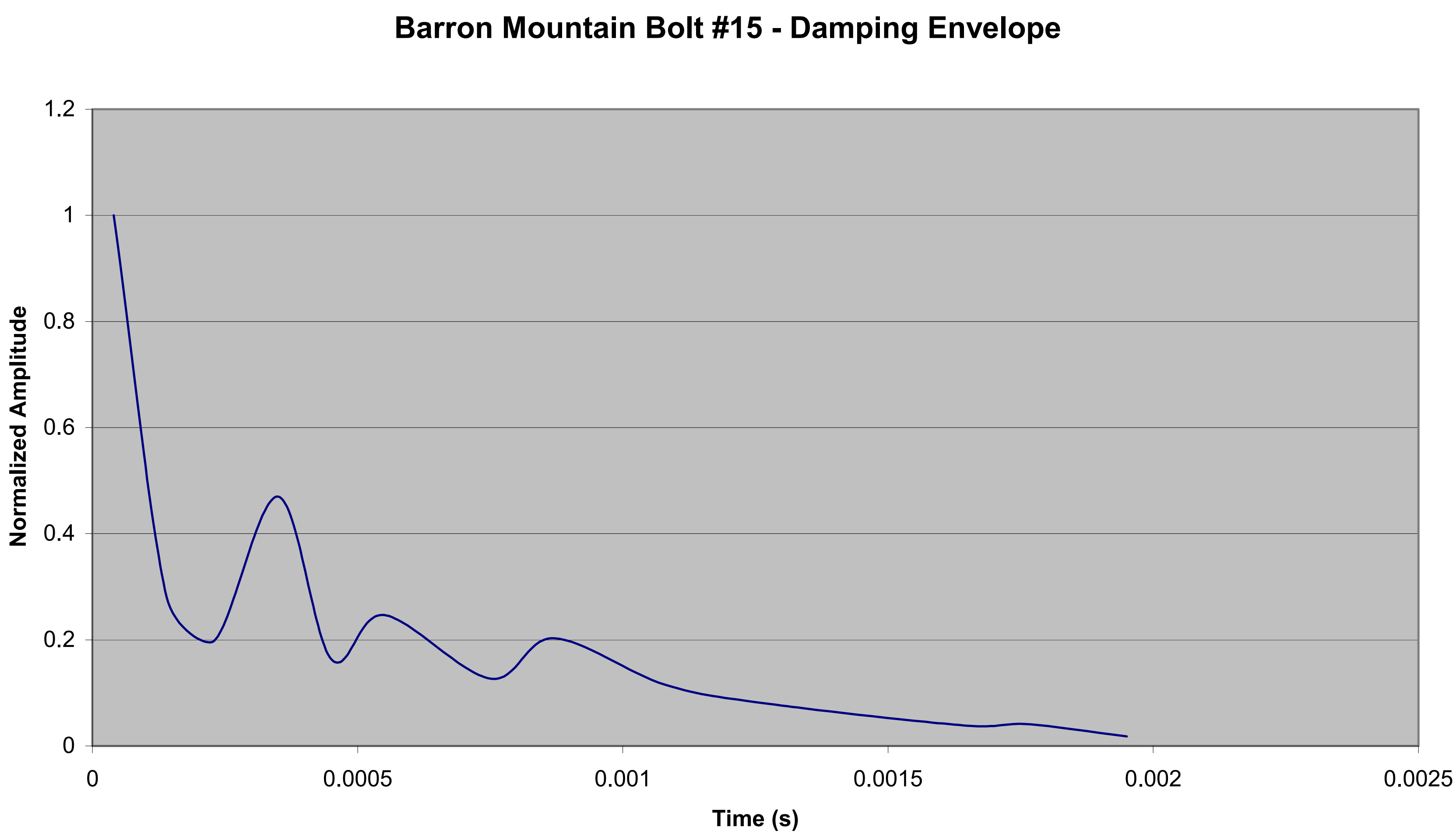


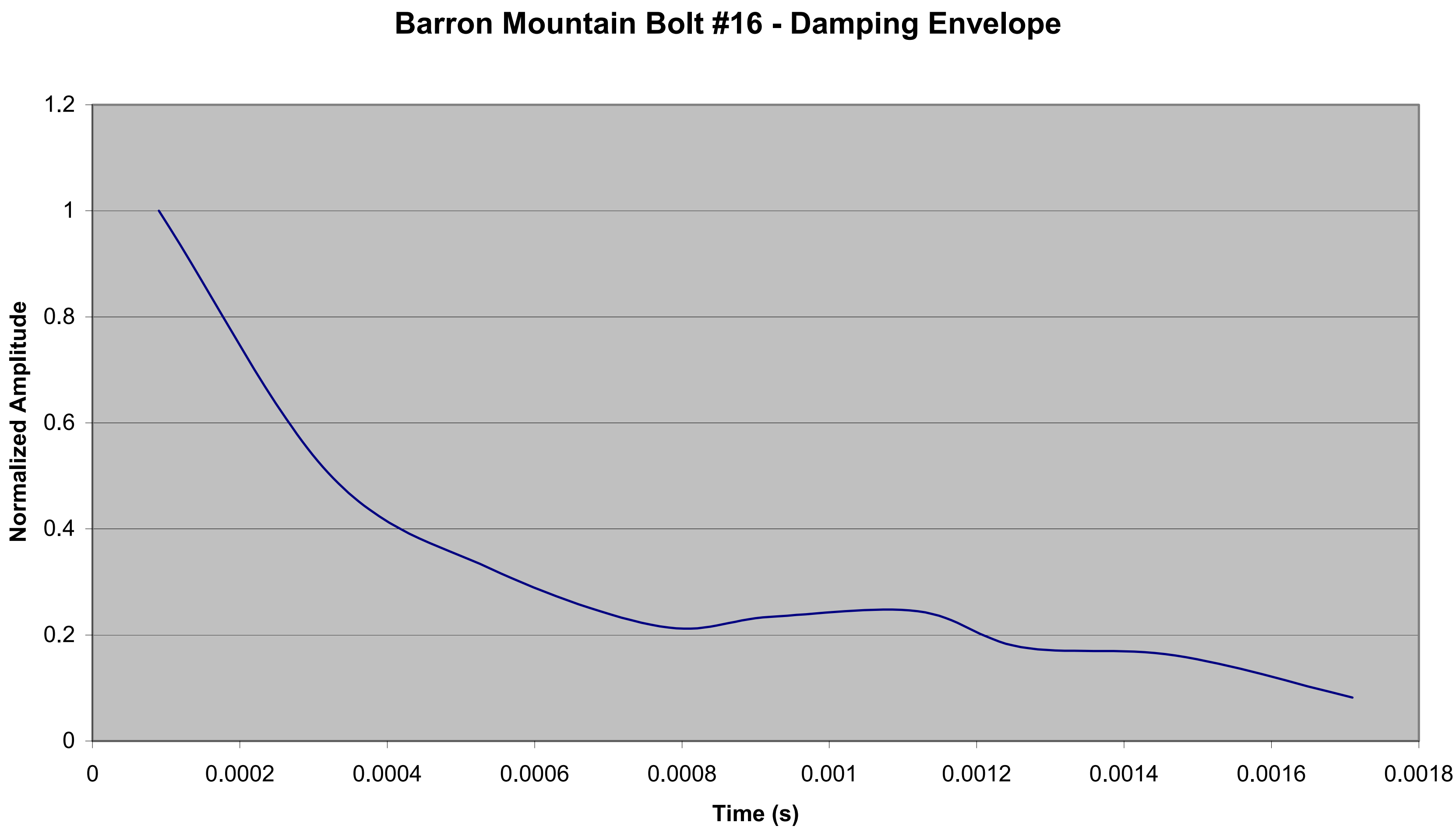
Barron Mountain Bolt #11 - Damping Envelope
Normalized Amplitude
0
0.2
0.4
0.6
0.8
1
1.2
0
0.0002
0.0004
0.0006
0.0008
0.001
0.0012
0.0014
0.0016
0.0018
0.002
Time (s)

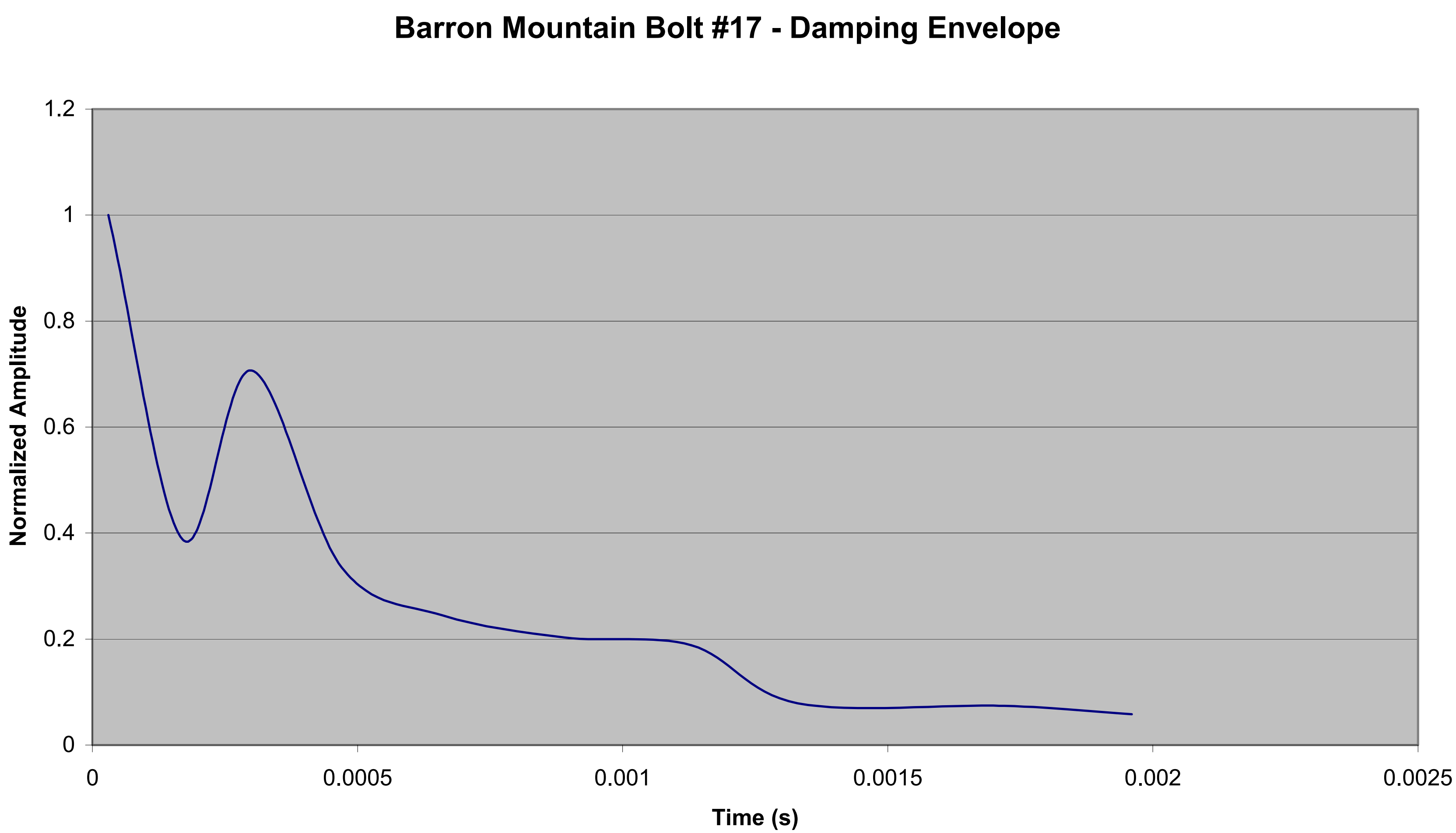

Barron Mountain Bolt #13- Damping Envelope
Normalized Amplitude
1.2
1
0.8
0.6
0.4
0.2
0
0
0.0005
0.001
0.0015
0.002
0.0025
Time (s)



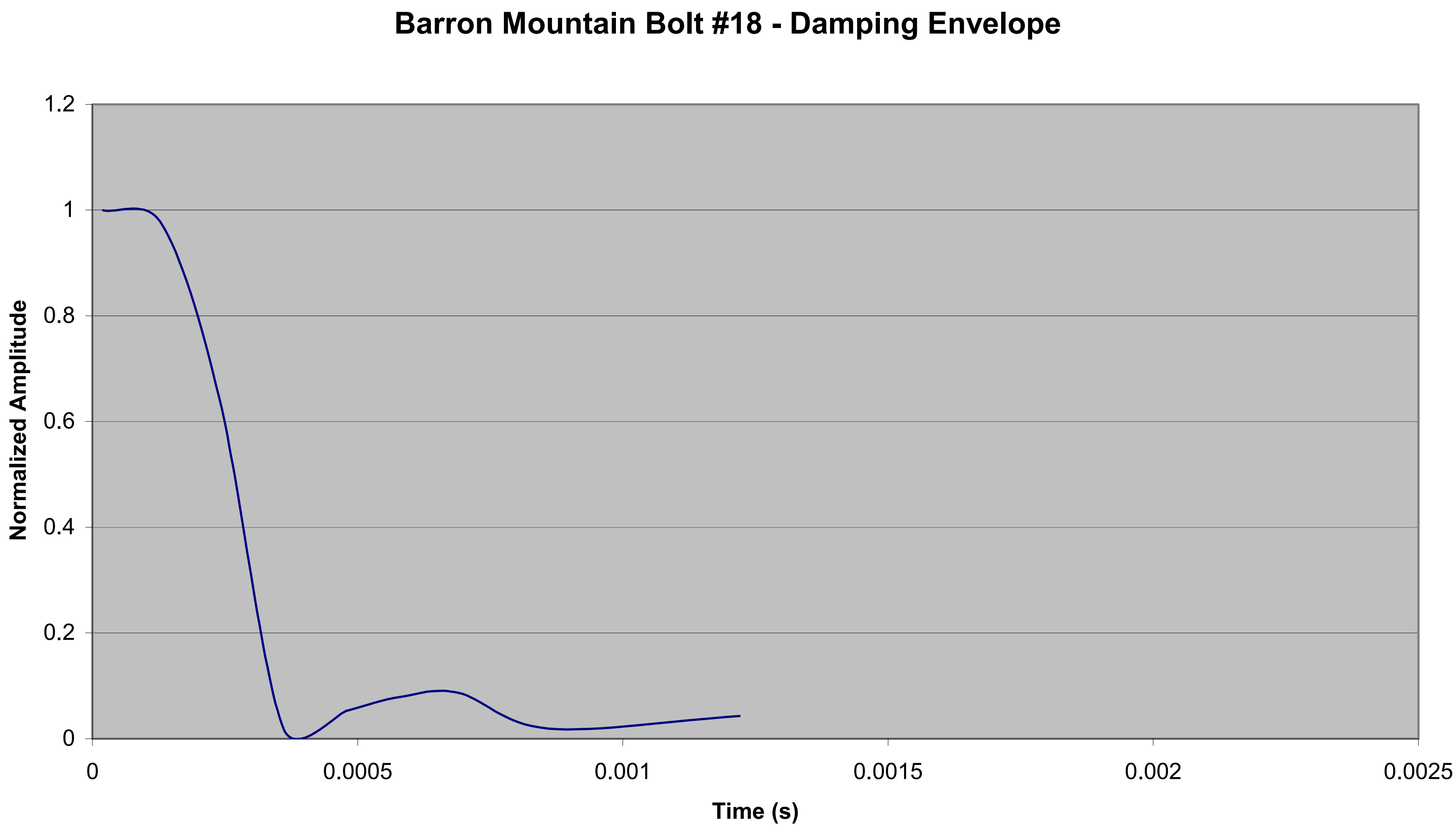
Barron Mountain Bolt #14 - Damping Envelope
Normalized Amplitude
1.2
1
0.8
0.6
0.4
0.2
0
0
0.0005
0.001
0.0015
0.002
0.0025
Time (s)

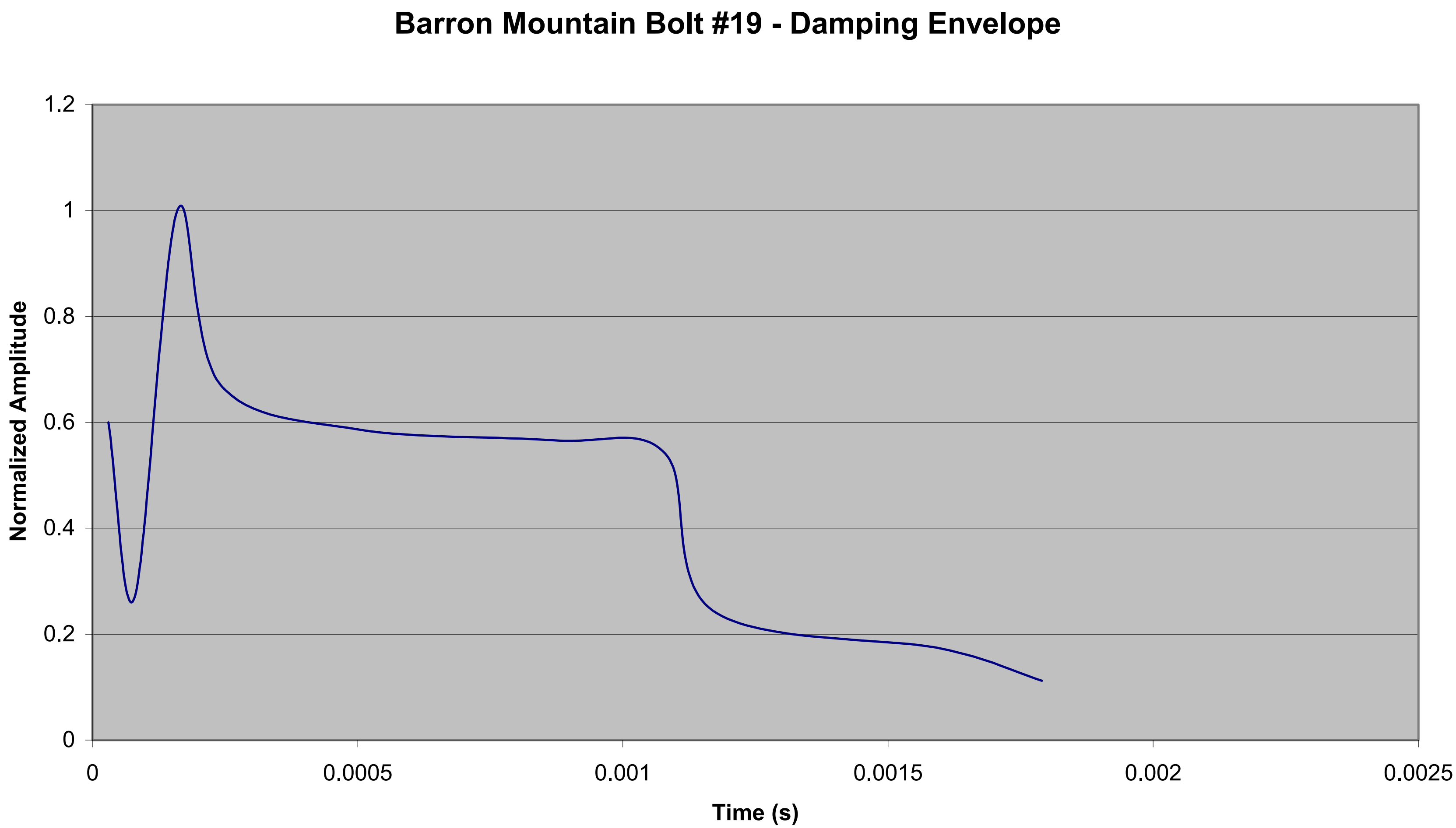

Barron Mountain Bolt #15 - Damping Envelope
Normalized Amplitude
1.2
1
0.8
0.6
0.4
0.2
0
0
0.0005
0.001
0.0015
0.002
0.0025
Time (s)

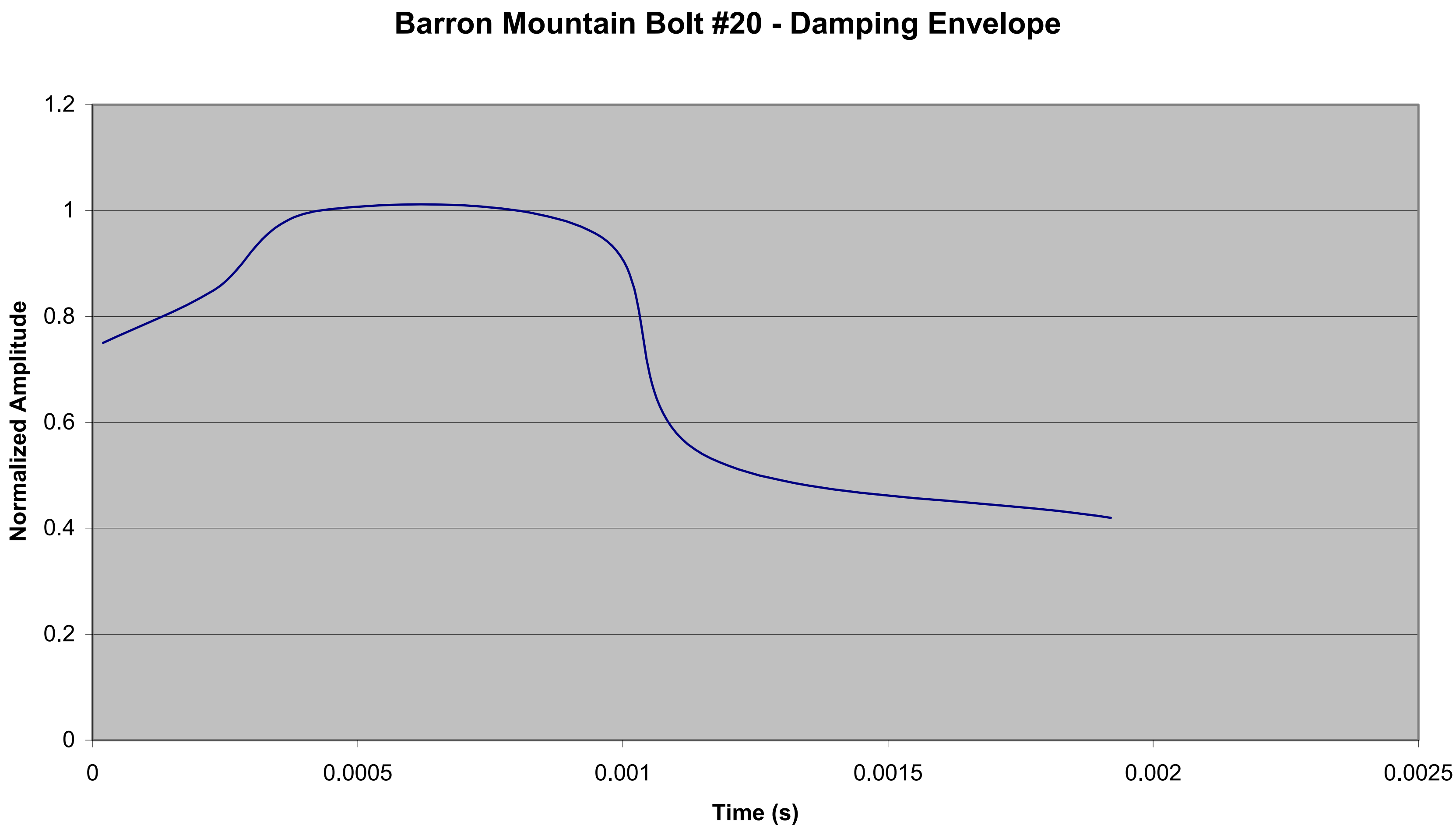

Barron Mountain Bolt #16 - Damping Envelope
Normalized Amplitude
1.2
1
0.8
0.6
0.4
0.2
0
0
0.0002
0.0004
0.0006
0.0008
0.001
0.0012
0.0014
0.0016
0.0018
Time (s)

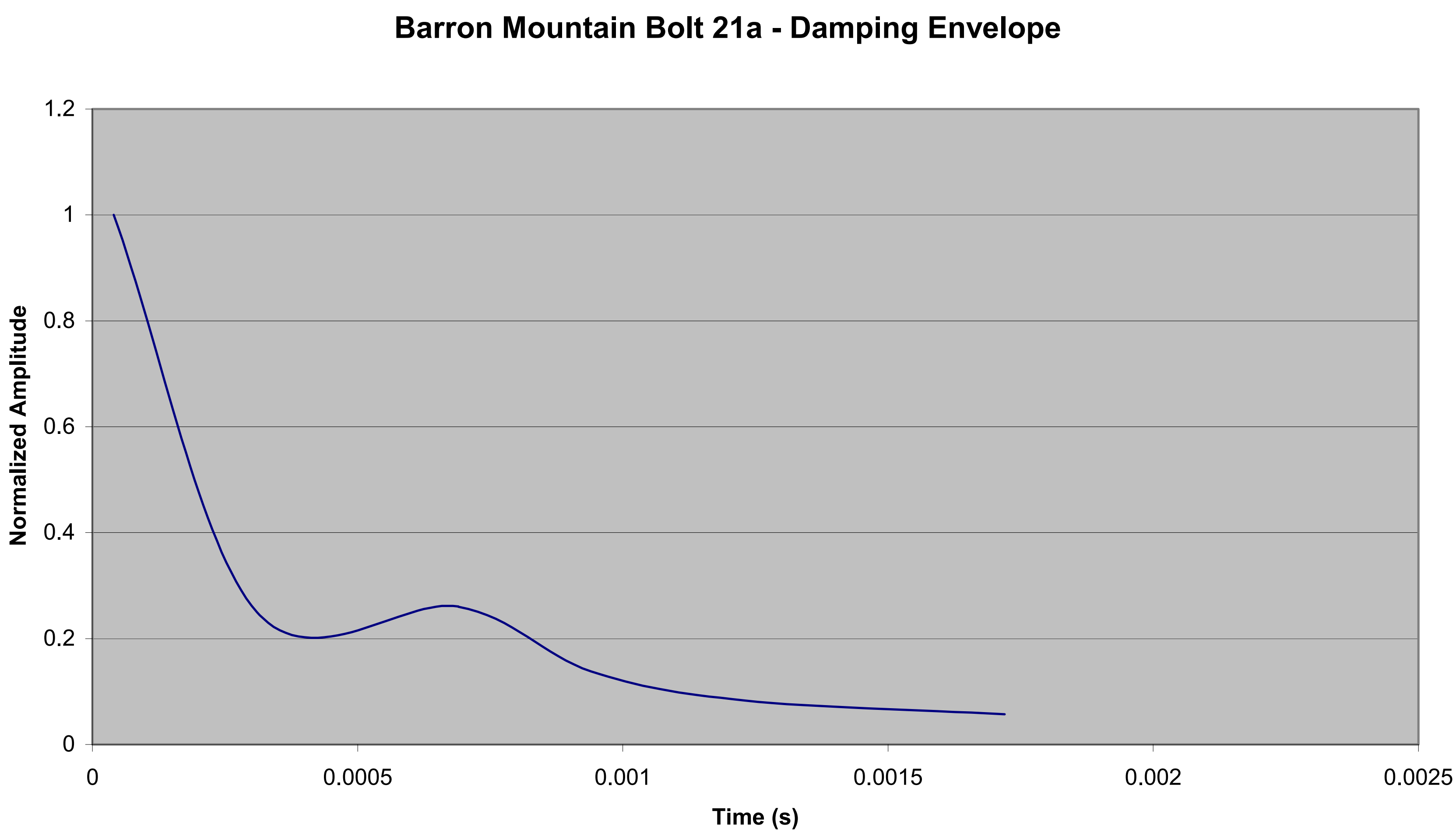


Barron Mountain Bolt #17 - Damping Envelope
Normalized Amplitude
Time (s)
0
0.2
0.4
0.6
0.8
1
1.2
0
0.0005
0.001
0.0015
0.002
0.0025


Barron Mountain Bolt #18 - Damping Envelope
Normalized Amplitude
1.2
1
0.8
0.6
0.4
0.2
0
0
0.0005
0.001
0.0015
0.002
0.0025
Time (s)


Barron Mountain Bolt #19 - Damping Envelope
Normalized Amplitude
1.2
1
0.8
0.6
0.4
0.2
0
0
0.0005
0.001
0.0015
0.002
0.0025
Time (s)


Barron Mountain Bolt #20 - Damping Envelope
Normalized Amplitude
1.2
1
0.8
0.6
0.4
0.2
0
0
0.0005
0.001
0.0015
0.002
0.0025
Time (s)

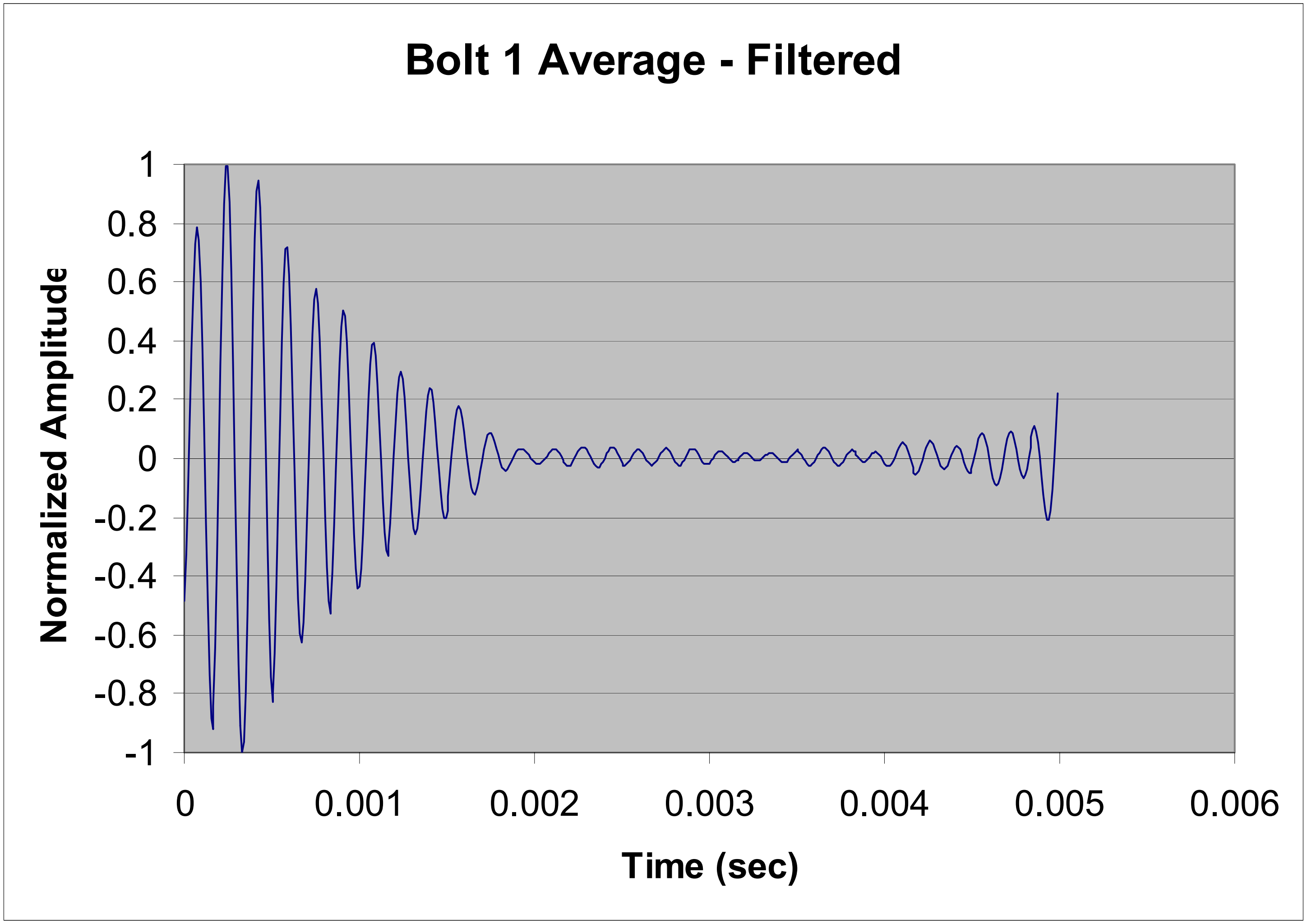

Barron Mountain Bolt 21a - Damping Envelope
1.2
1
0.8
0.6
0.4
0.2
0
Normalized Amplitude
0
0.0005
0.001
0.0015
0.002
0.0025
Time (s)

APPENDIX VII

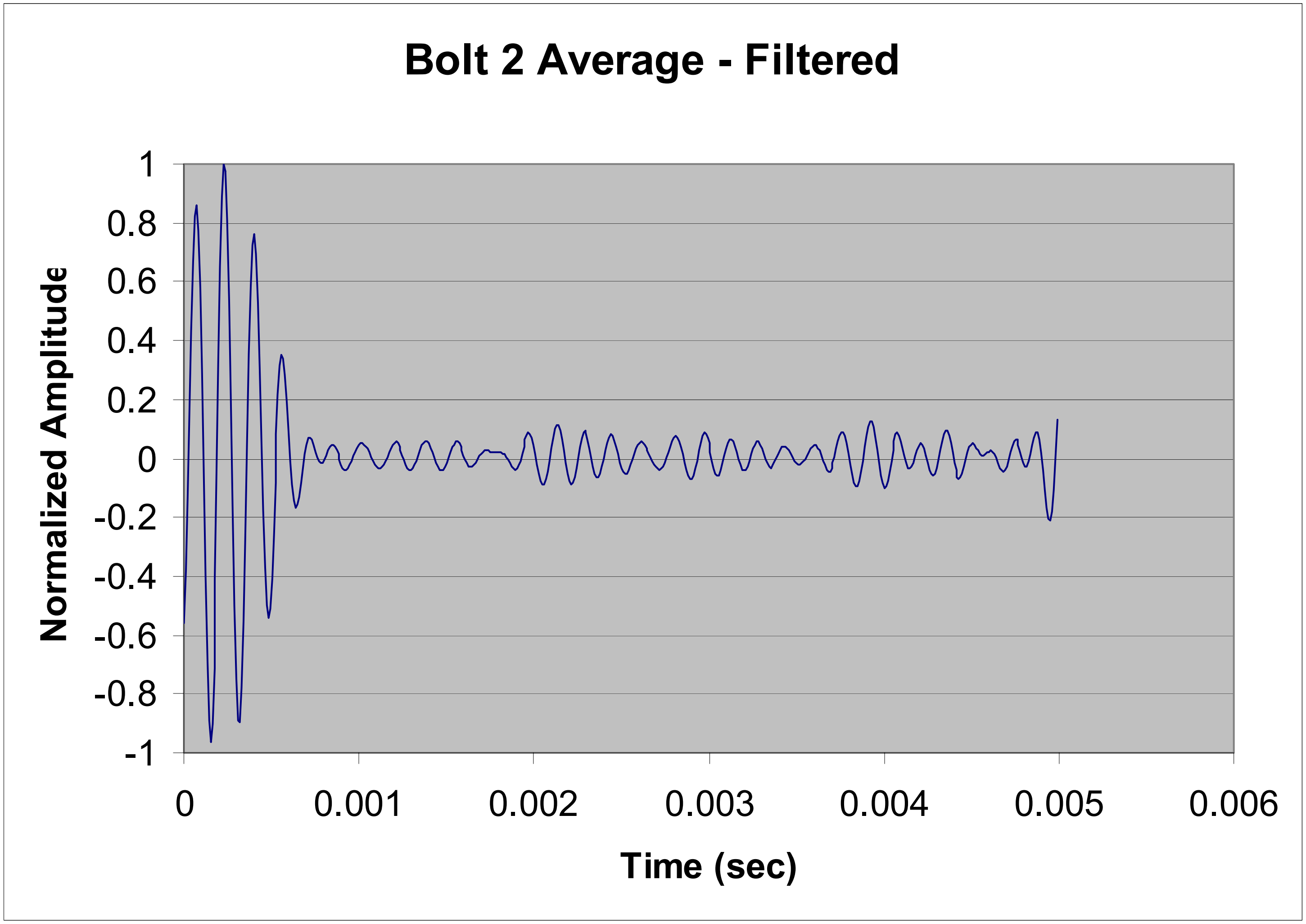
ROCK BOLTS
IMPACT ACCELERATION TIME HISTORIES

Table VII-1. Observed Reflections from Impact Test on Rock Bolts.

Test Bolt #	Relative Signal Attenuation	Observed L_1 (ft)	Observed L_2 (ft)	"Known" L_T (ft)
1	Strong	~	16	15
2	Strong	7	17	25
3	Strong	10	15	25
4	Strong	5	15	30
5	Weak	8	~	25
6	Weak	7	~	15
7	Weak	~	7	10
8	Weak	~	9	10
9	Weak	~	10	10
10	Strong	~	16	15
11	Strong	15	26	25
12	Weak	8	~	25
13	Strong	12	33	30
14	Weak	10	33	30
15	Weak	14	~	20
16	Weak	8	17	20
17	Weak	11	25	25
18	Weak	12	28	25
19	Strong	~	20	20
20	Weak	13	17	20
21	Weak	~	12	10
22	Weak	8	25	25



Bolt 1 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.001
0.002
0.003
0.004
0.005
0.006
Time (sec)



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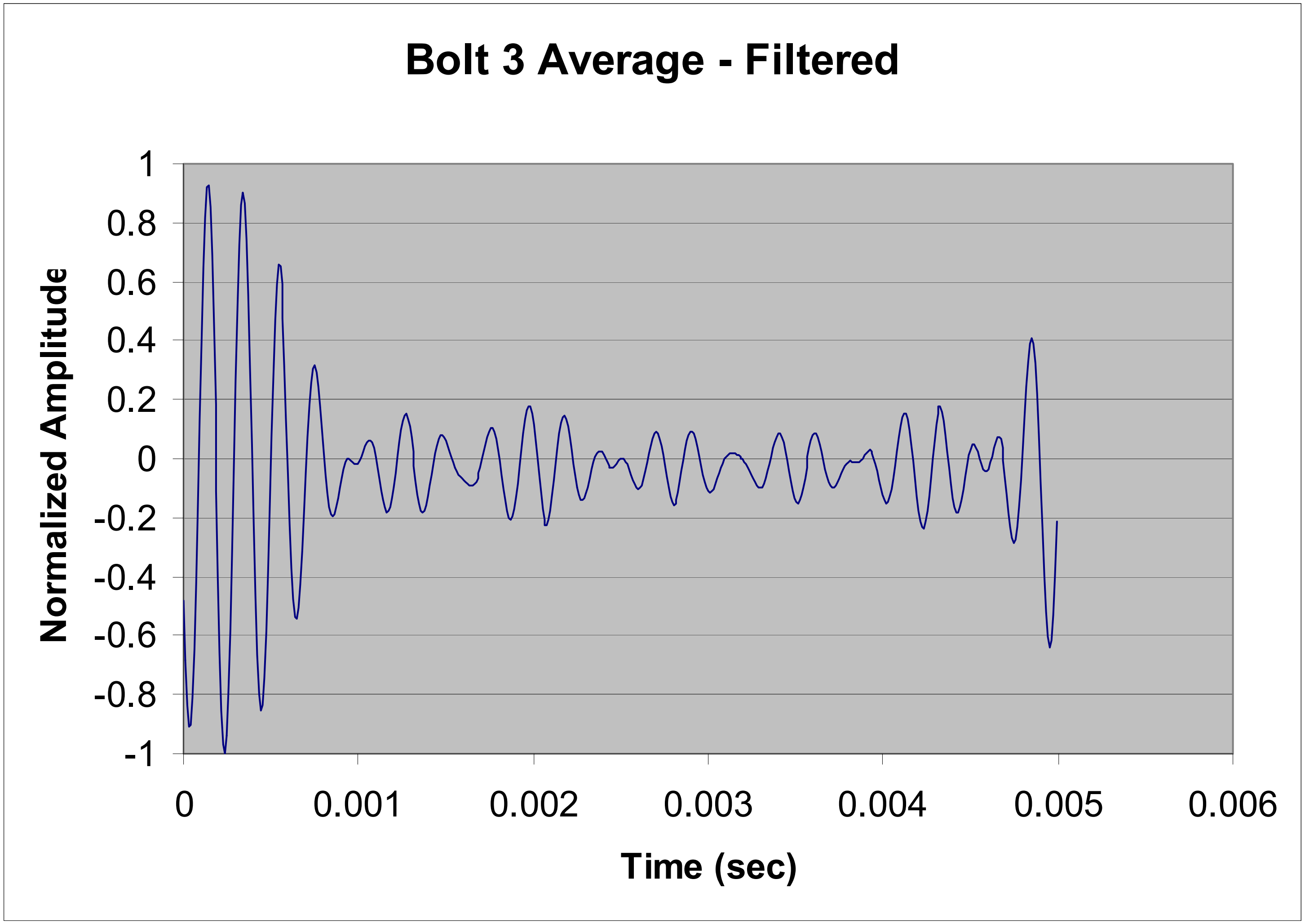
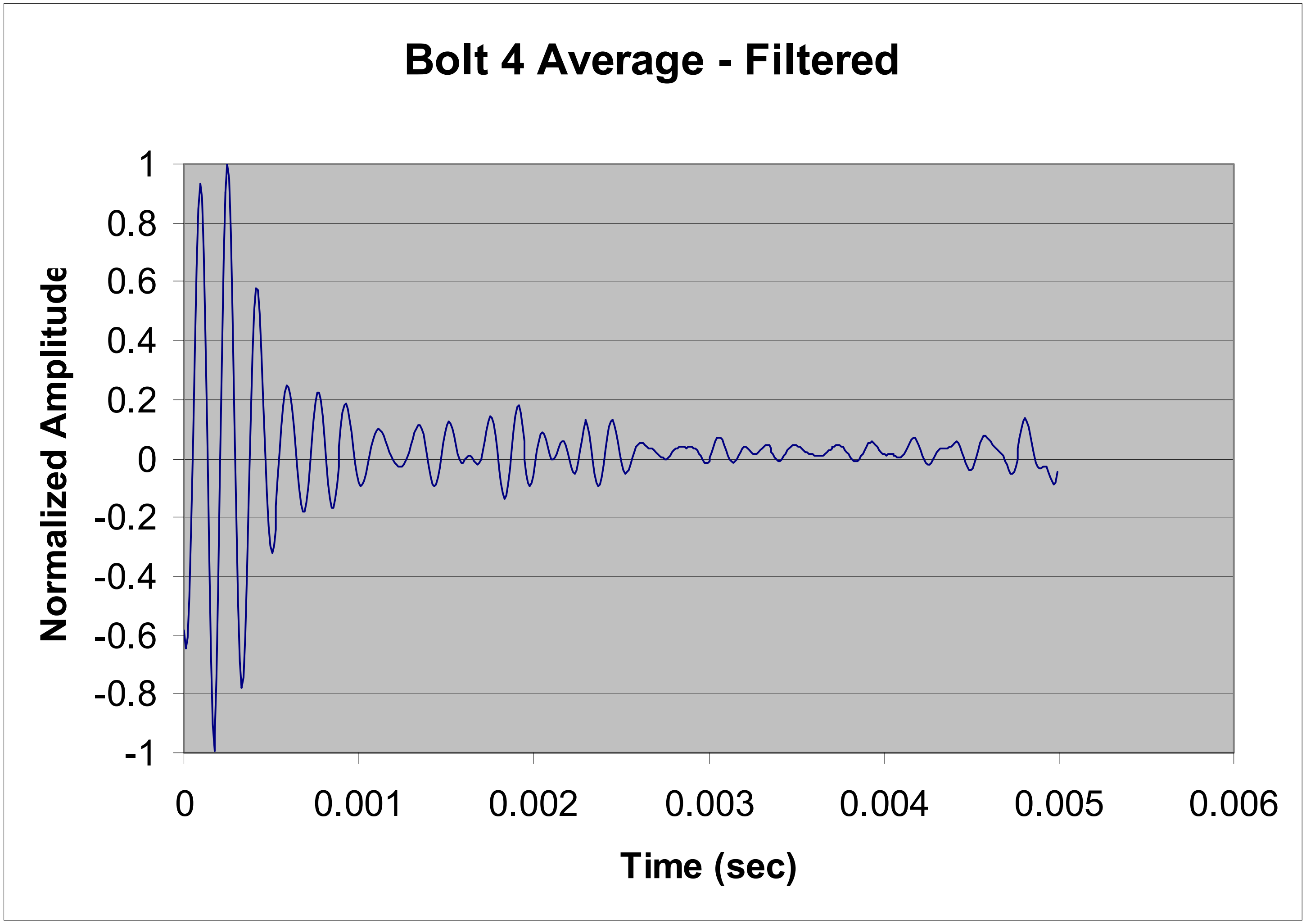
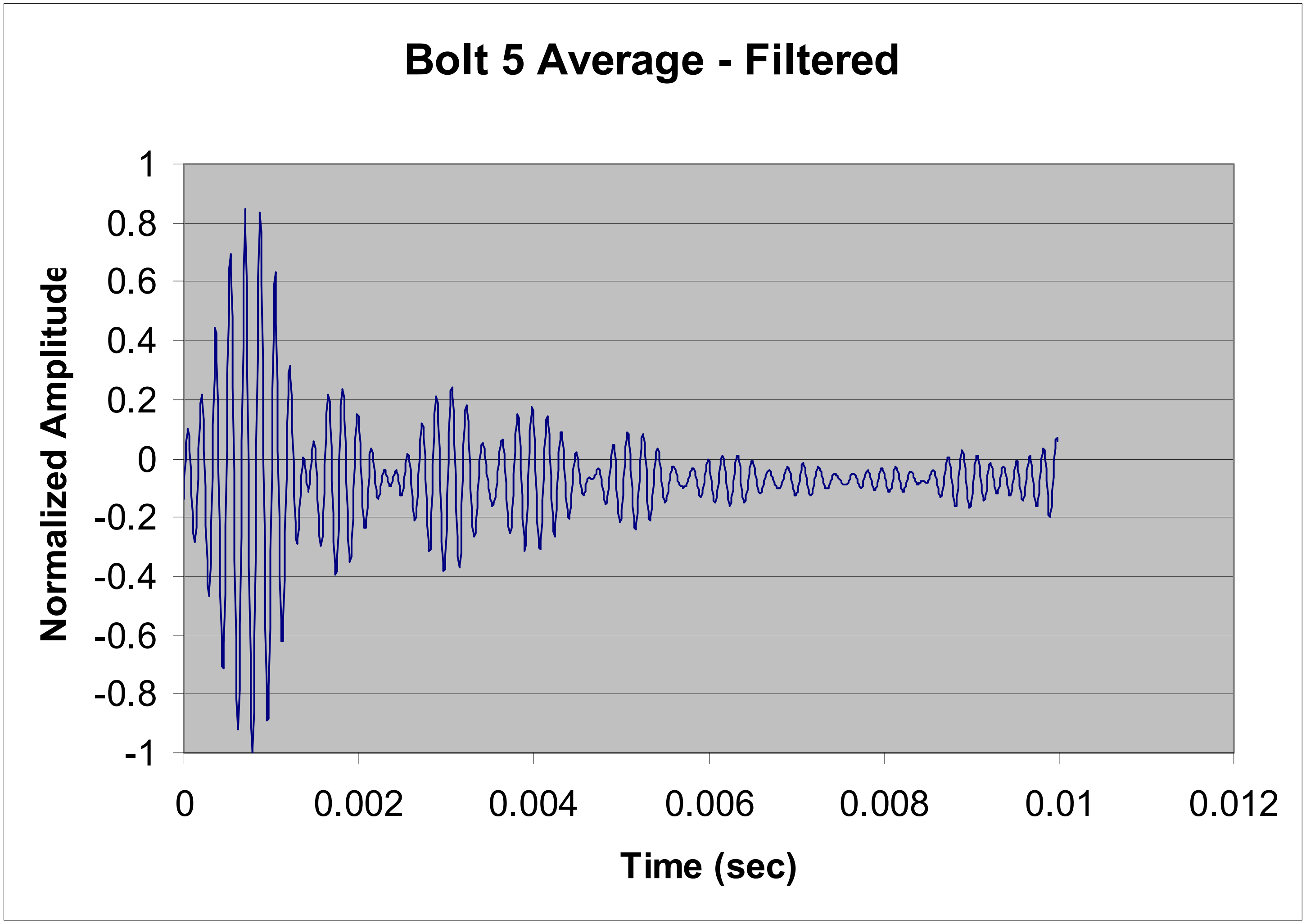
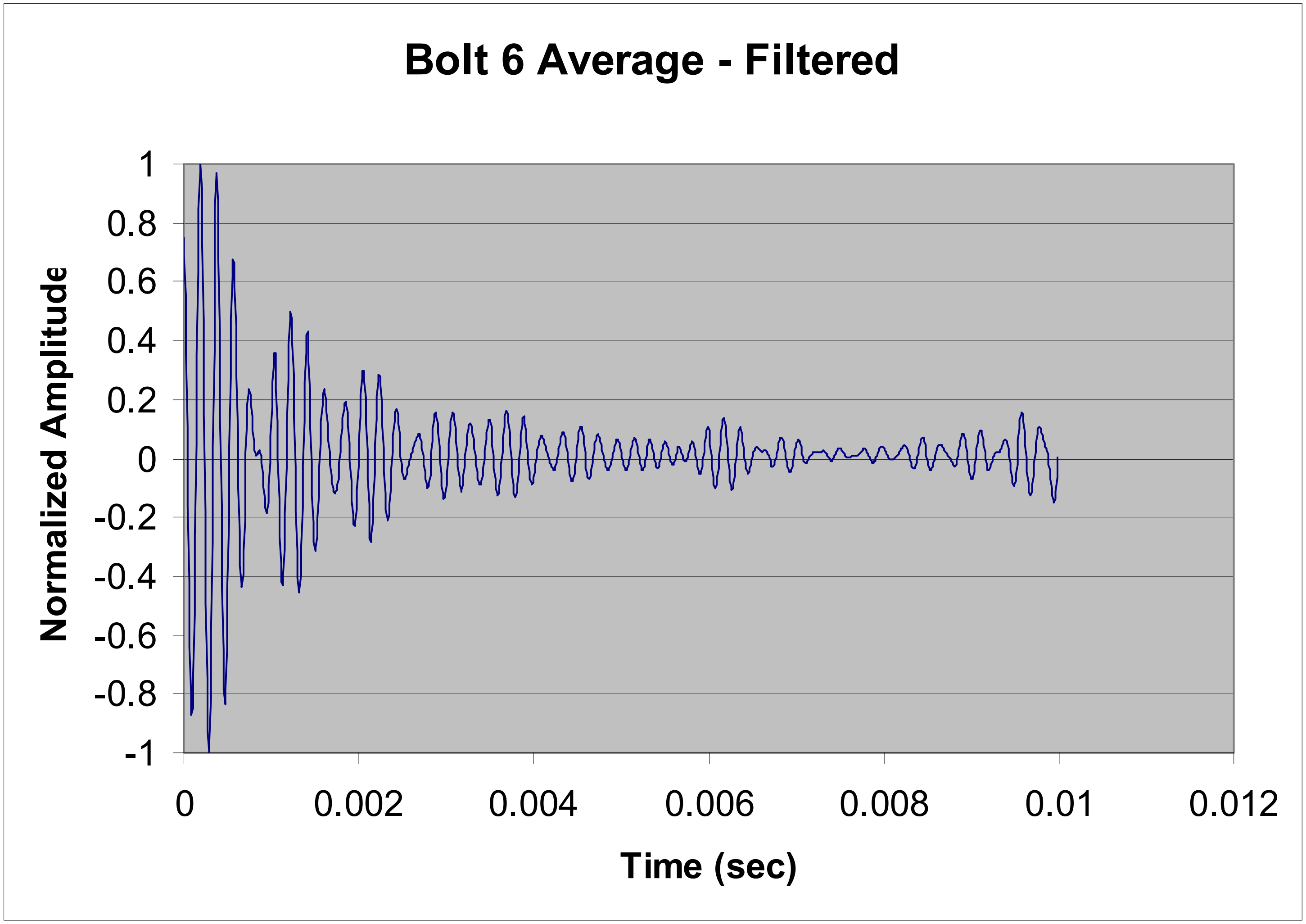

Bolt 3 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.001
0.002
0.003
0.004
0.005
0.006
Time (sec)

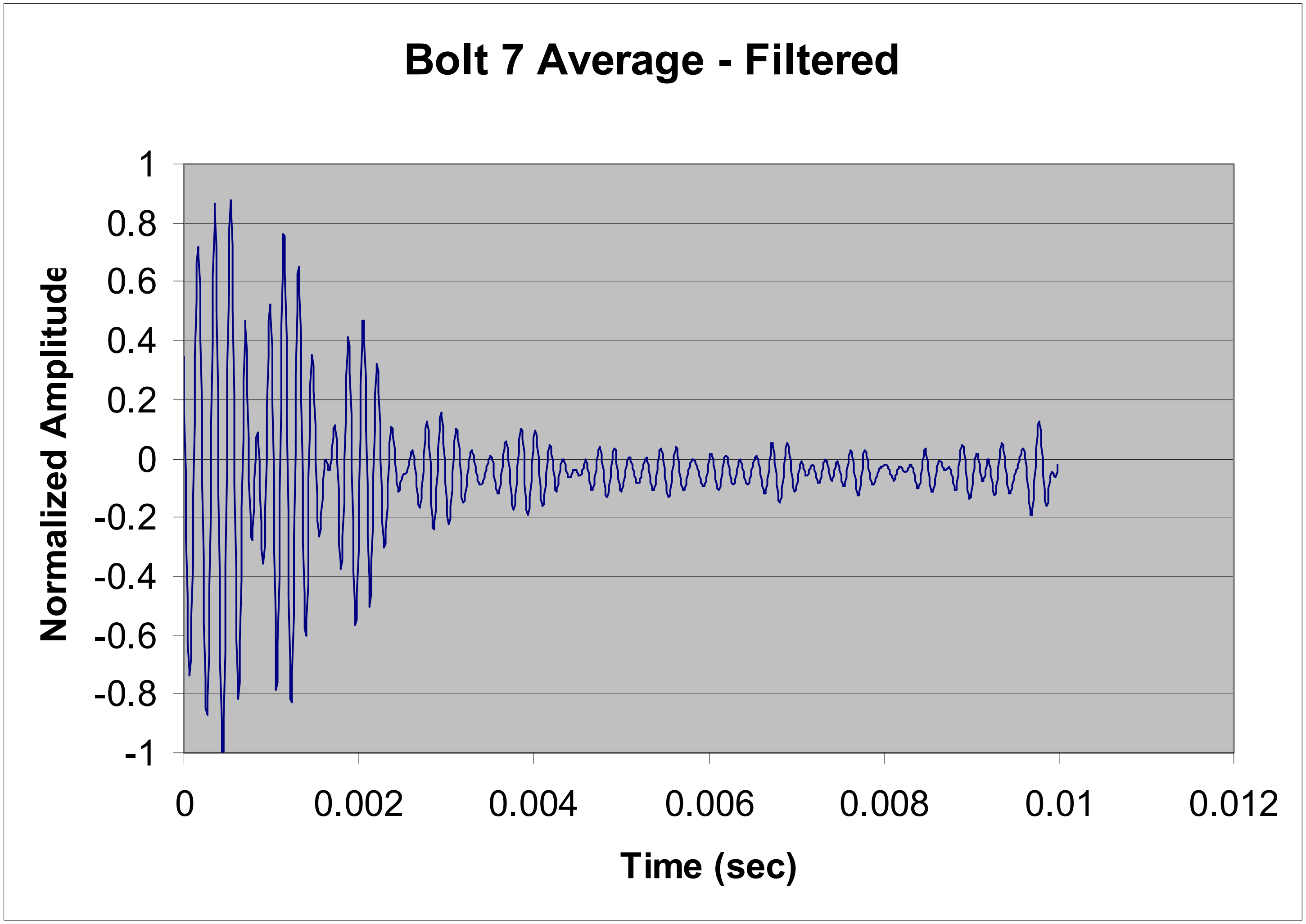


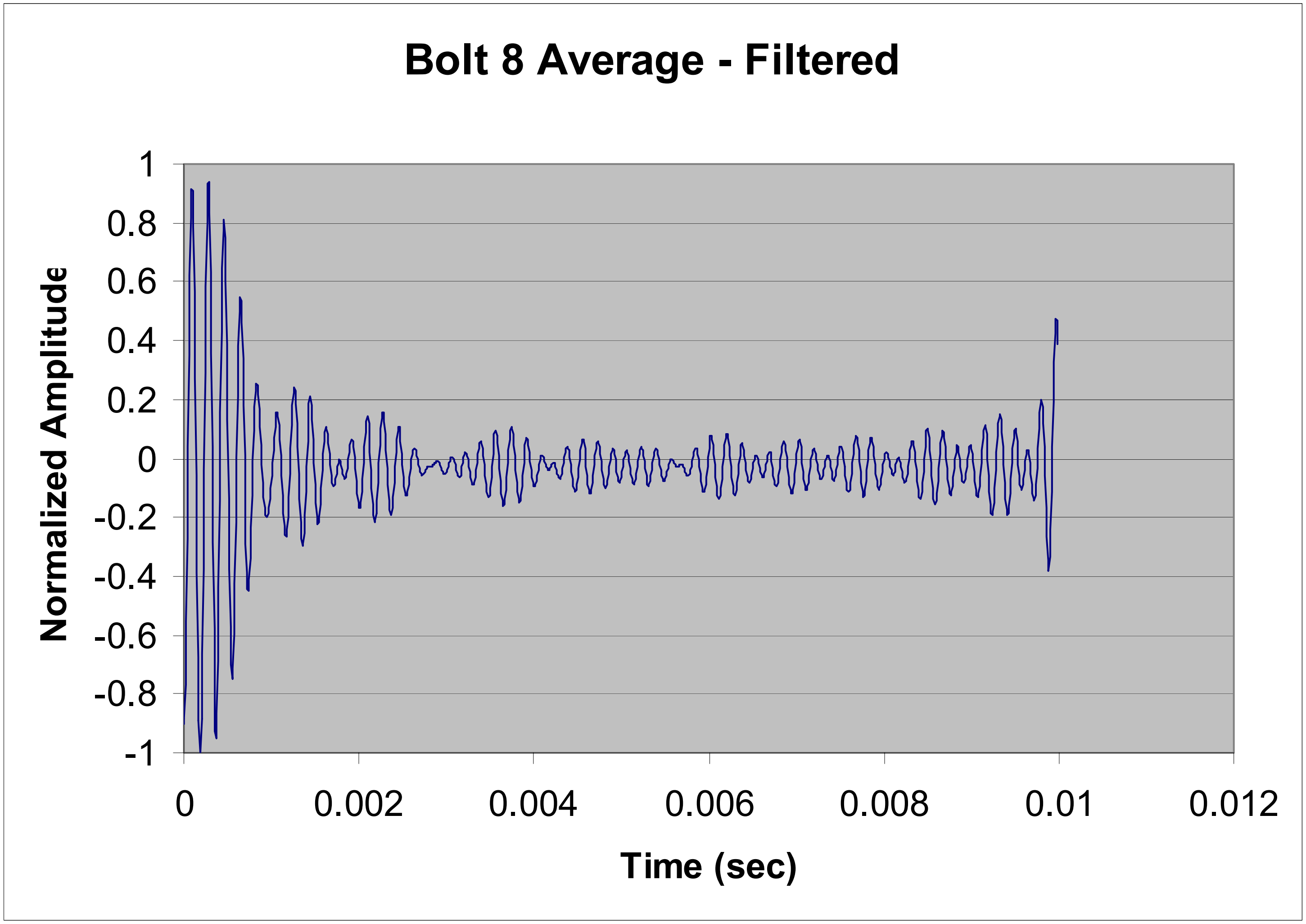
Bolt 4 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.001
0.002
0.003
0.004
0.005
0.006
Time (sec)

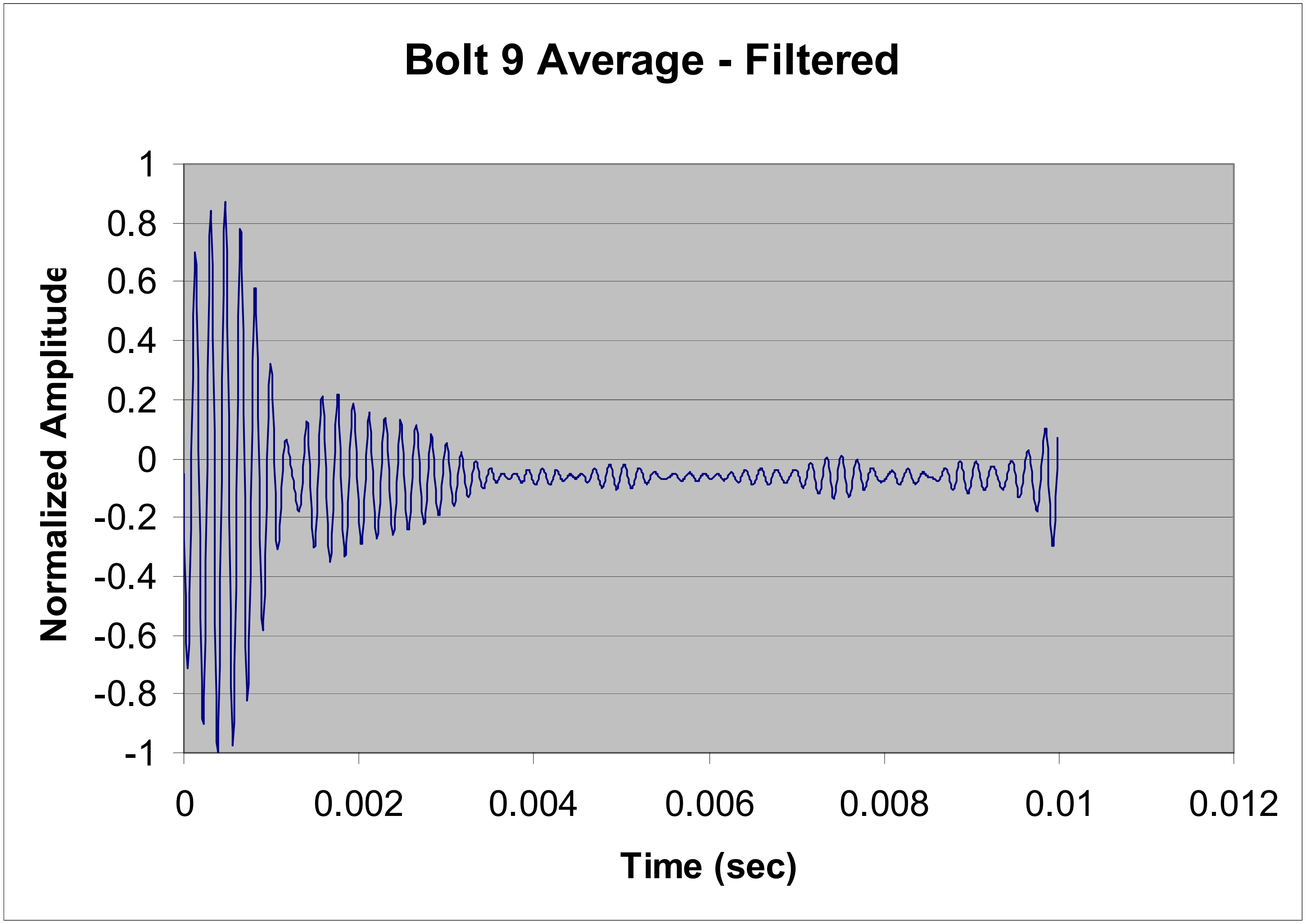

Bolt 5 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
0.012
Time (sec)

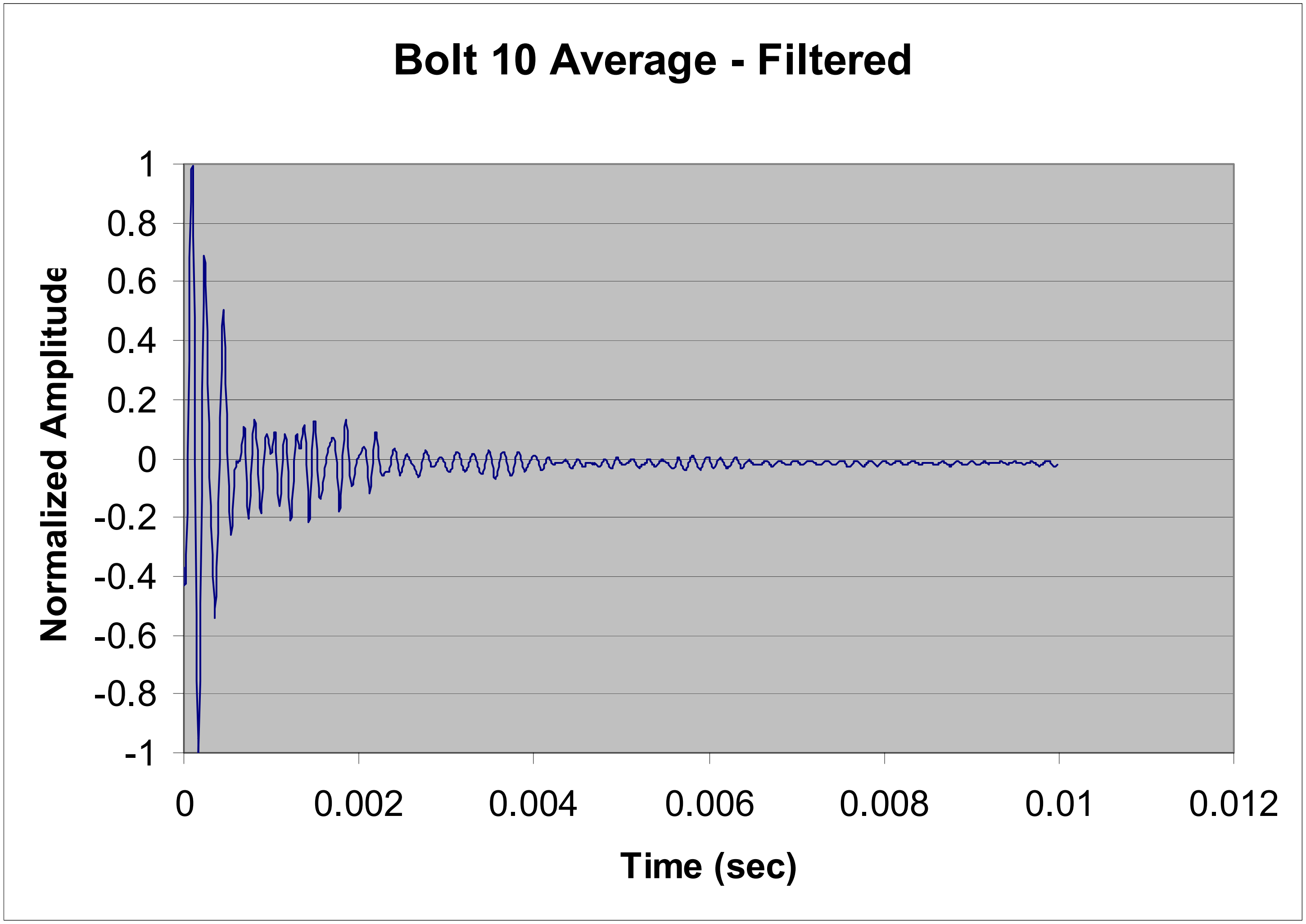


Bolt 6 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
0.012
Time (sec)

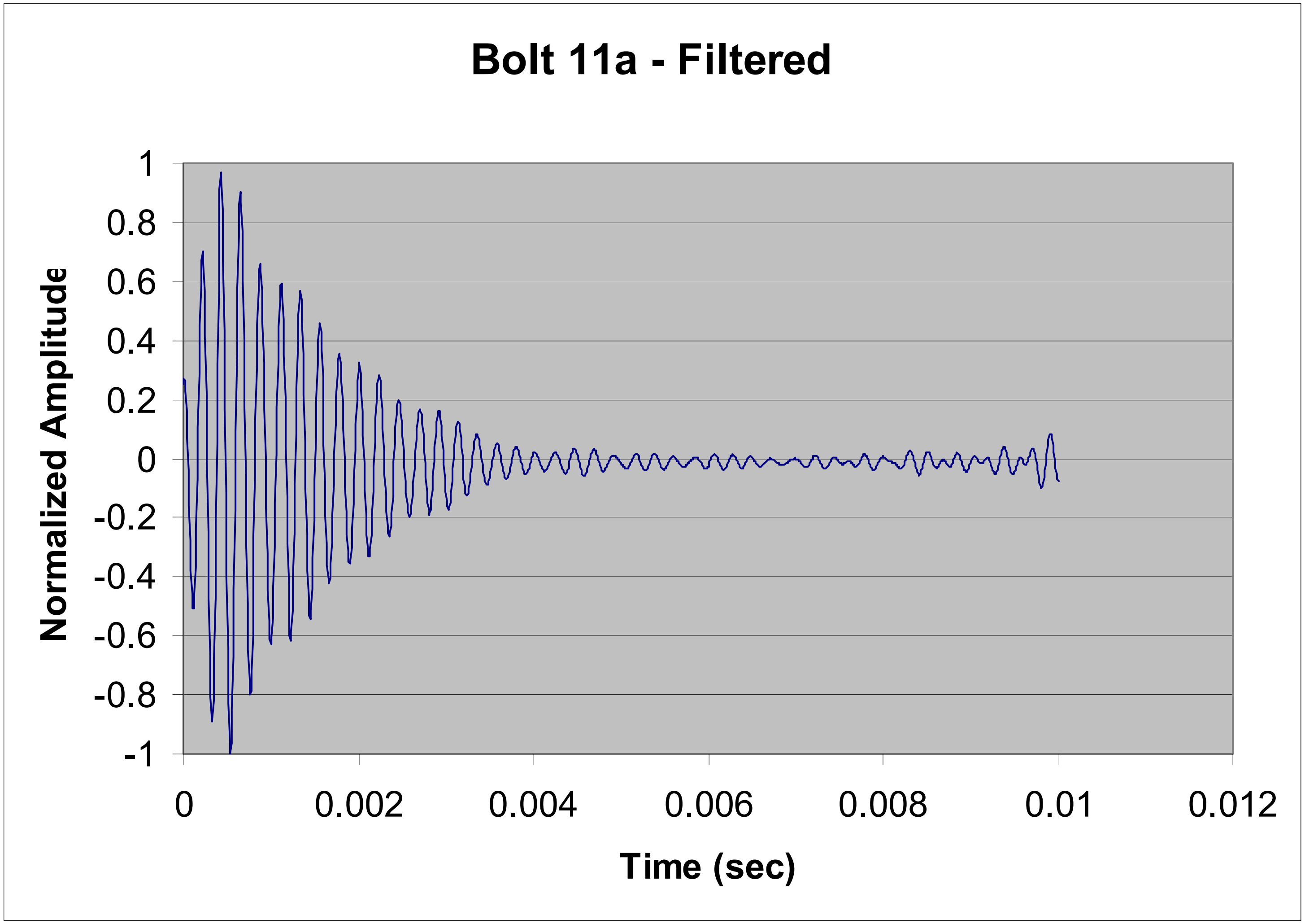

Bolt 7 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
0.012
Time (sec)


Bolt 8 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
0.012
Time (sec)


Bolt 9 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
0.012
Time (sec)

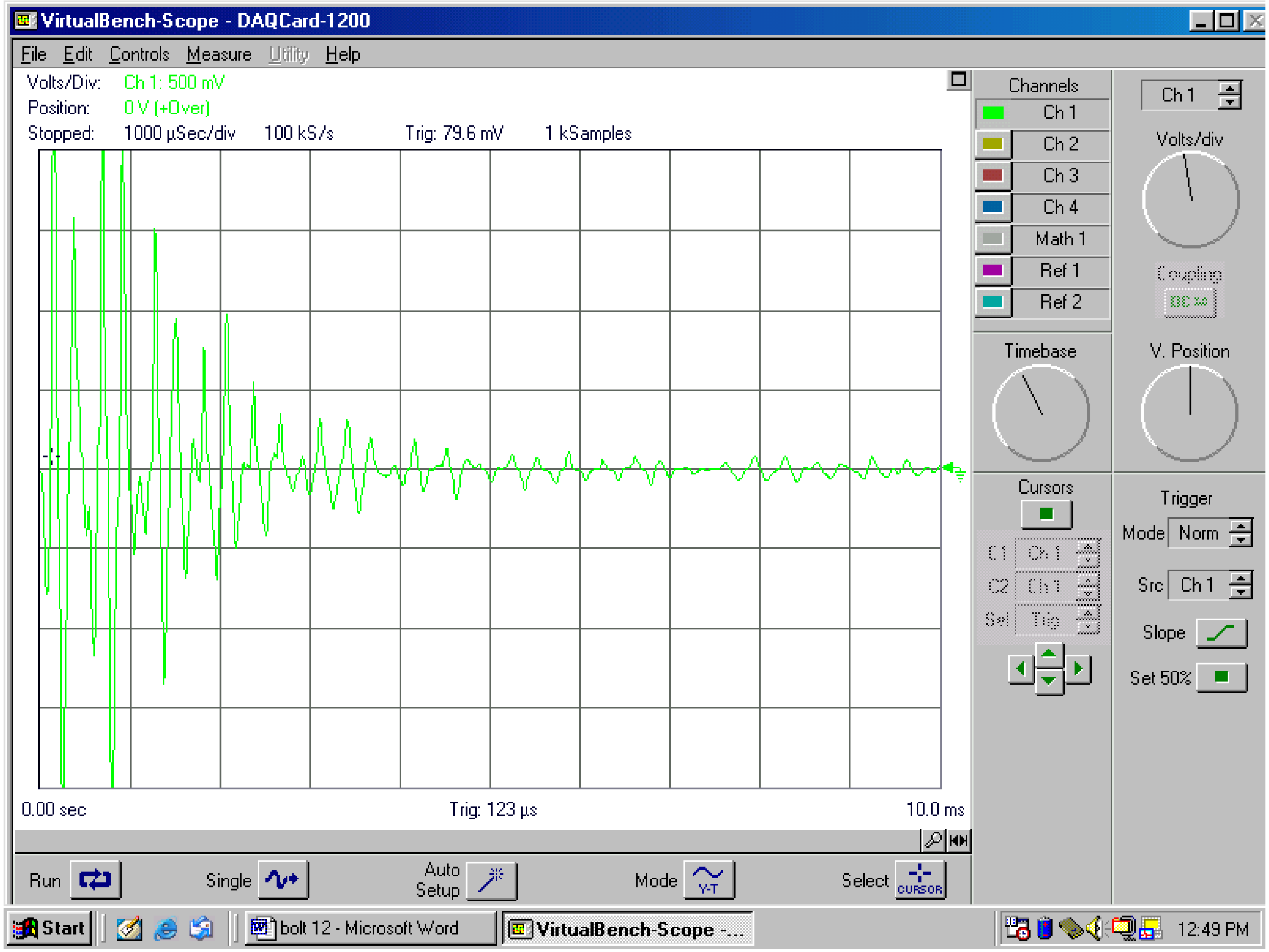


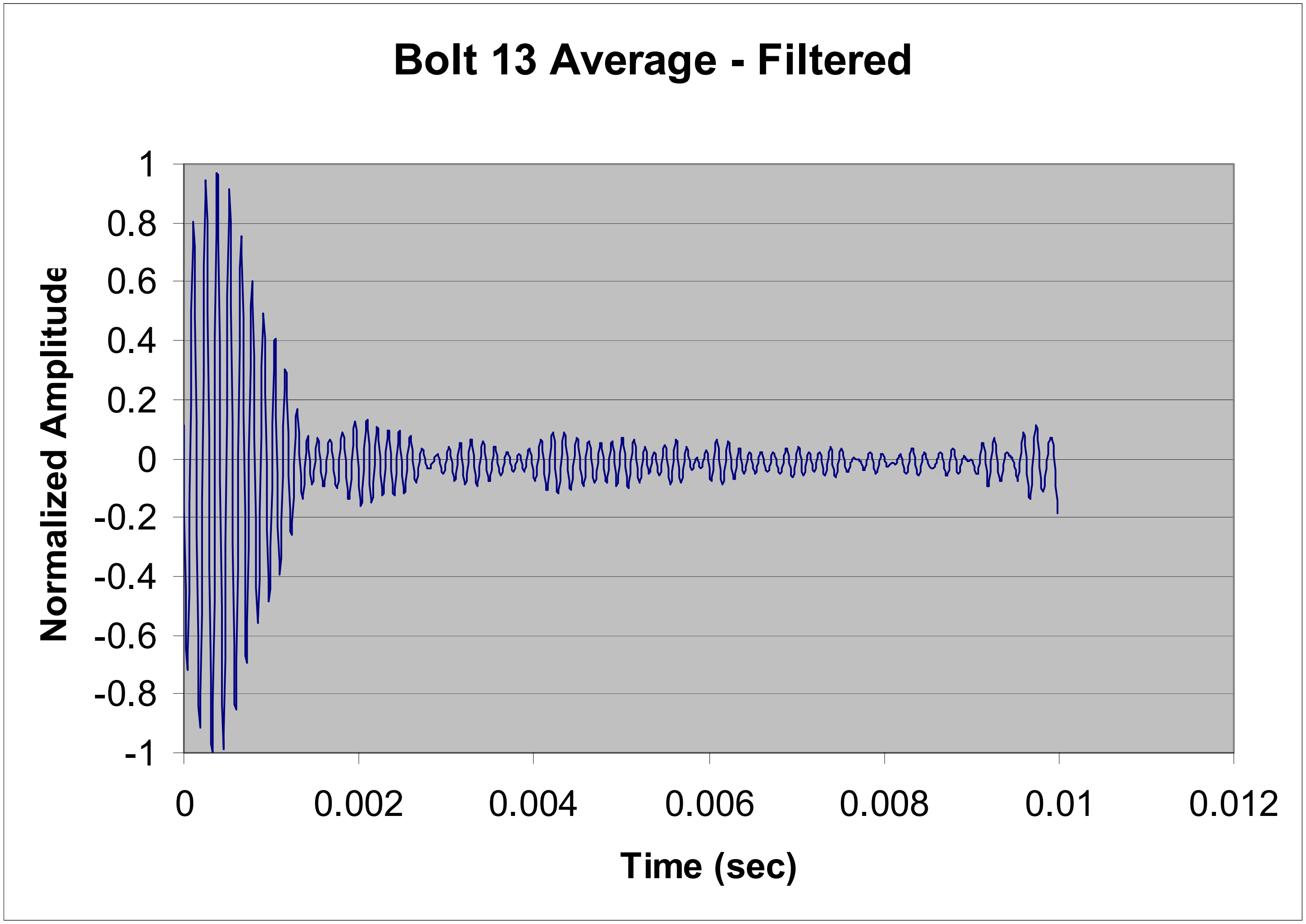
Bolt 10 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
0.012
Time (sec)



Bolt 11a - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
0.012
Time (sec)

BARRON MOUNTAIN BOLT #12- TIME HISTORY





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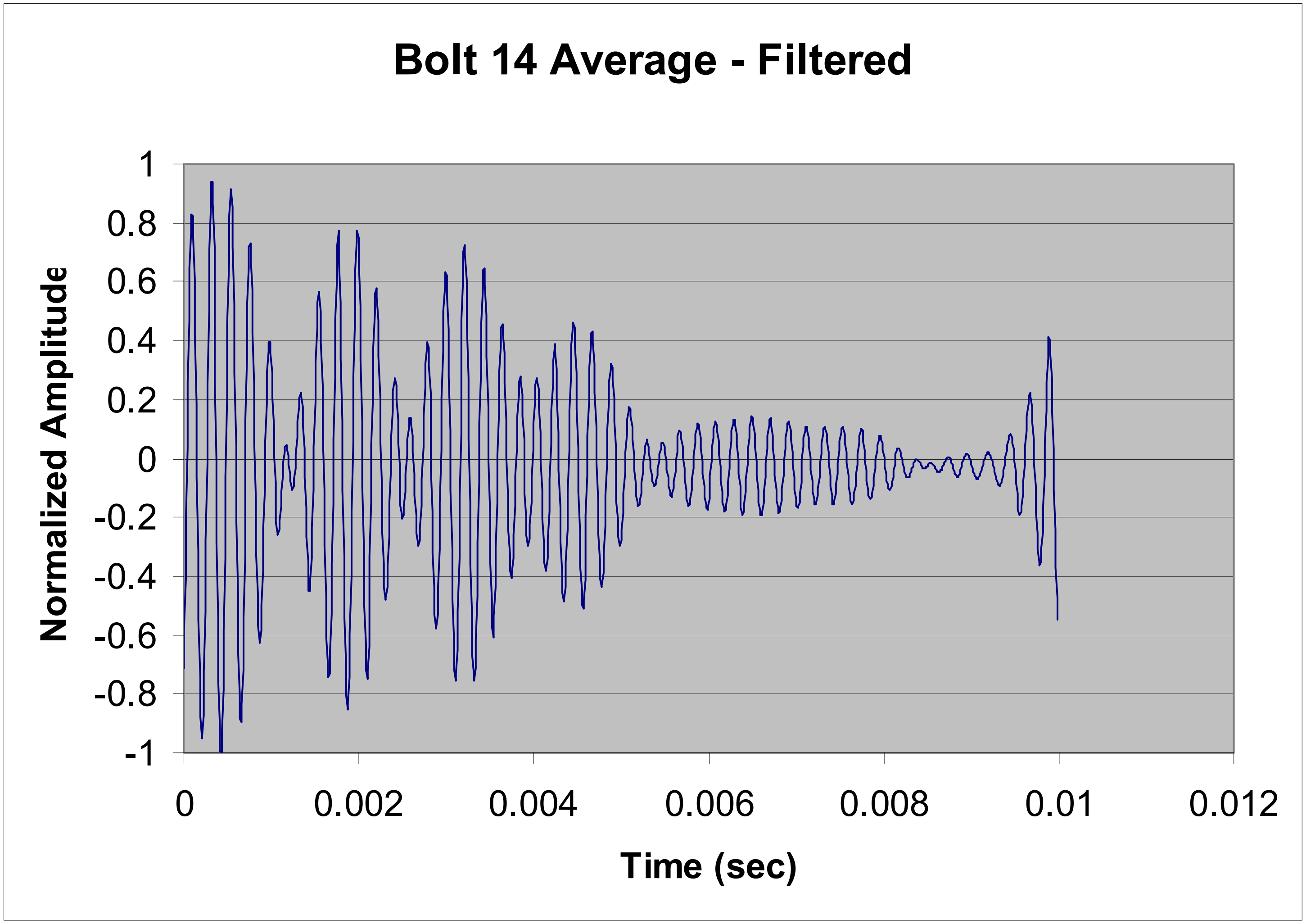

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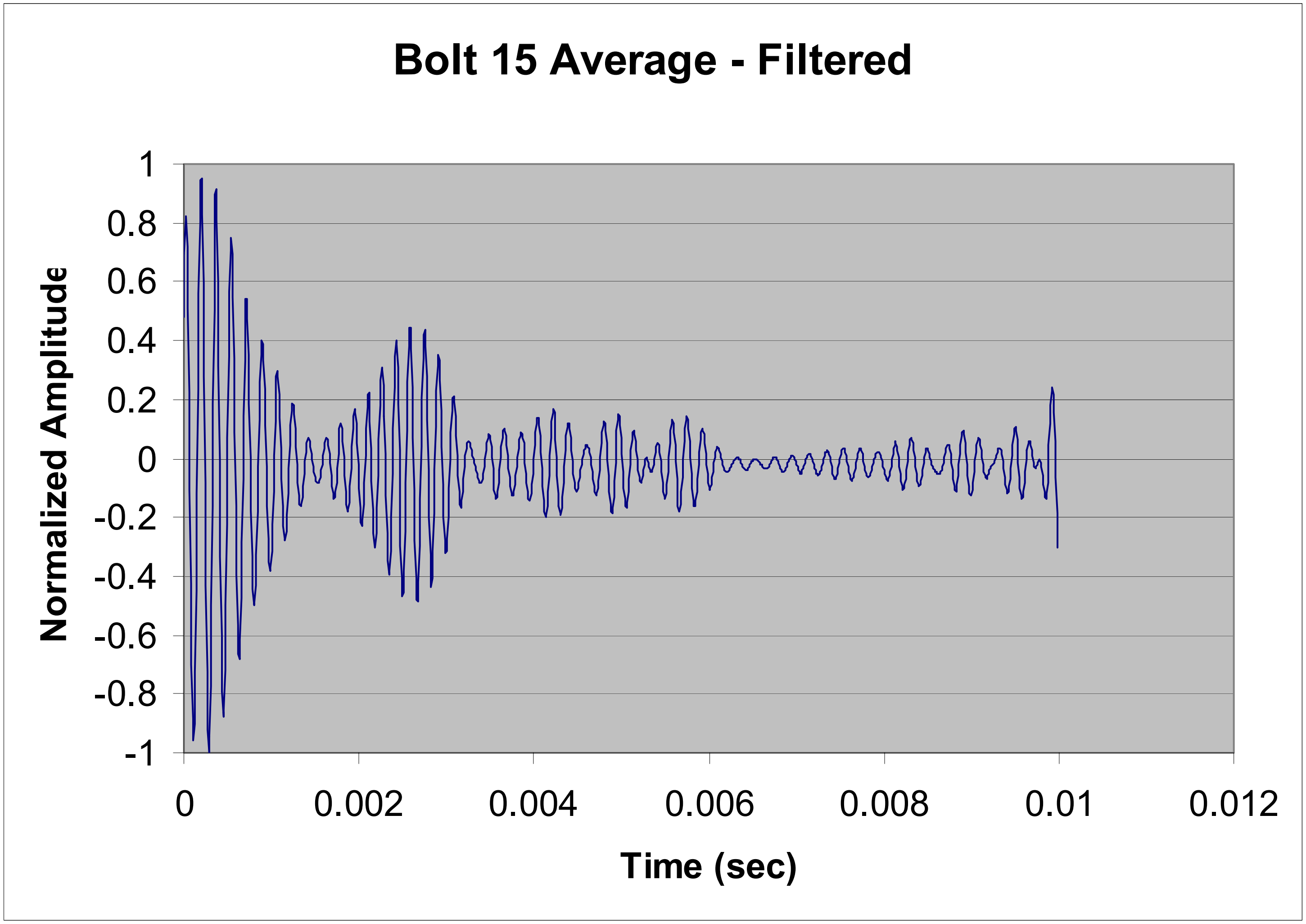
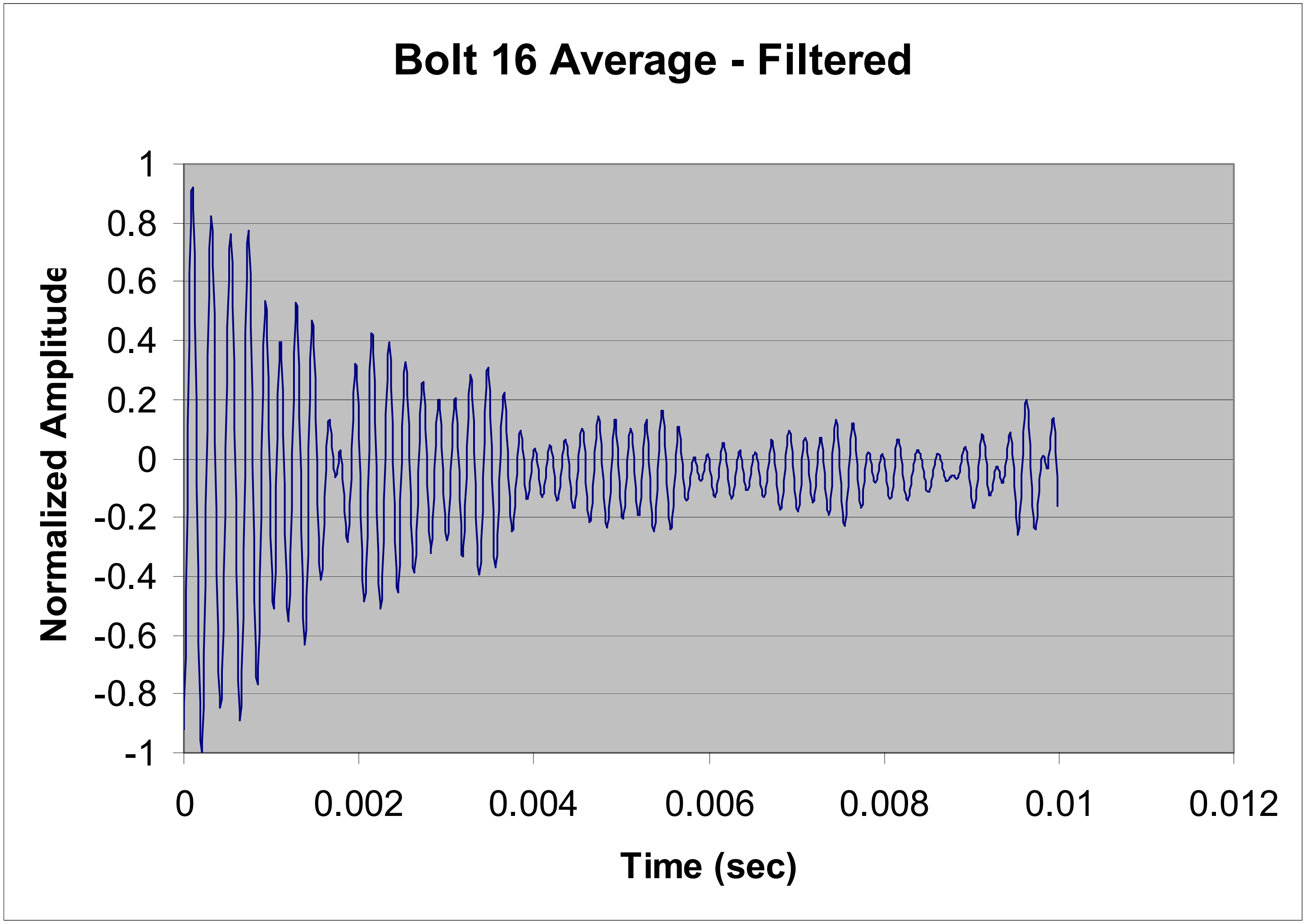

Bolt 15 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
0.012
Time (sec)



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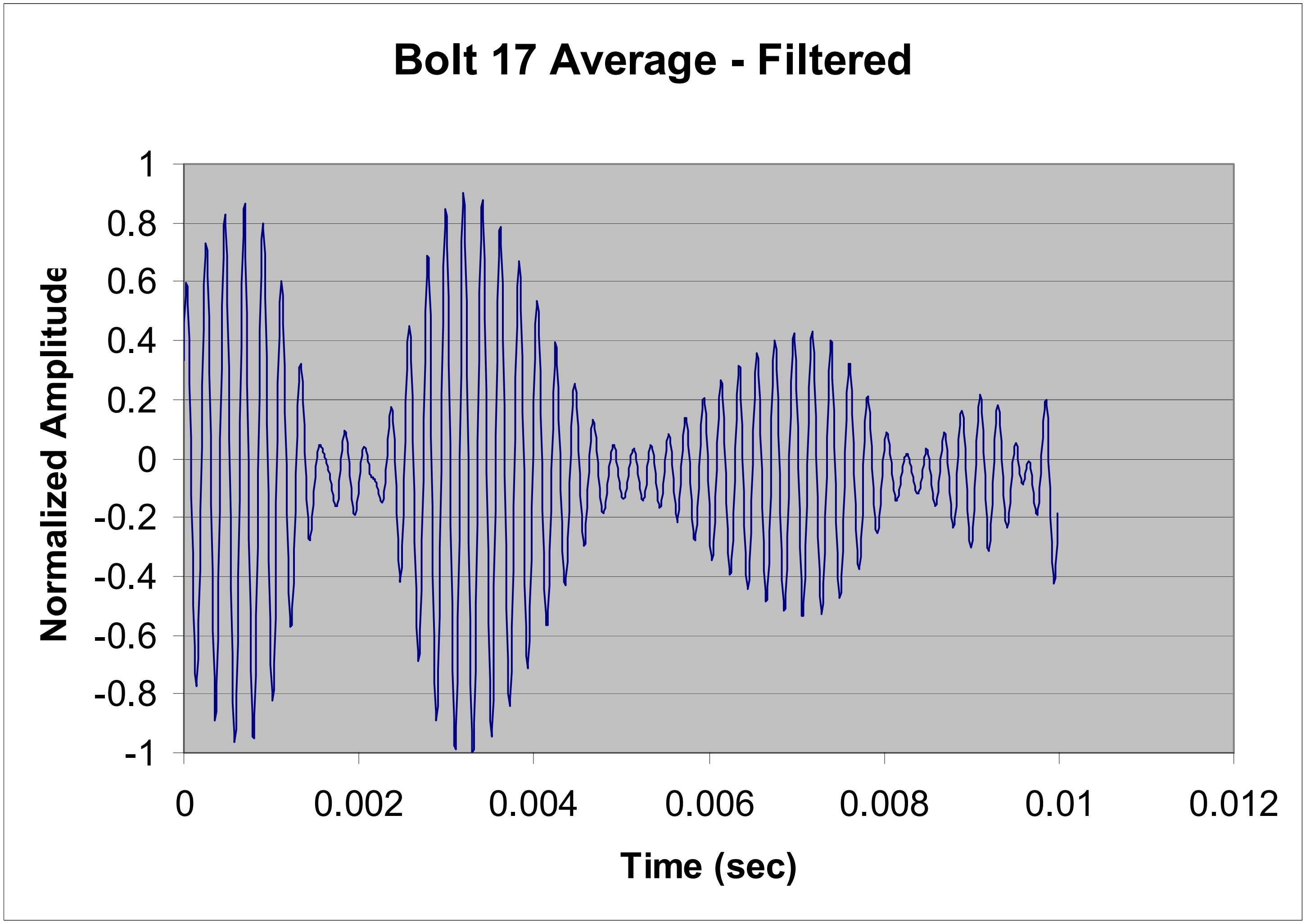
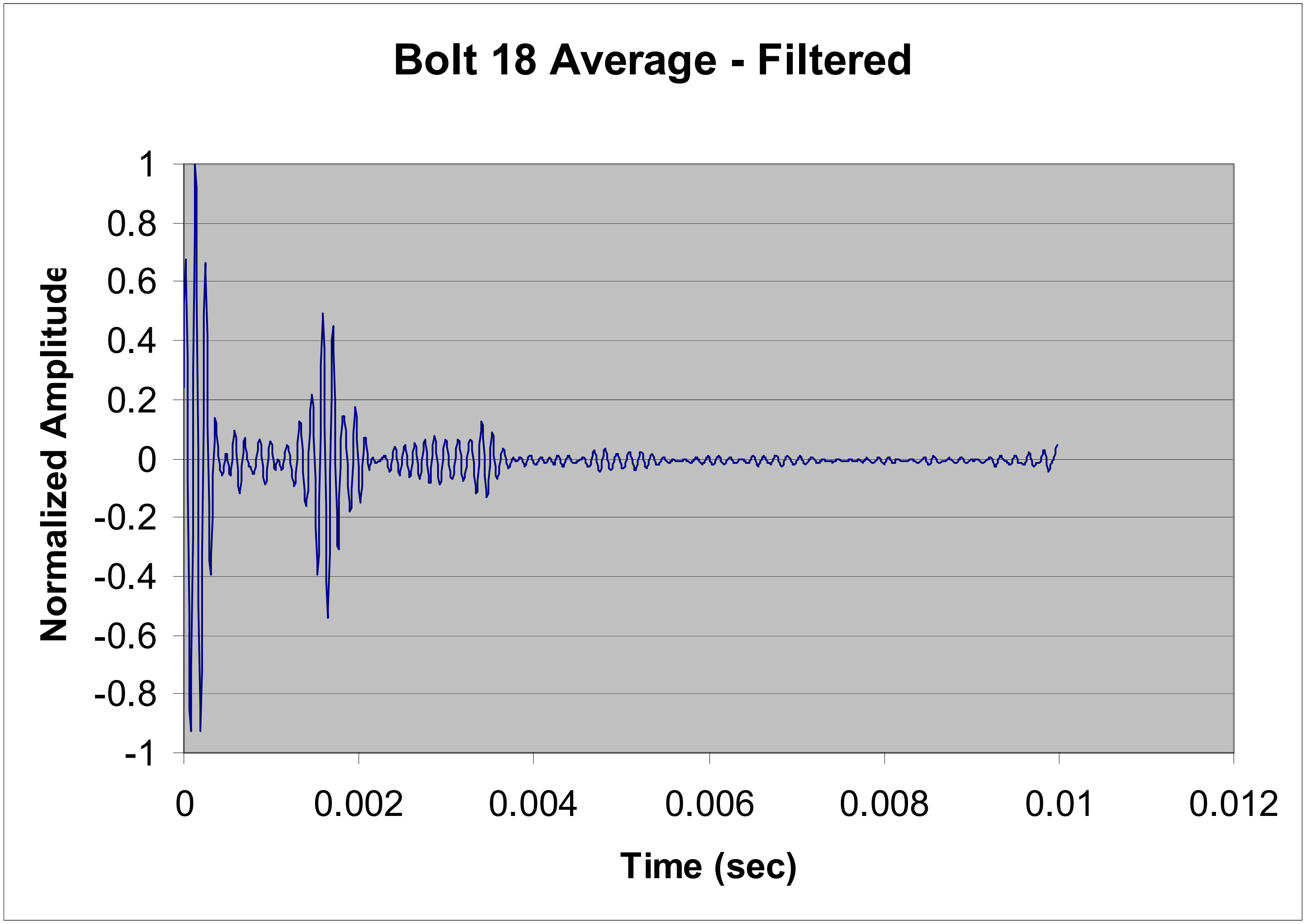
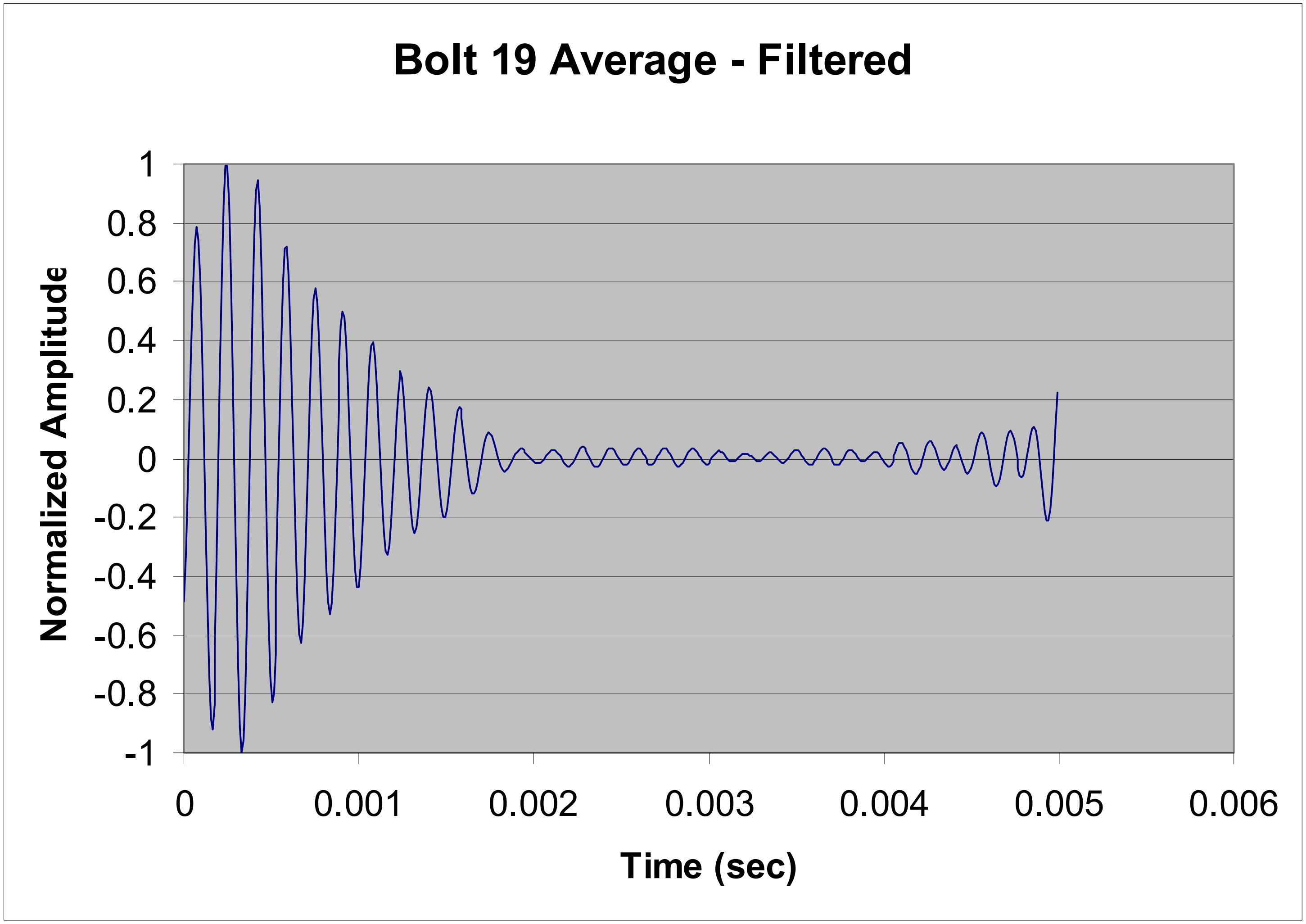
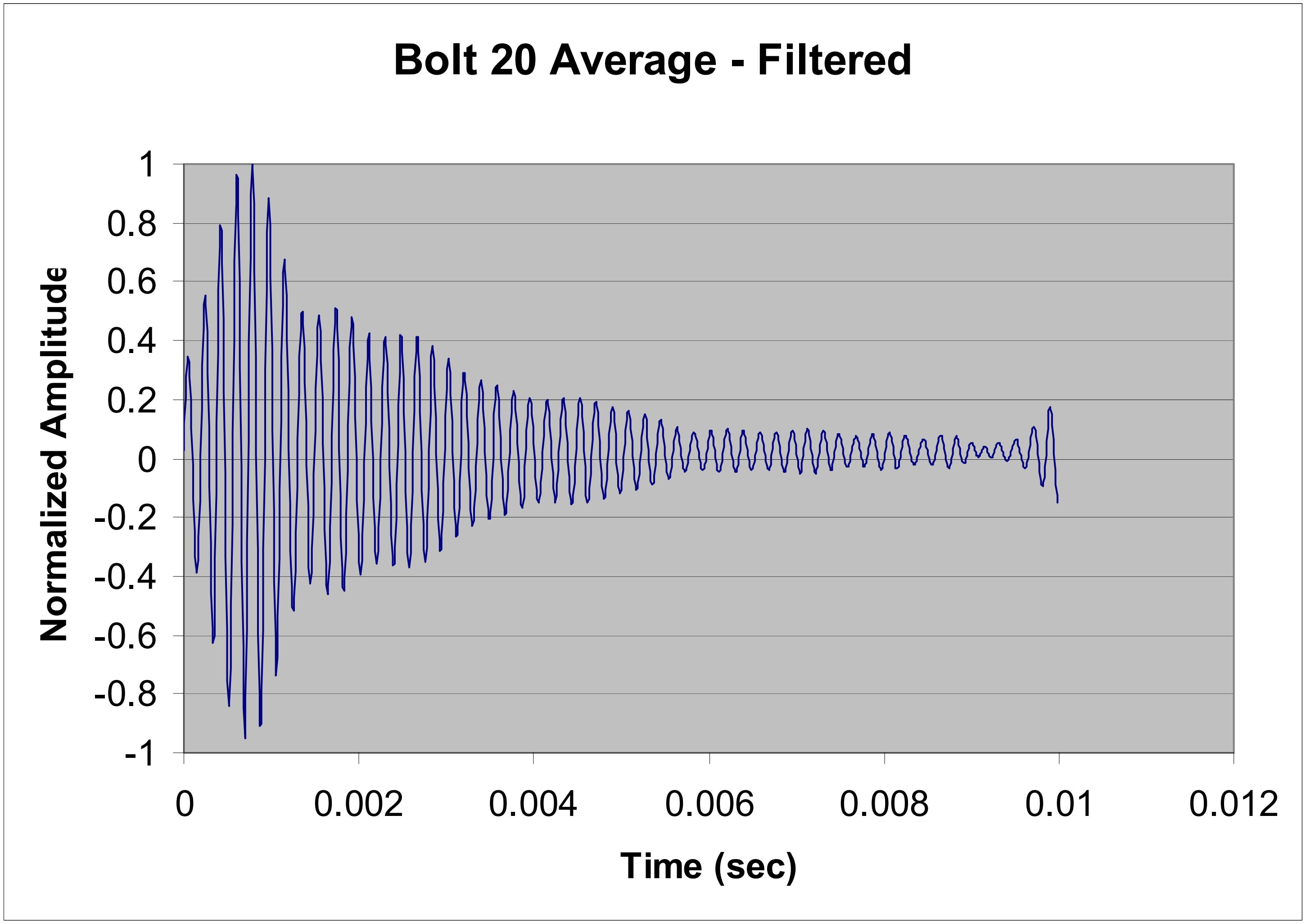

Bolt 17 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
0.012
Time (sec)



Bolt 18 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
0.012
Time (sec)



Bolt 19 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.001
0.002
0.003
0.004
0.005
0.006
Time (sec)



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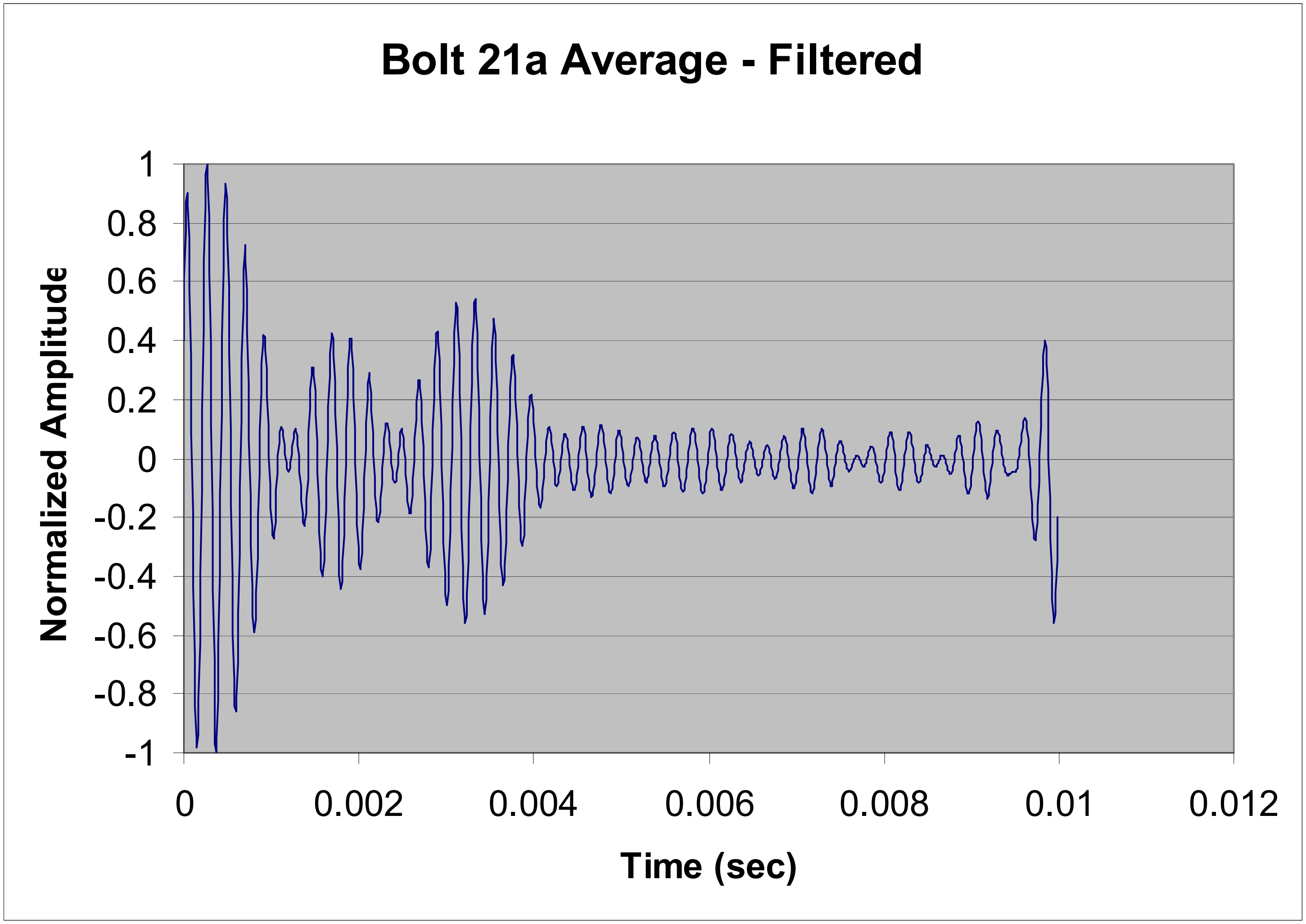
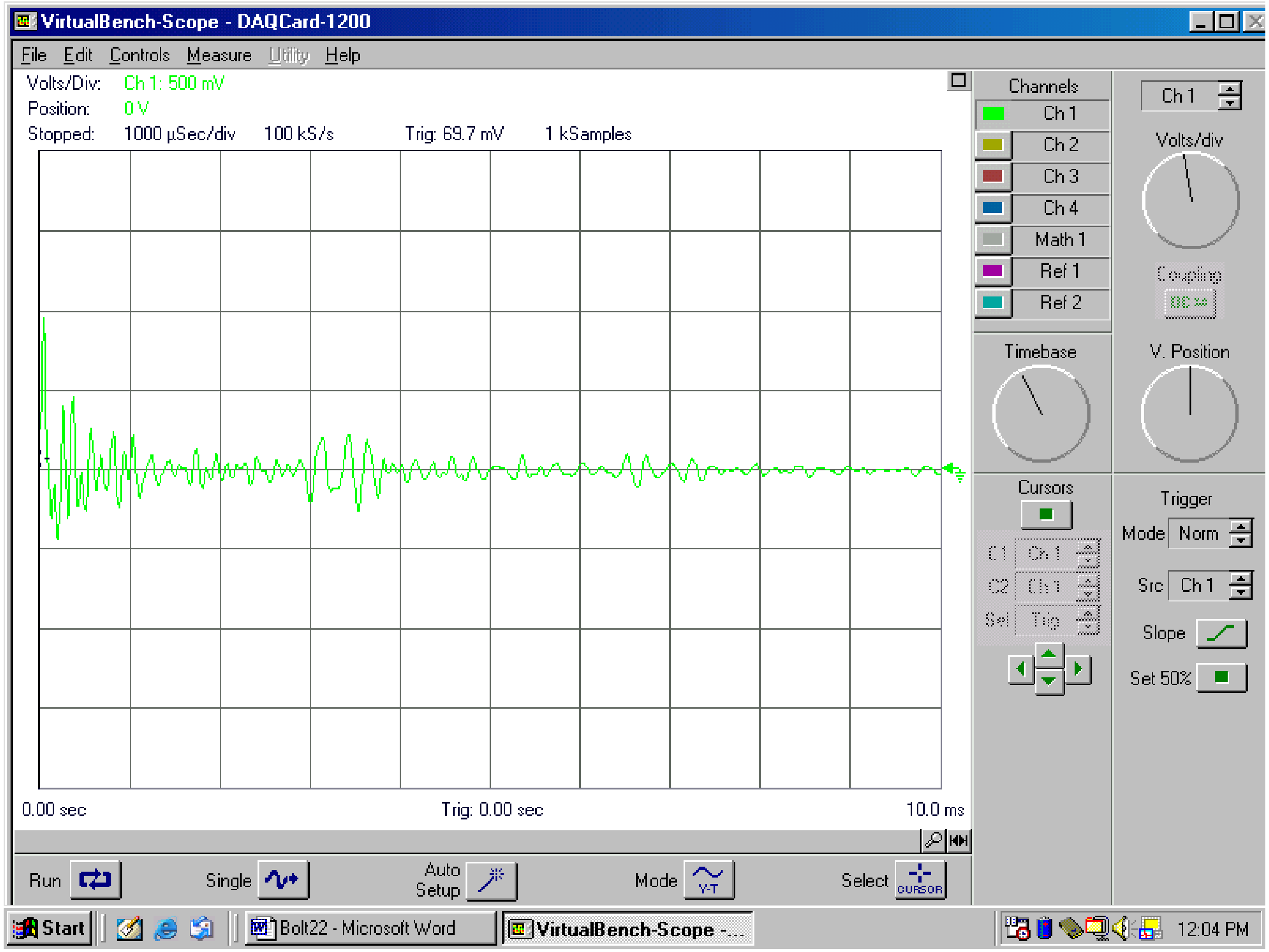

Bolt 21a Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
0.012
Time (sec)

BARRON MOUNTAIN BOLT #22- TIME HISTORY



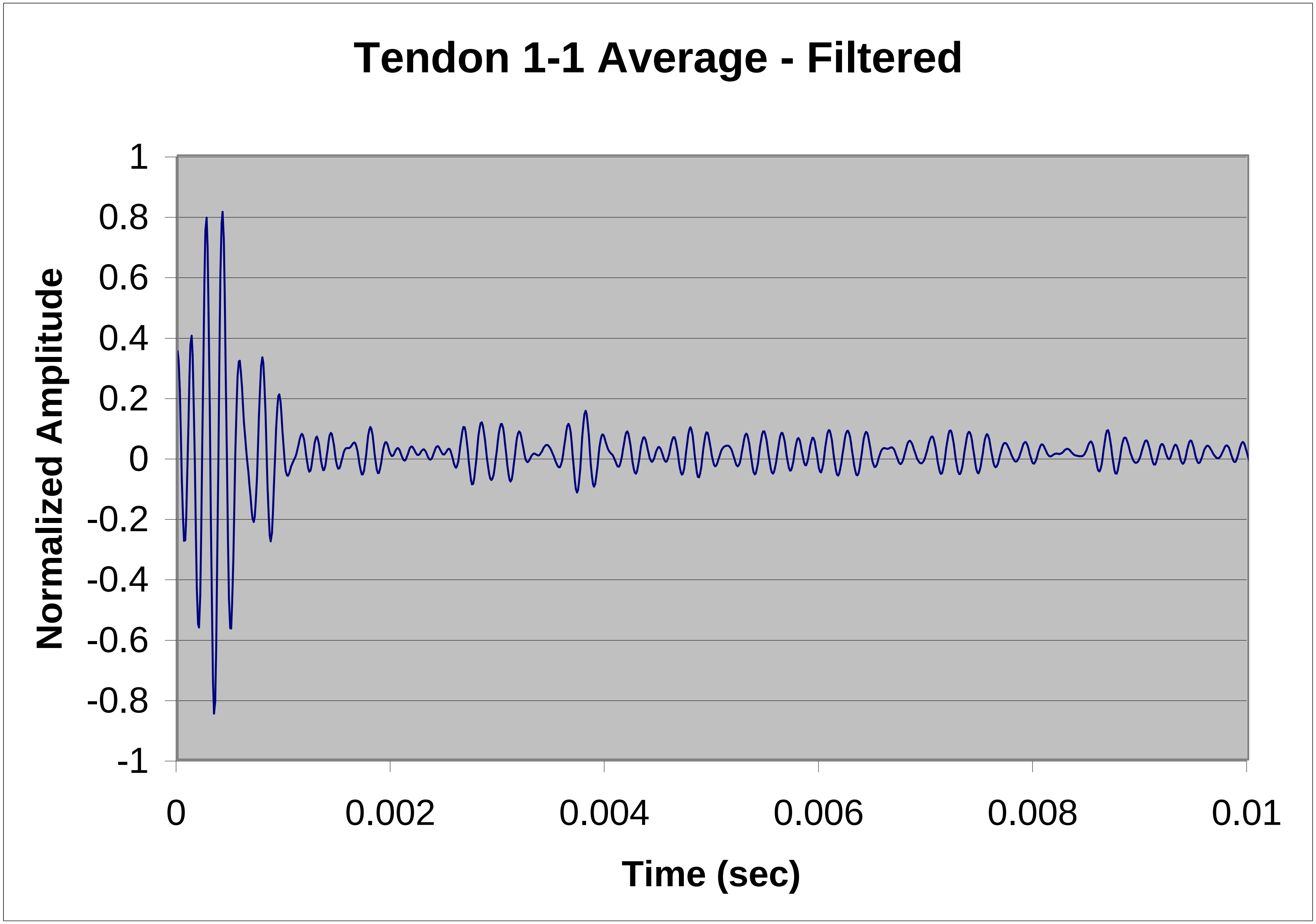
McMahon & Mann
Consulting Engineers, P.C.

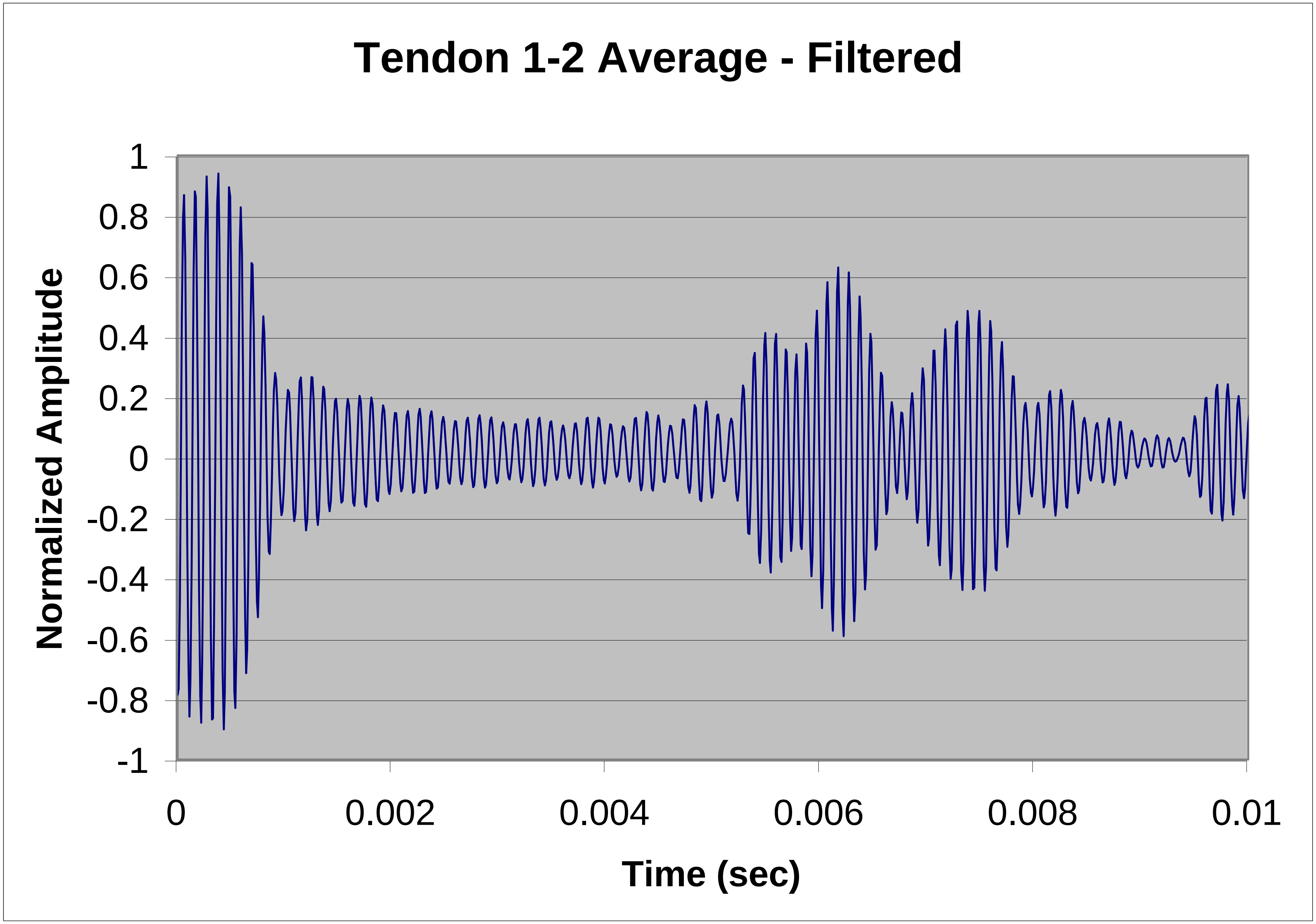
APPENDIX VIII

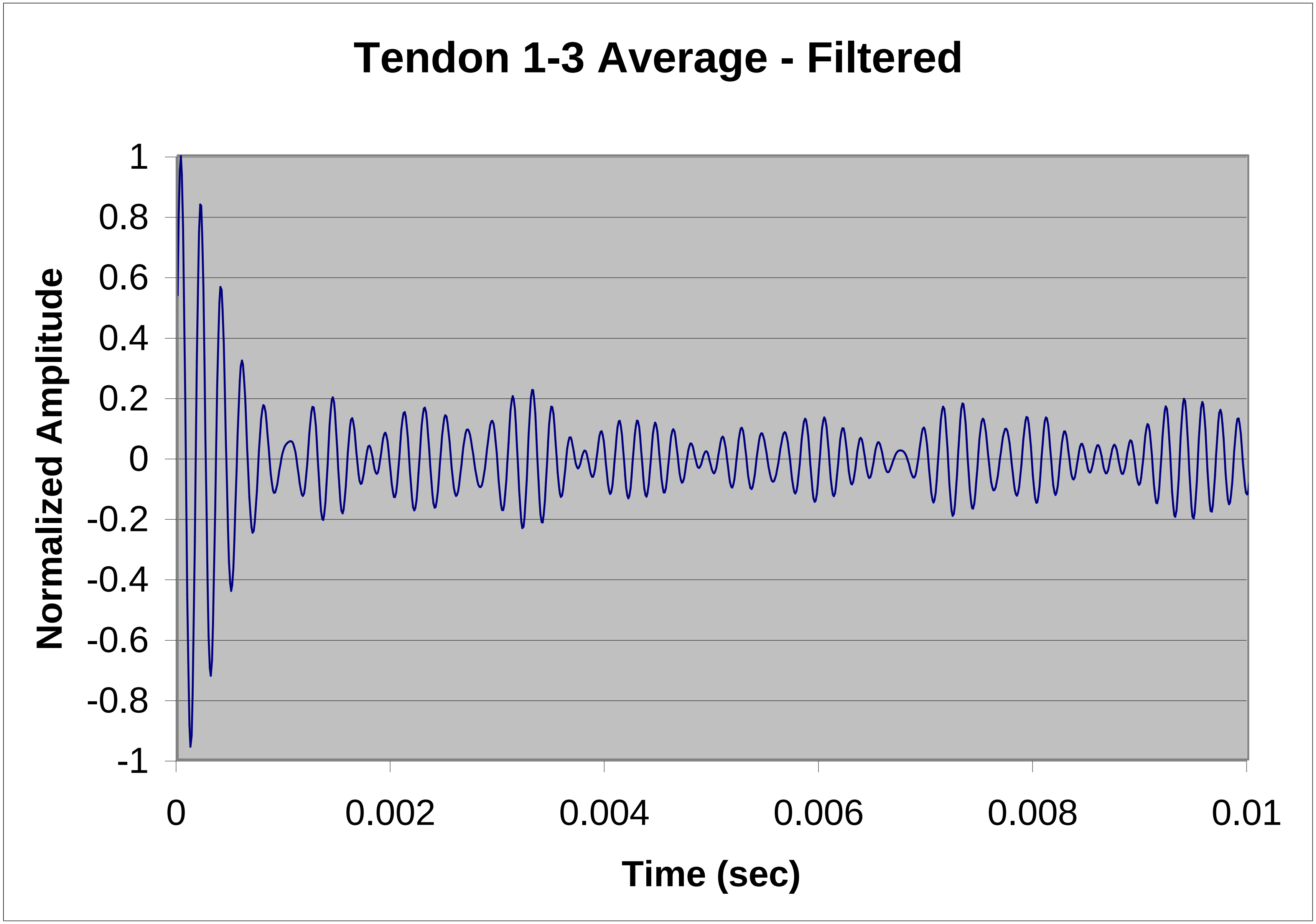
ROCK TENDONS
IMPACT ACCELERATION TIME HISTORIES

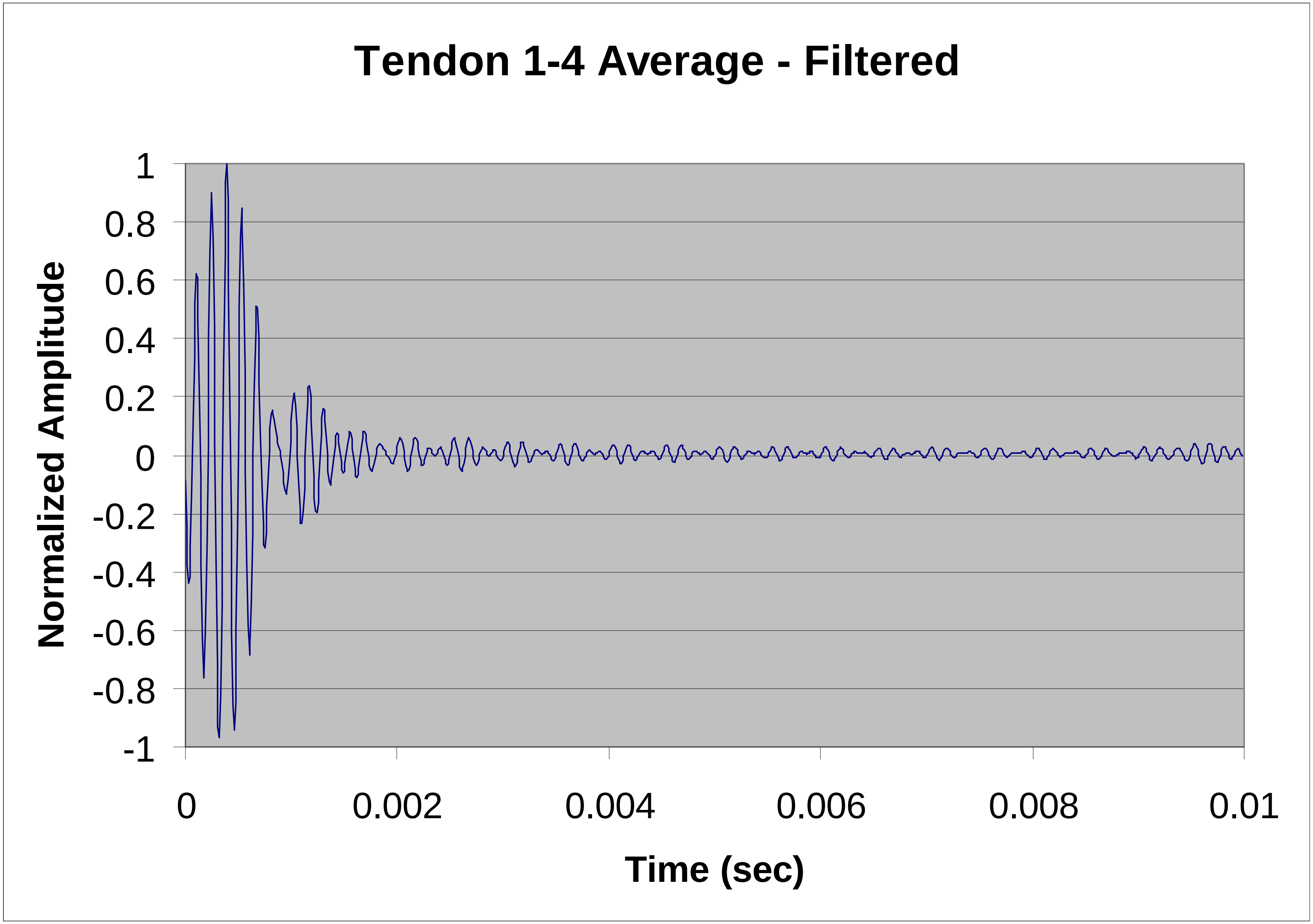
Table VIII-1. Observed Reflections from Impact Test on Tendon Reinforcements.

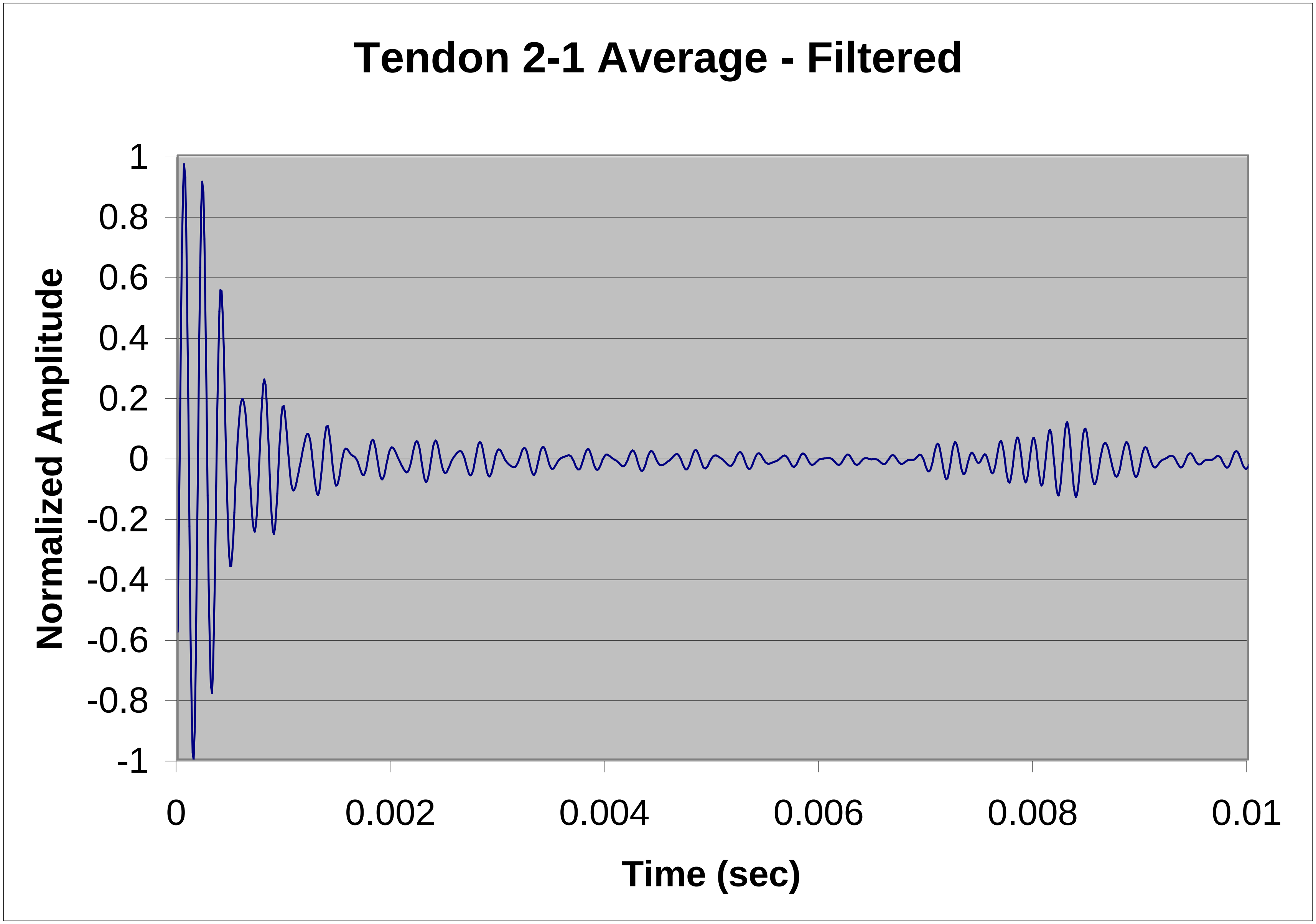
Test Tendon #	Relative Signal Attenuation	Observed L_1 (ft)	Observed L_2 (ft)
1-1	Strong	8	55
1-2	Weak	~	51
1-3	Weak	27	58
1-4	Strong	8	~
2-1	Strong	~	57
2-2	Strong	5	~
2-3	Strong	5	63
2-4	Strong	10	~
3-1	Weak	17	~
3-2	Strong	8	62
3-3	Strong	6	~
3-4	Strong	6	64
4-1	Strong	9	~
4-2	Strong	8	~
4-3	Strong	4	~
4-4	Strong	3	~
5-1	Strong	11	~
5-2	Weak	8	~
5-3	Strong	6	~
5-4	Strong	4	~


Tendon 1-1 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)

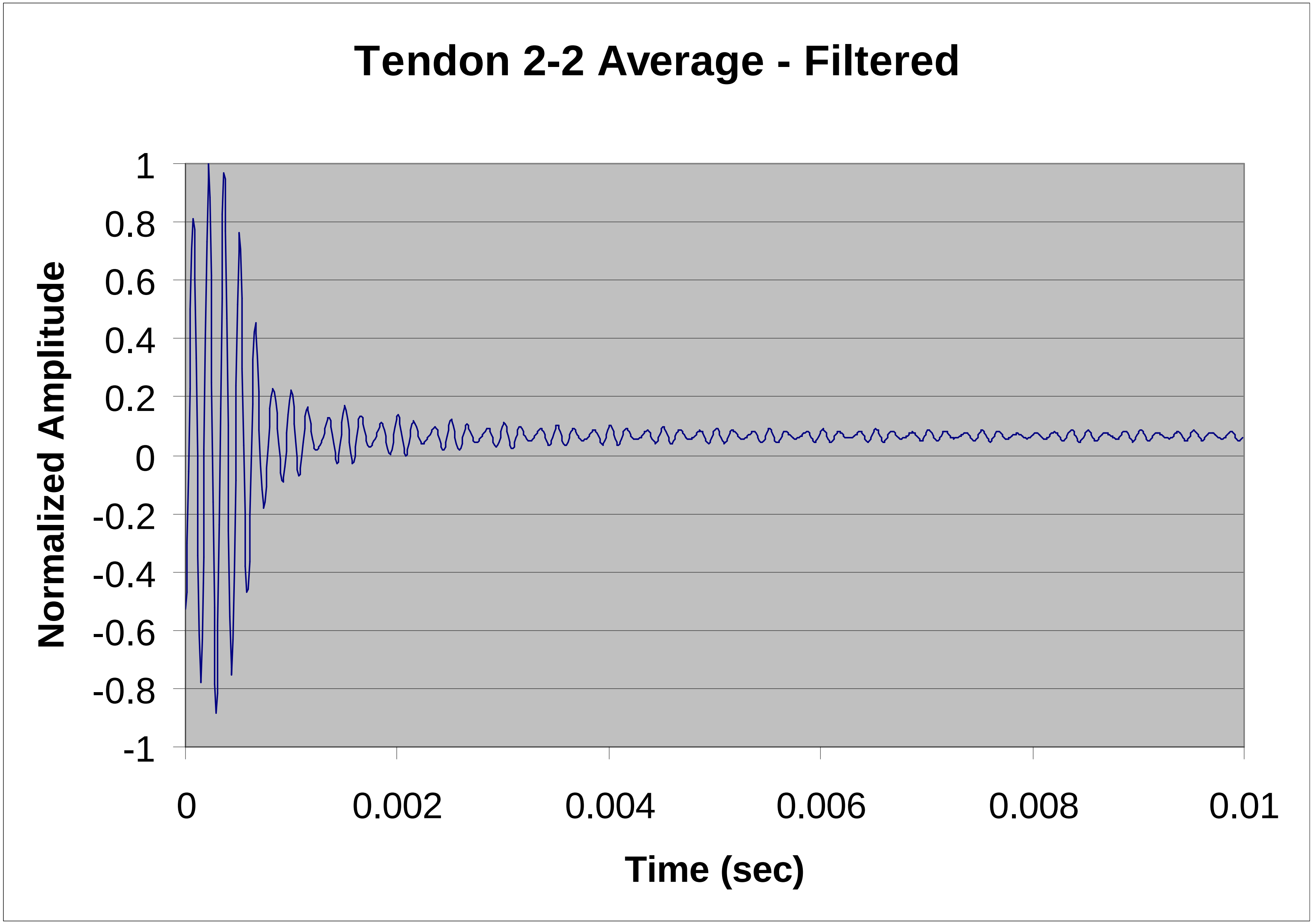

Tendon 1-2 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)

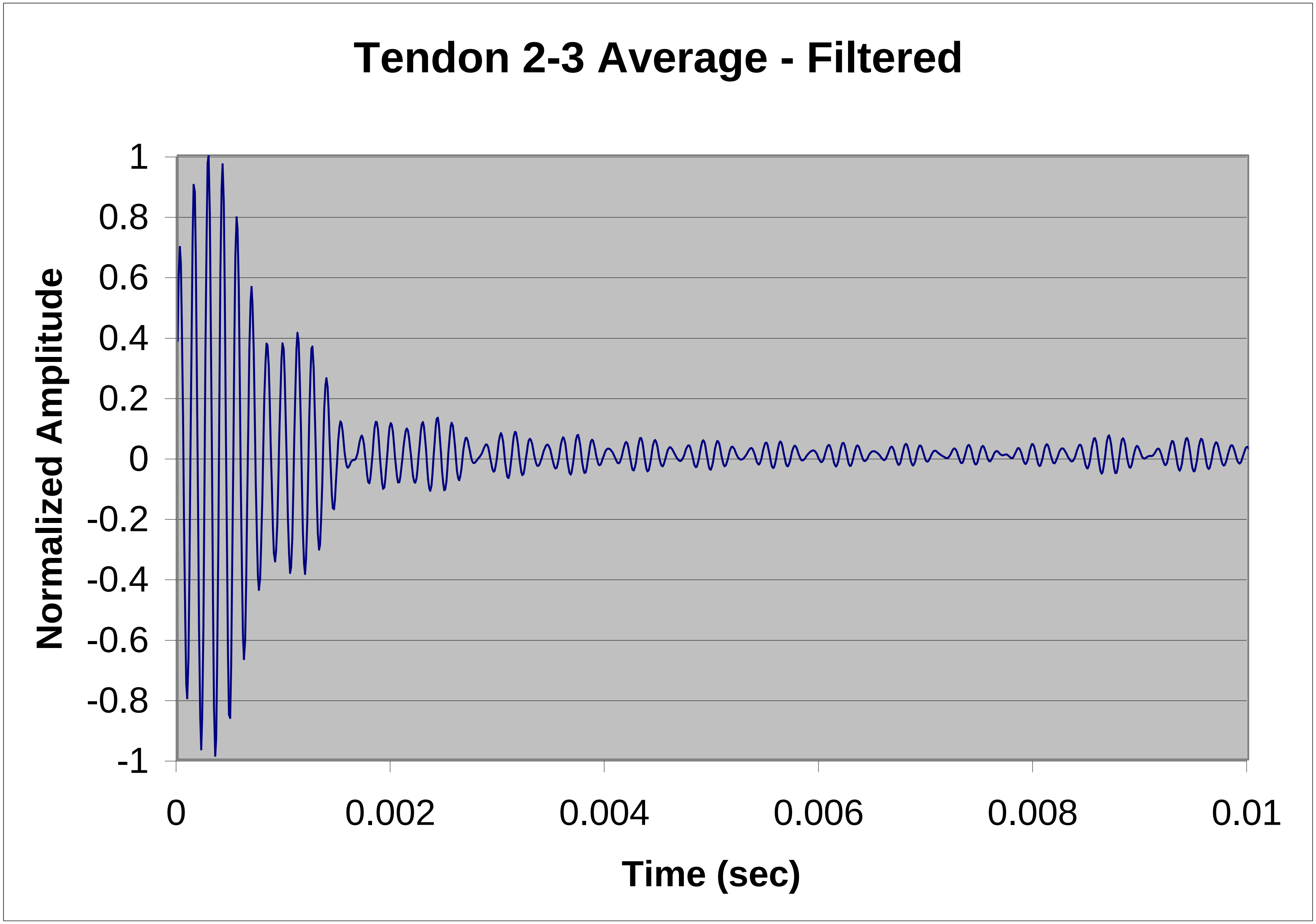

Tendon 1-3 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)


Tendon 1-4 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)

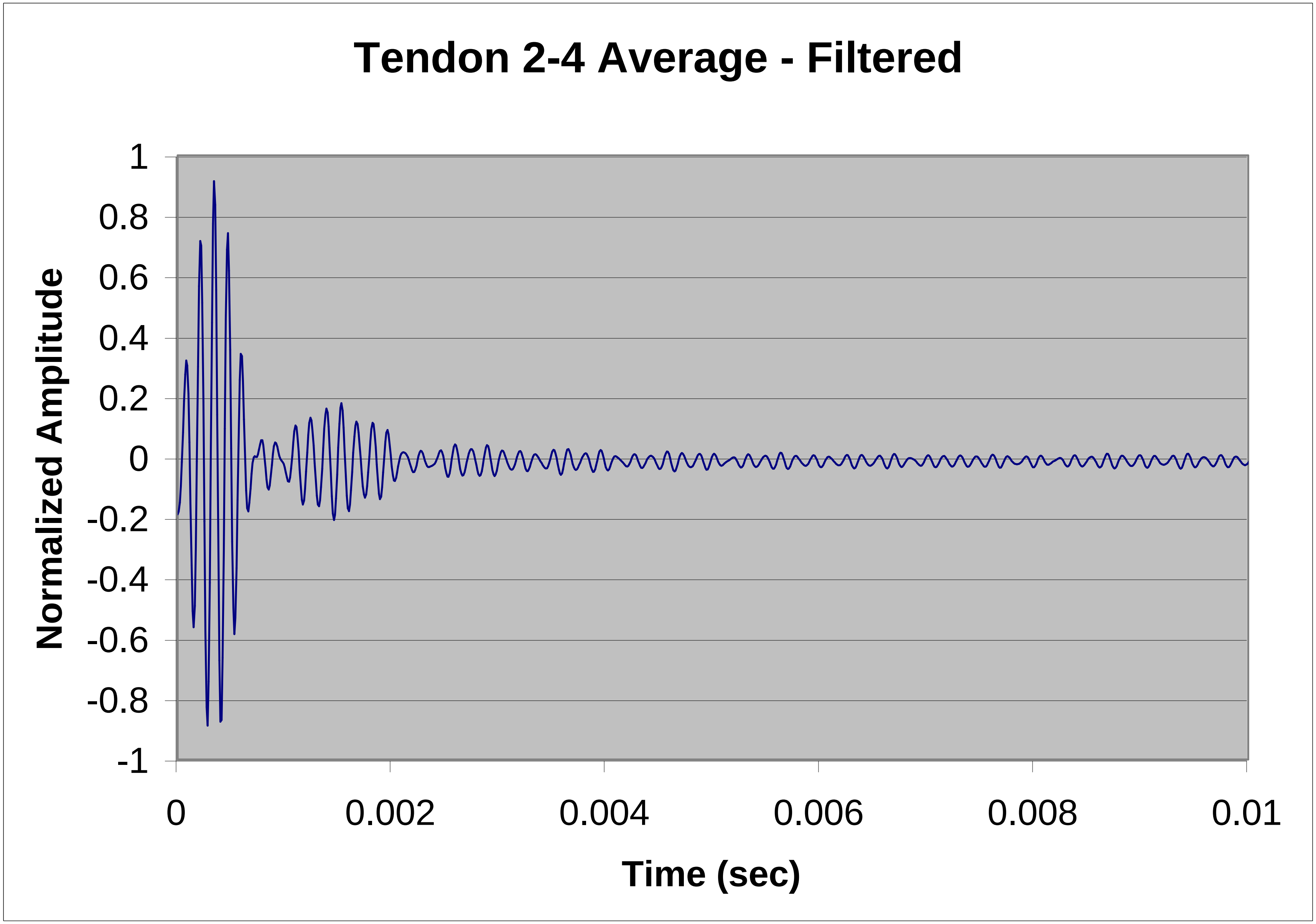


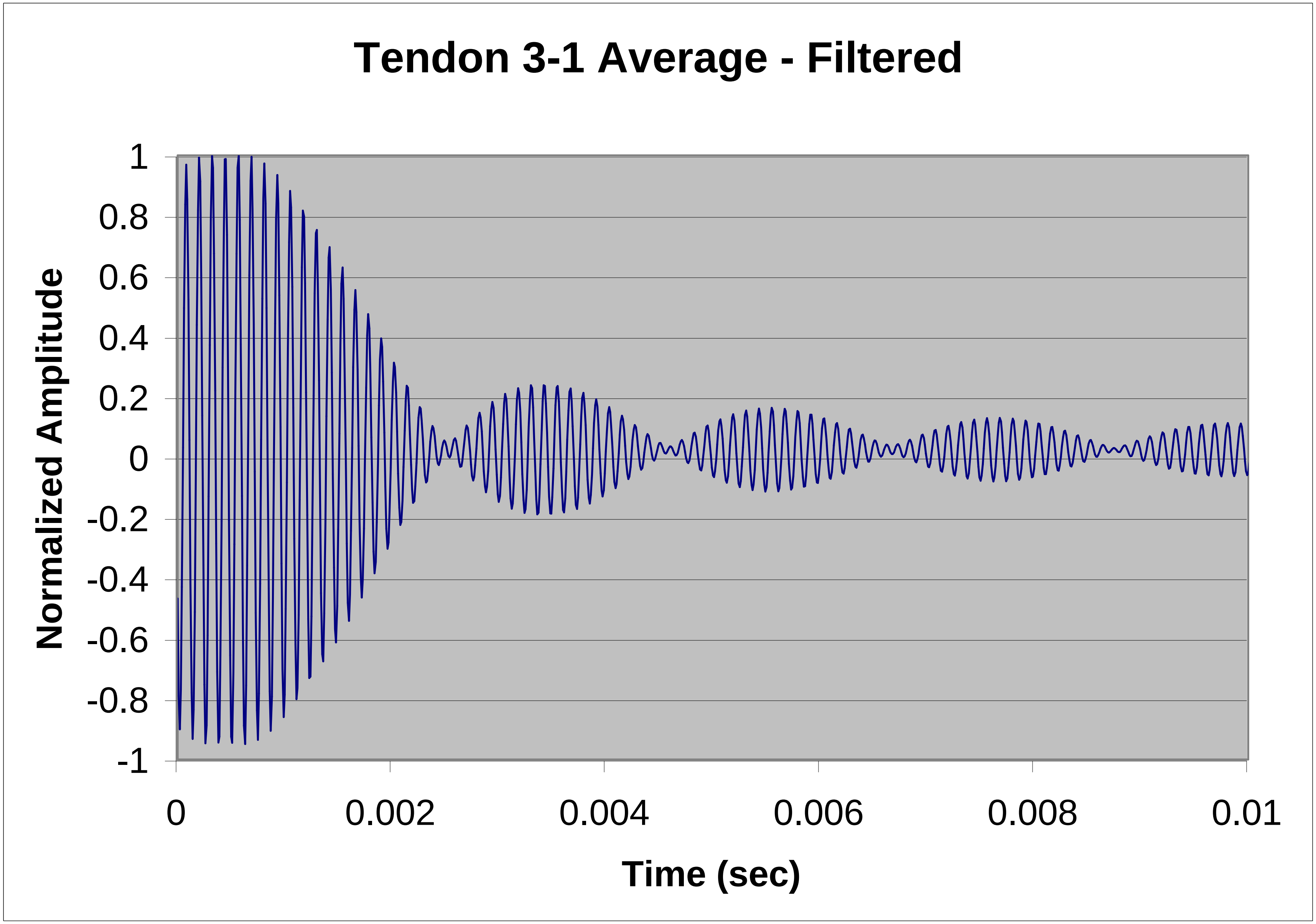
Tendon 2-1 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)

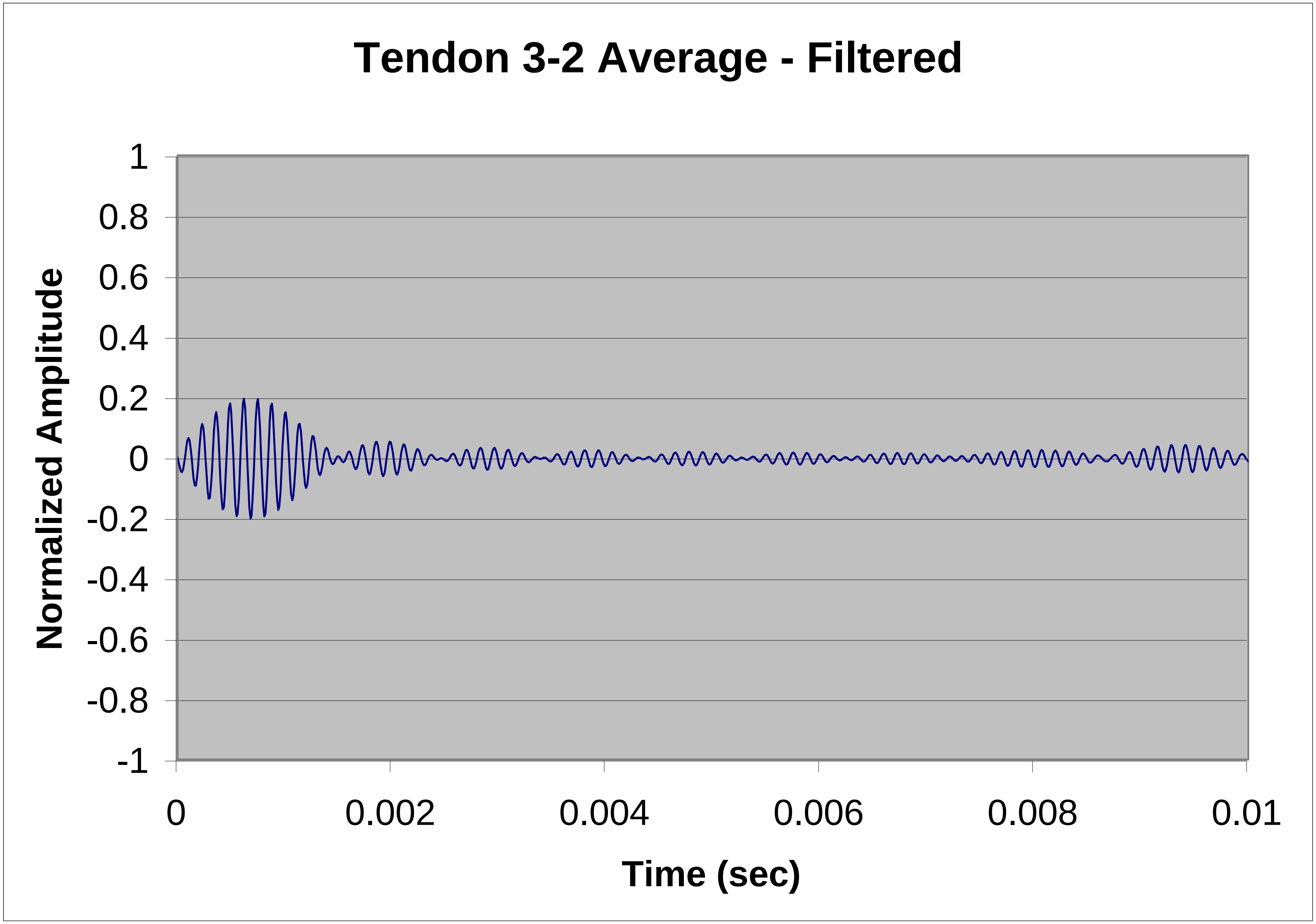

Tendon 2-2 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)

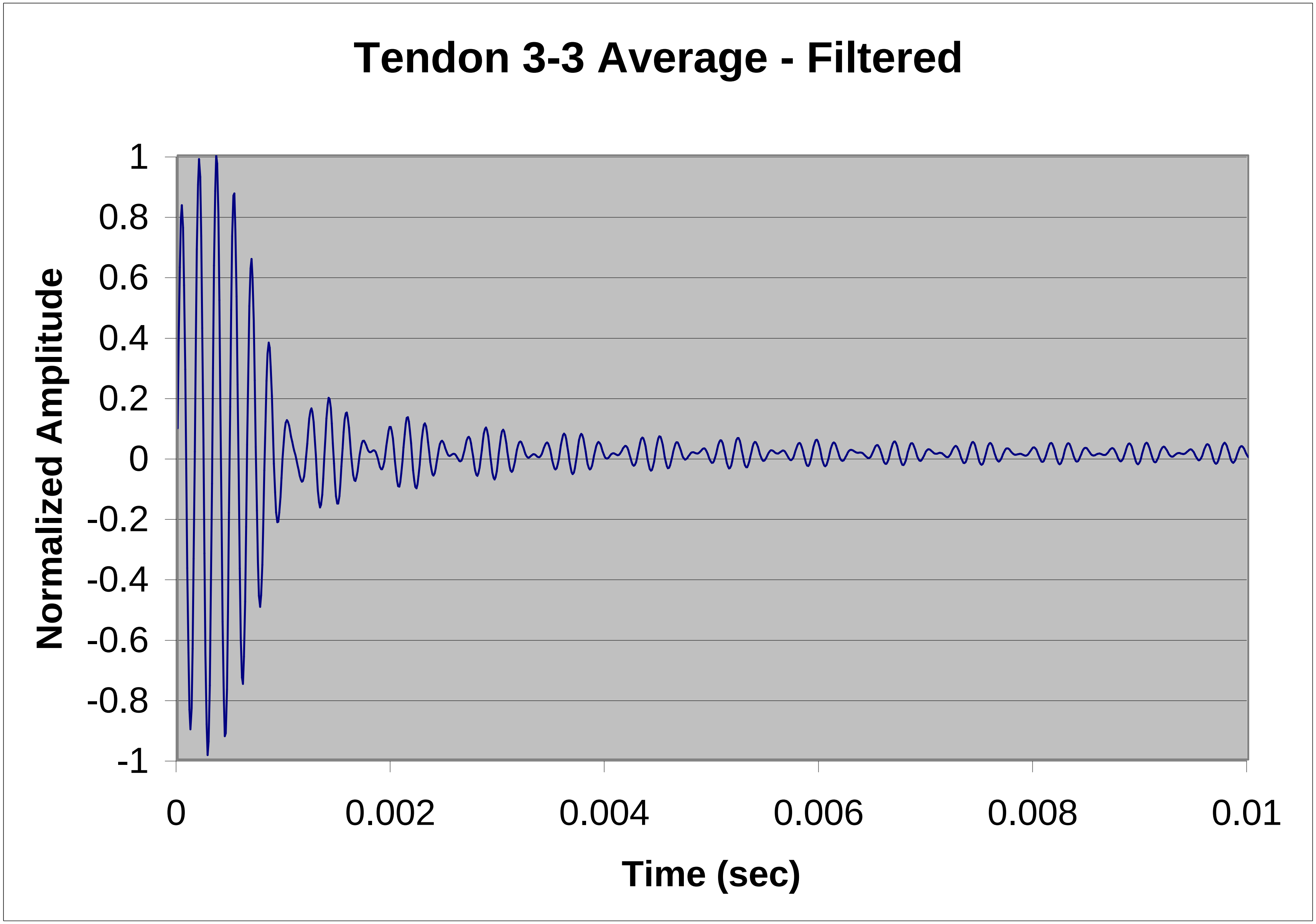


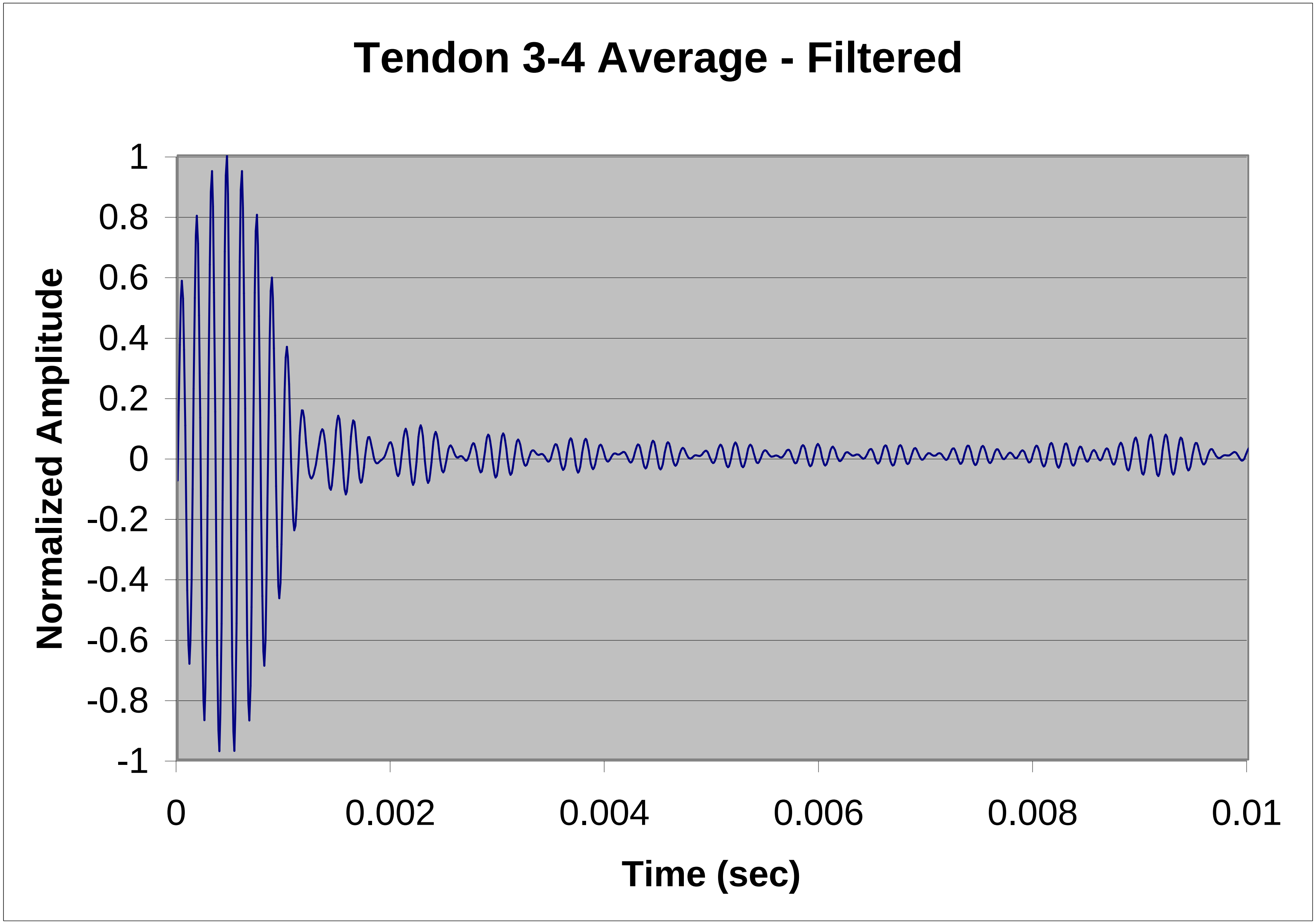
Tendon 2-3 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)

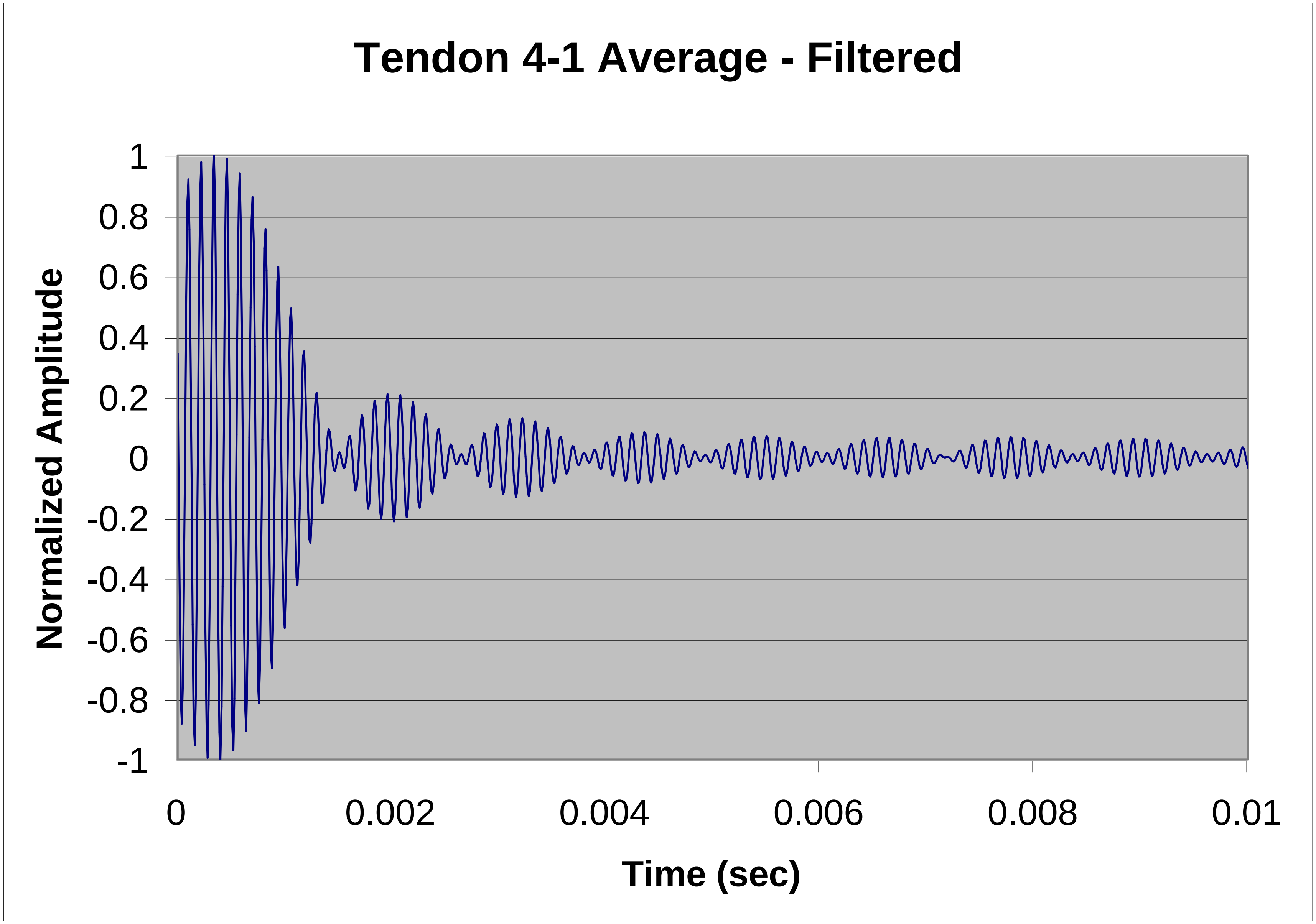

Tendon 2-4 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)


Tendon 3-1 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)

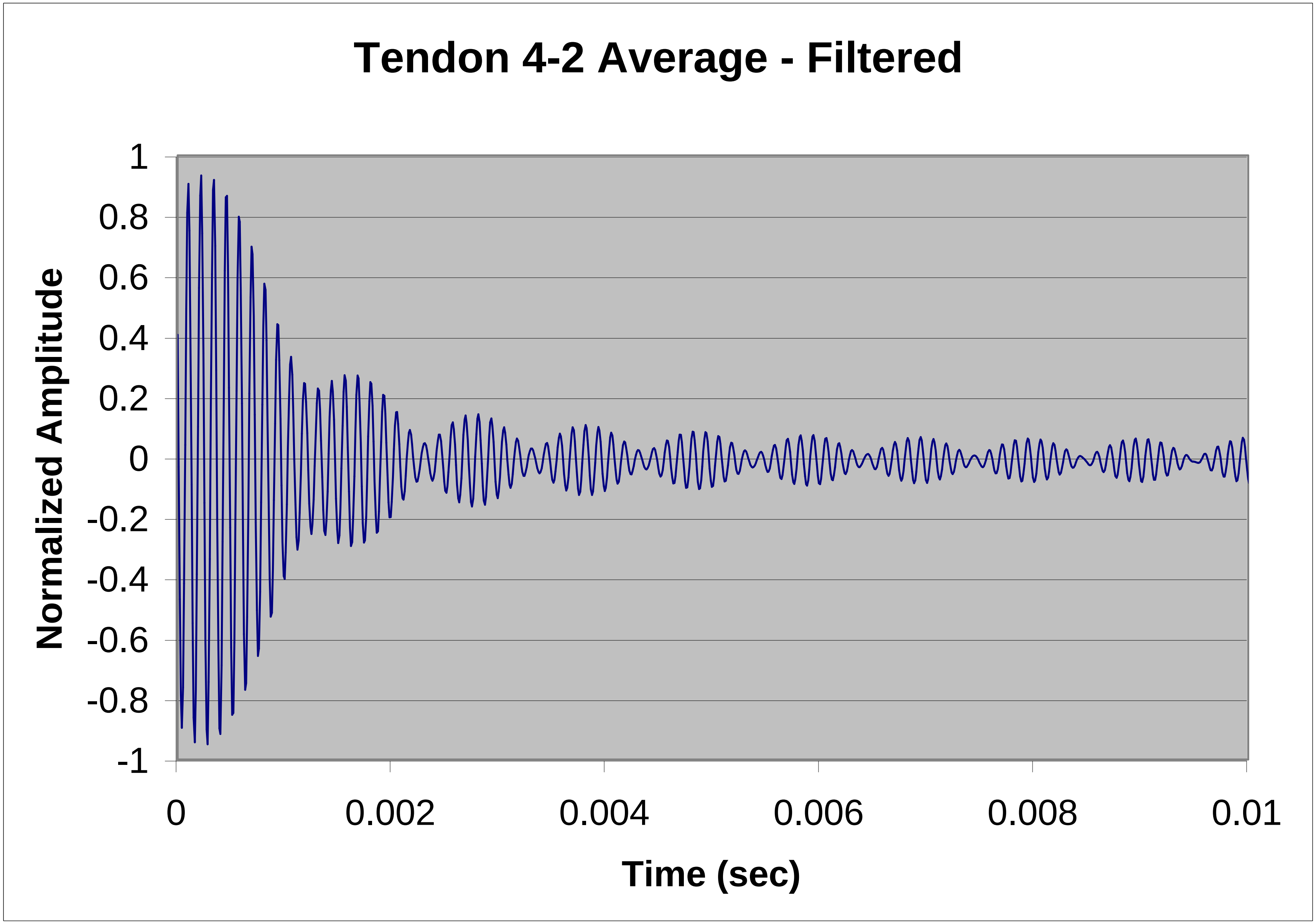

Tendon 3-2 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)

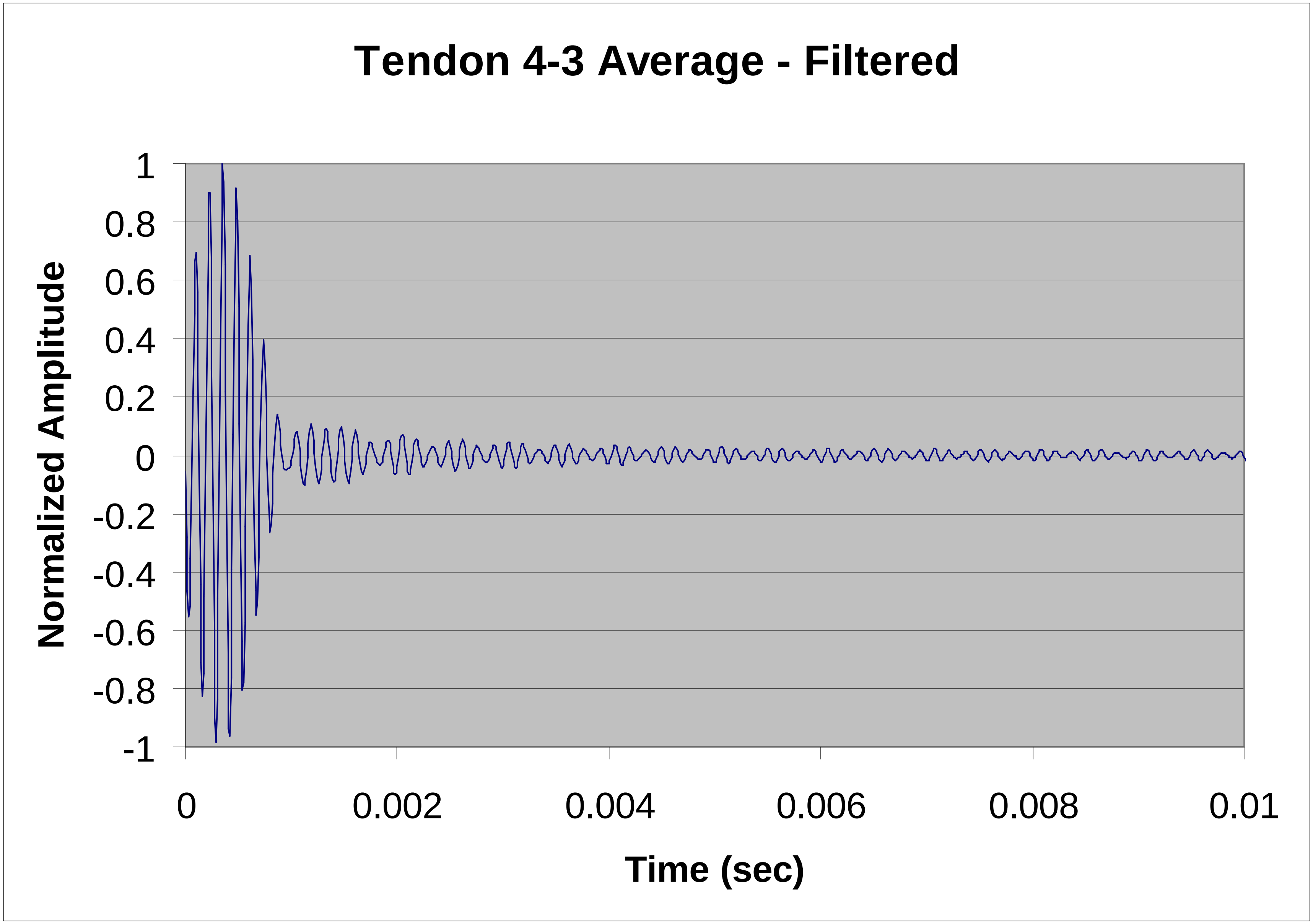

Tendon 3-3 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)


Tendon 3-4 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)

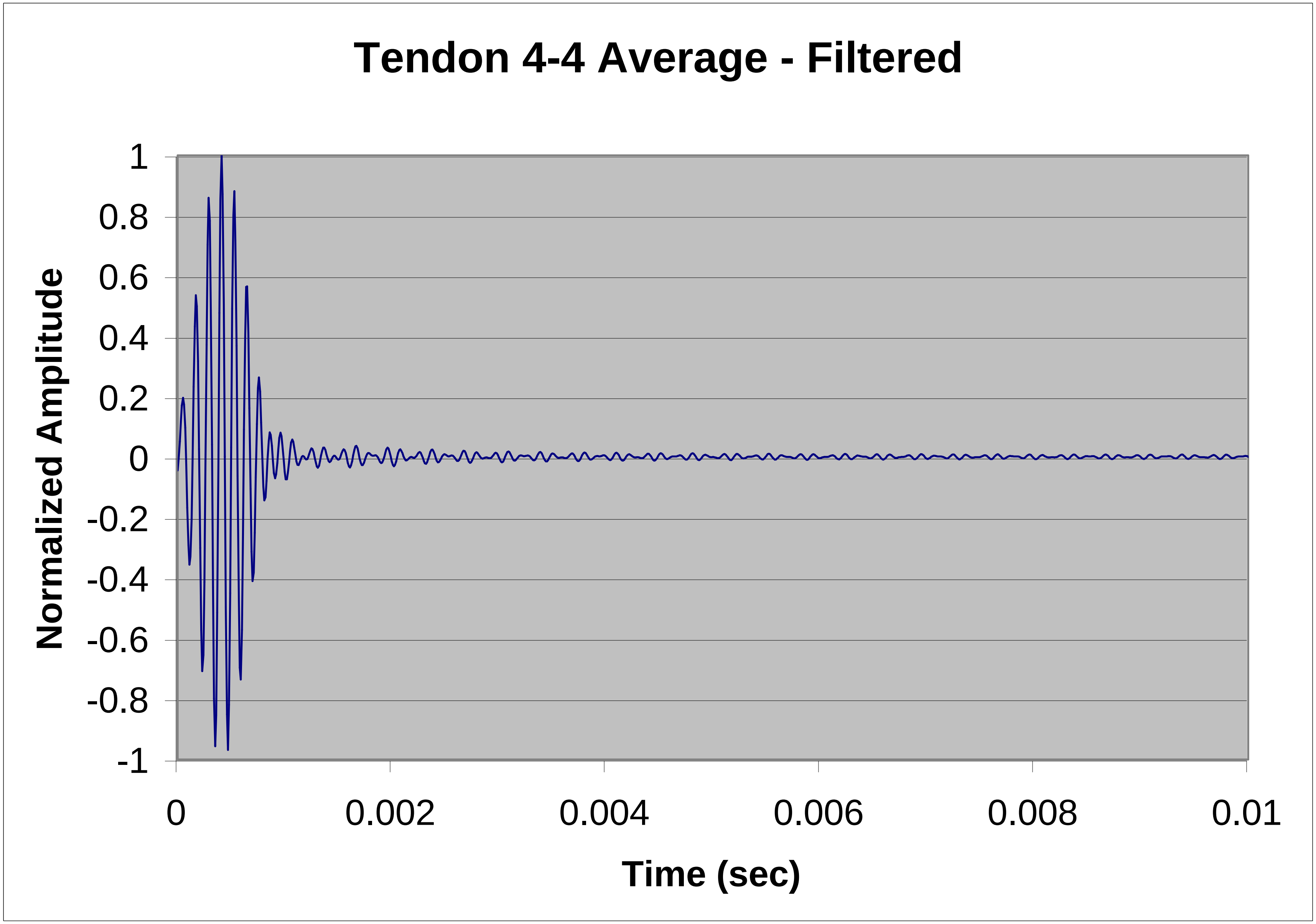


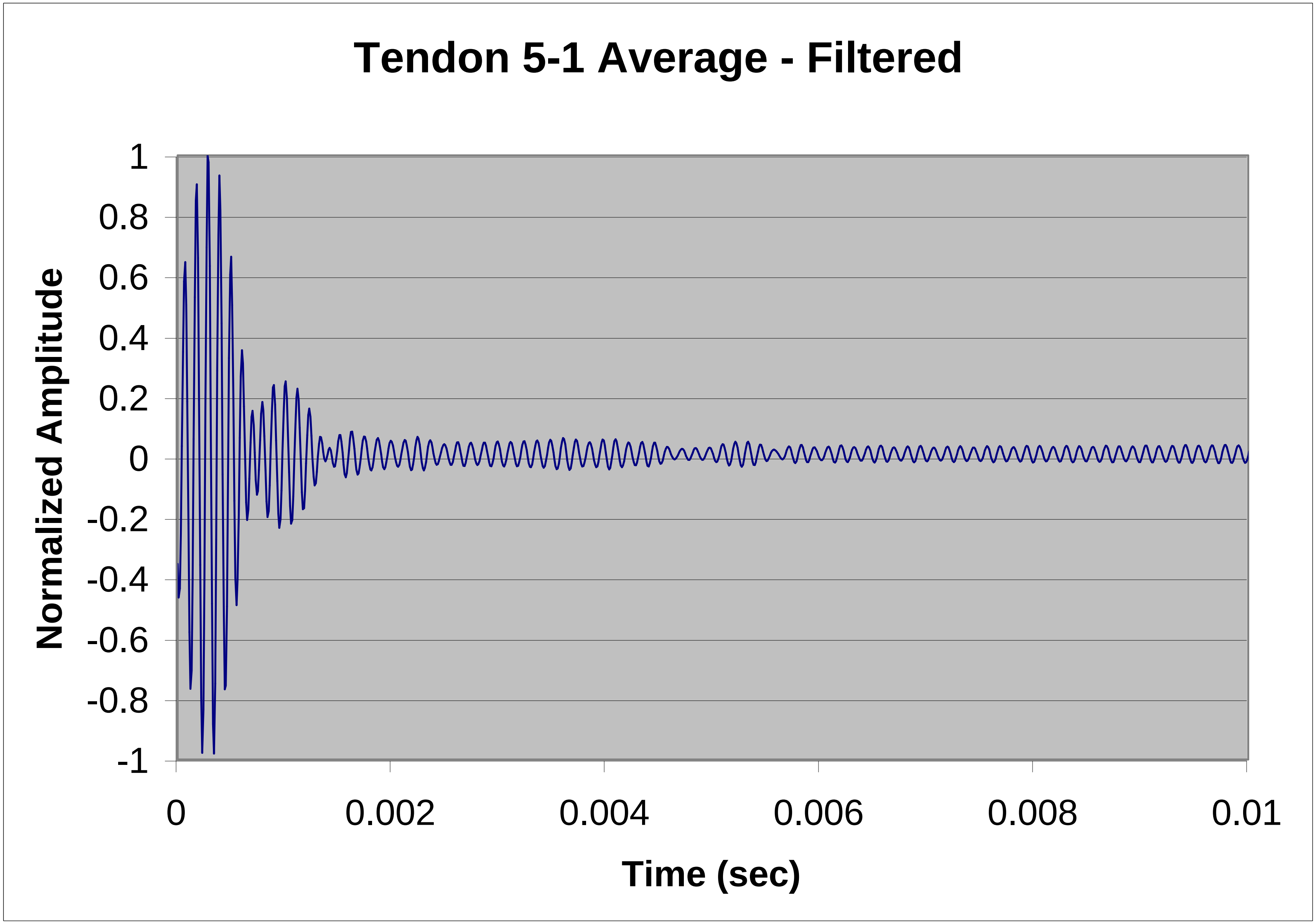
Tendon 4-1 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)


Tendon 4-2 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)

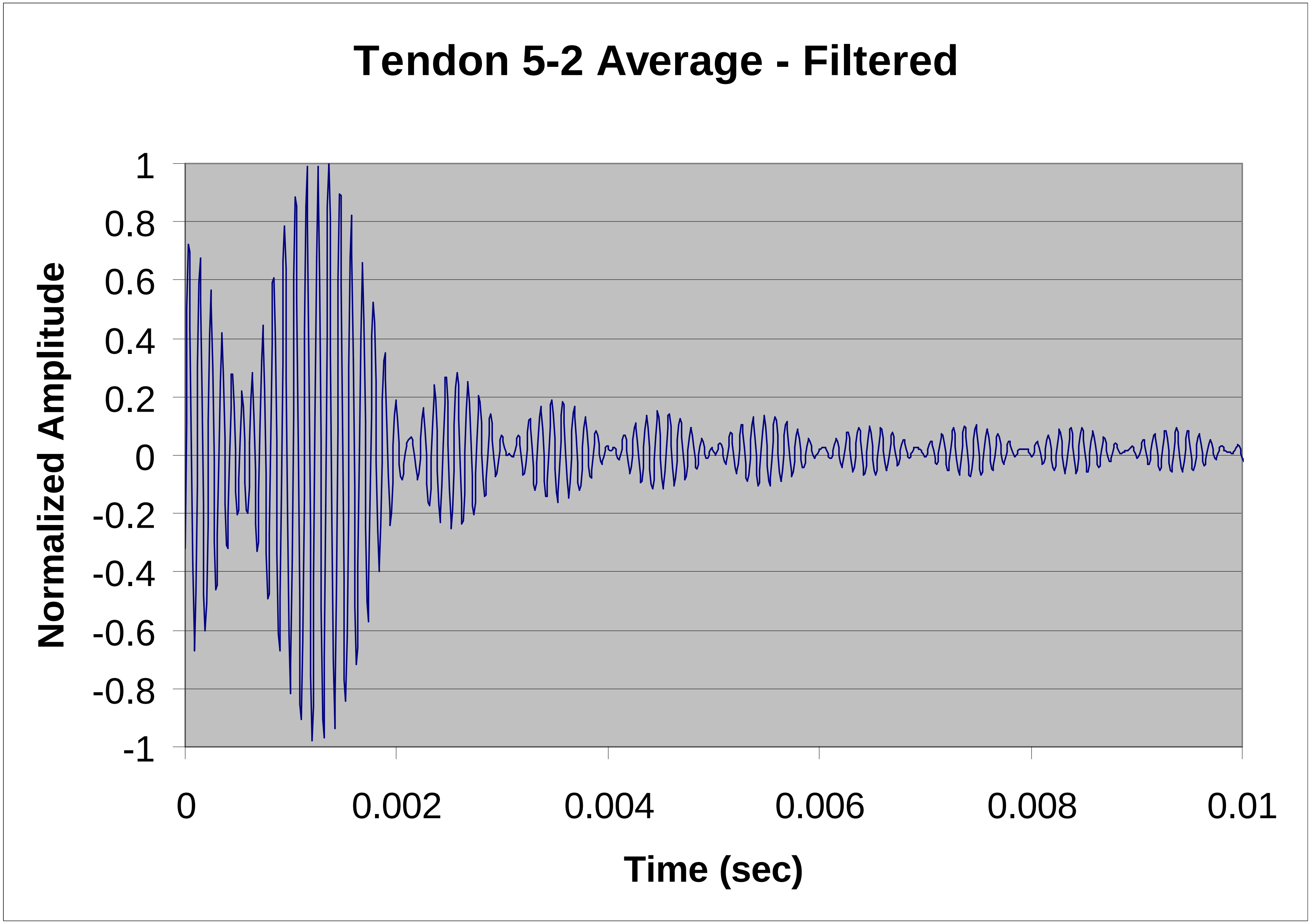


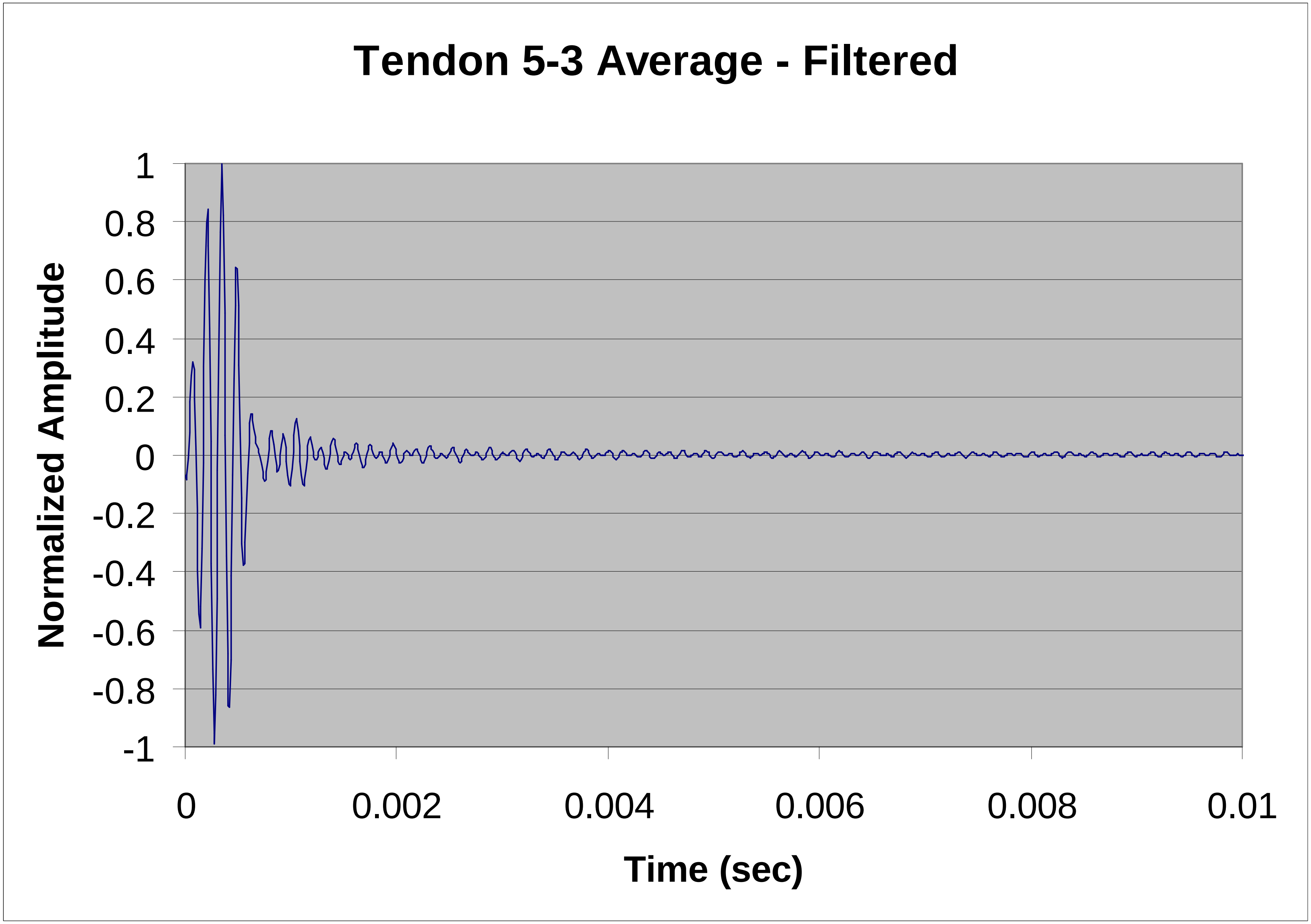
Tendon 4-3 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)

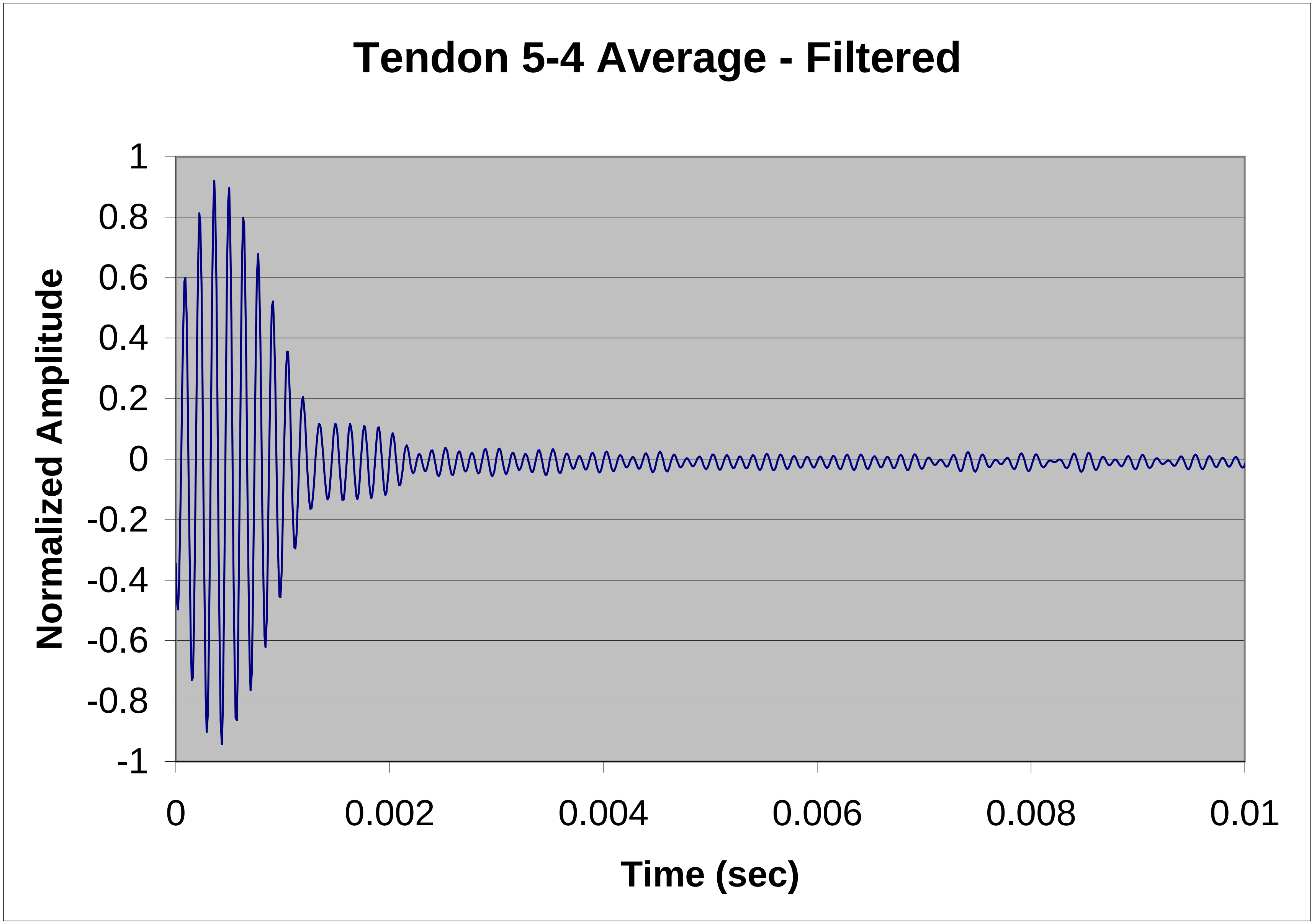

Tendon 4-4 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)



Tendon 5-1 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)


Tendon 5-2 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)

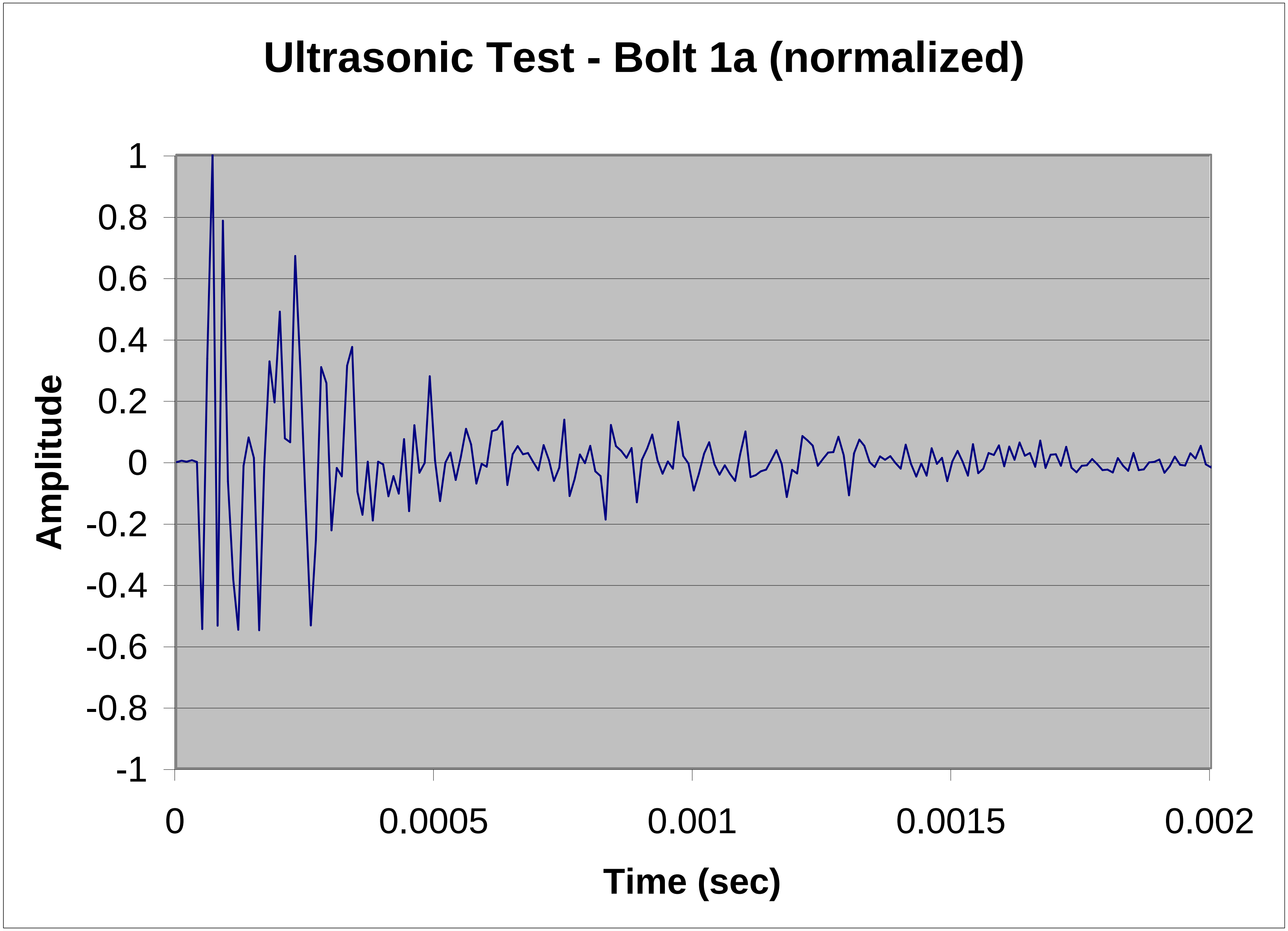

Tendon 5-3 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)



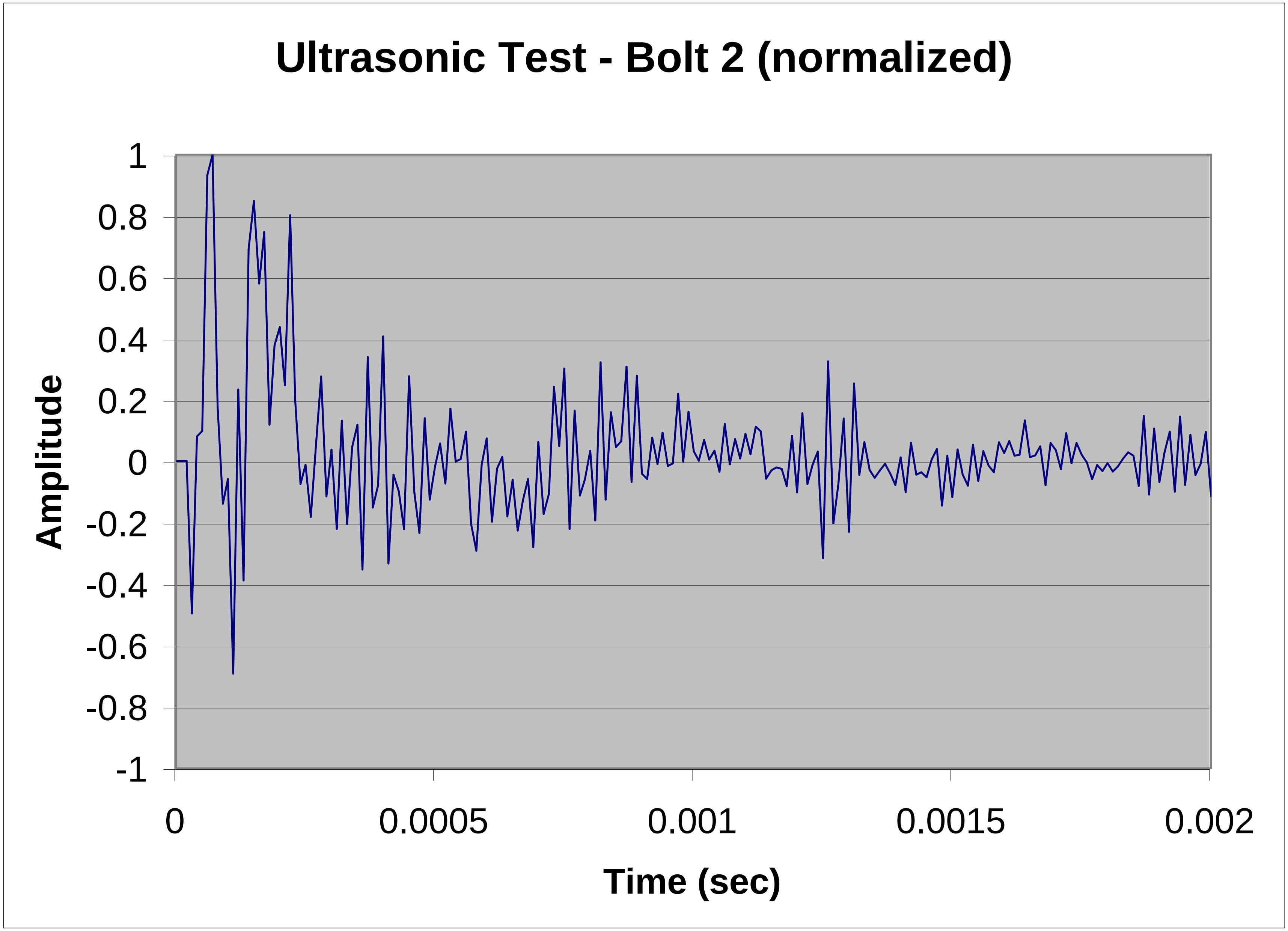
Tendon 5-4 Average - Filtered
Normalized Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.002
0.004
0.006
0.008
0.01
Time (sec)

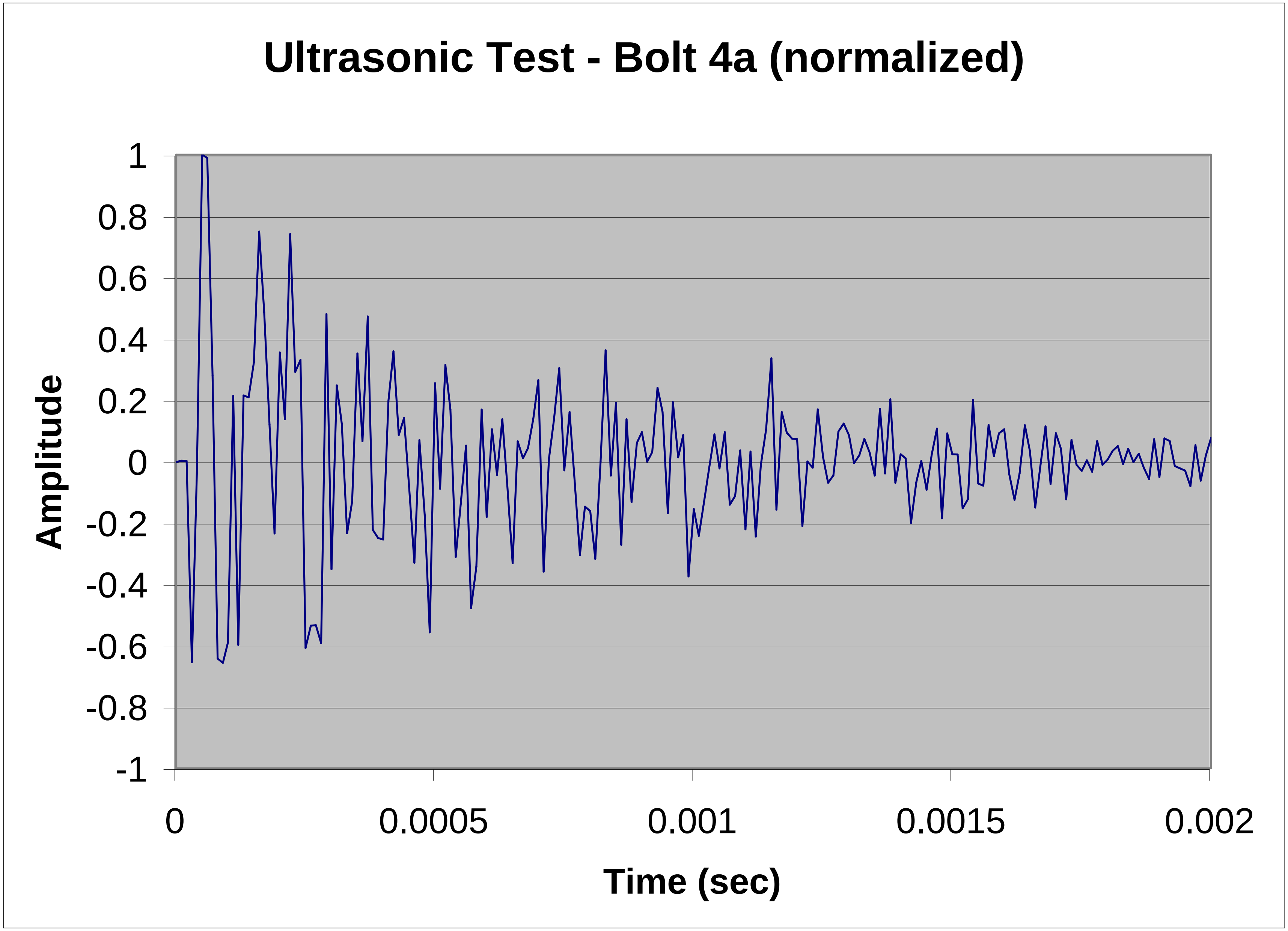
APPENDIX IX

ROCK BOLTS
ULTRASONIC TEST DATA

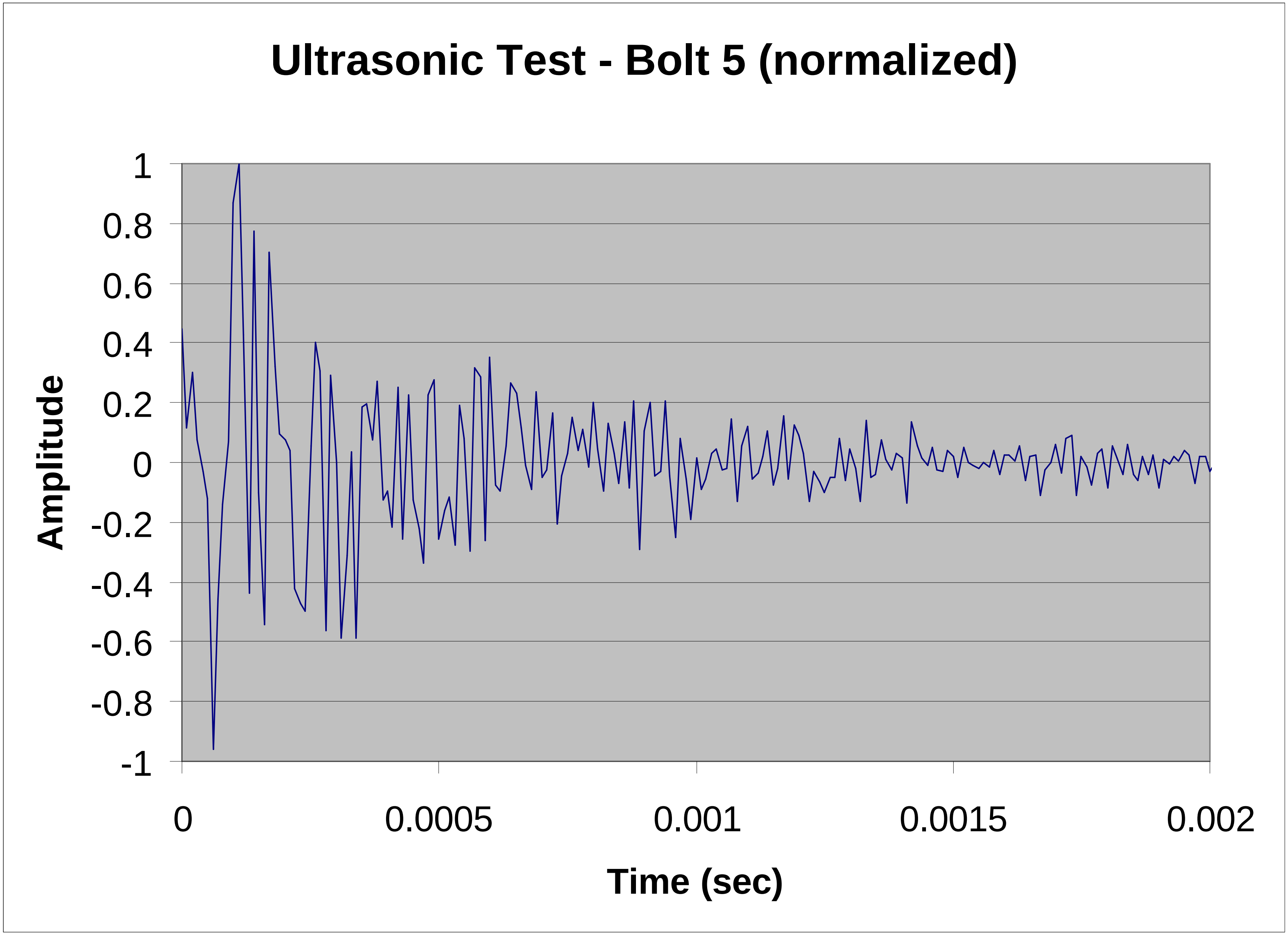


Ultrasonic Test - Bolt 1a (normalized)
Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.0005
0.001
0.0015
0.002
Time (sec)

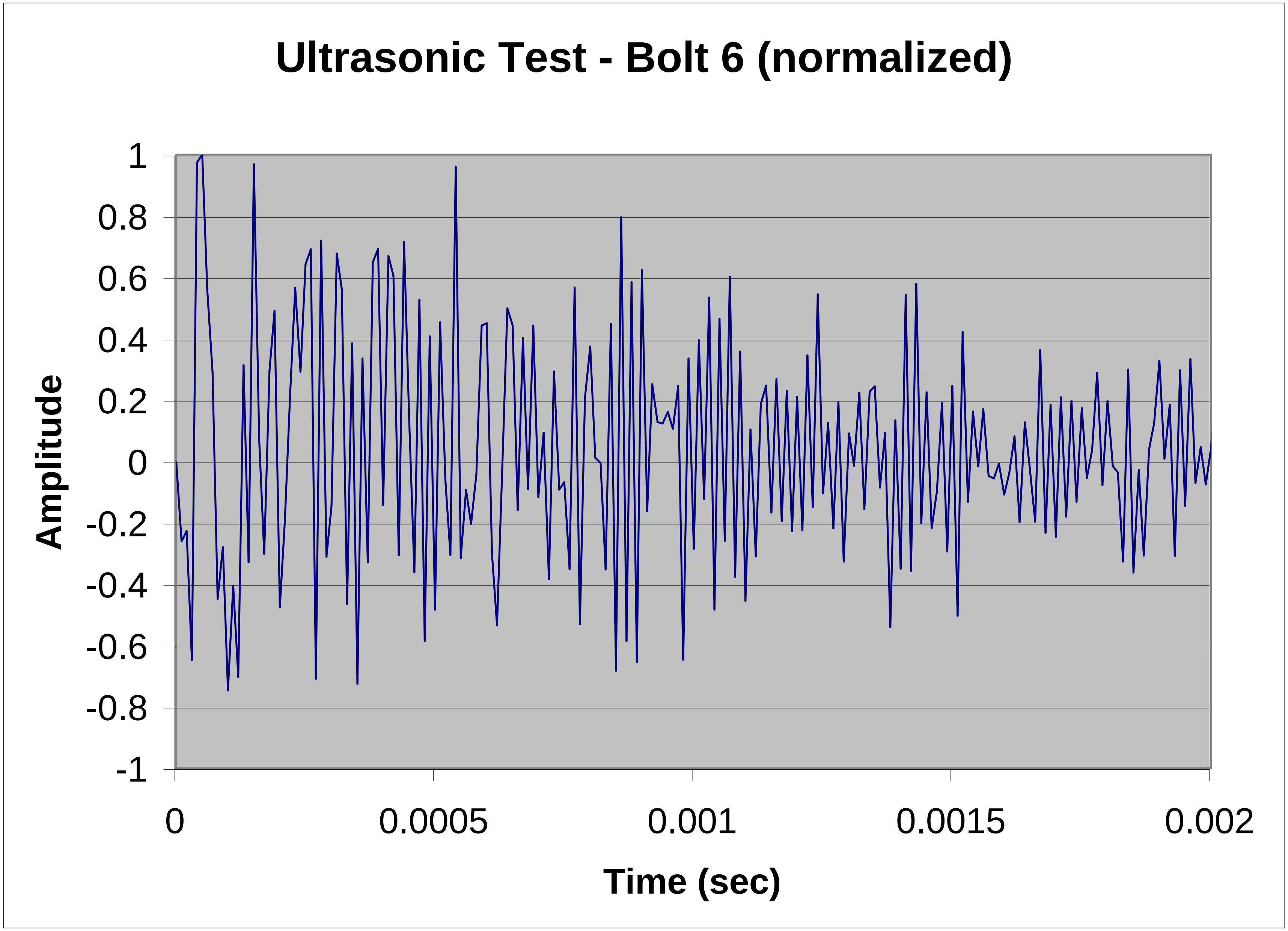

Ultrasonic Test - Bolt 2 (normalized)
Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.0005
0.001
0.0015
0.002
Time (sec)



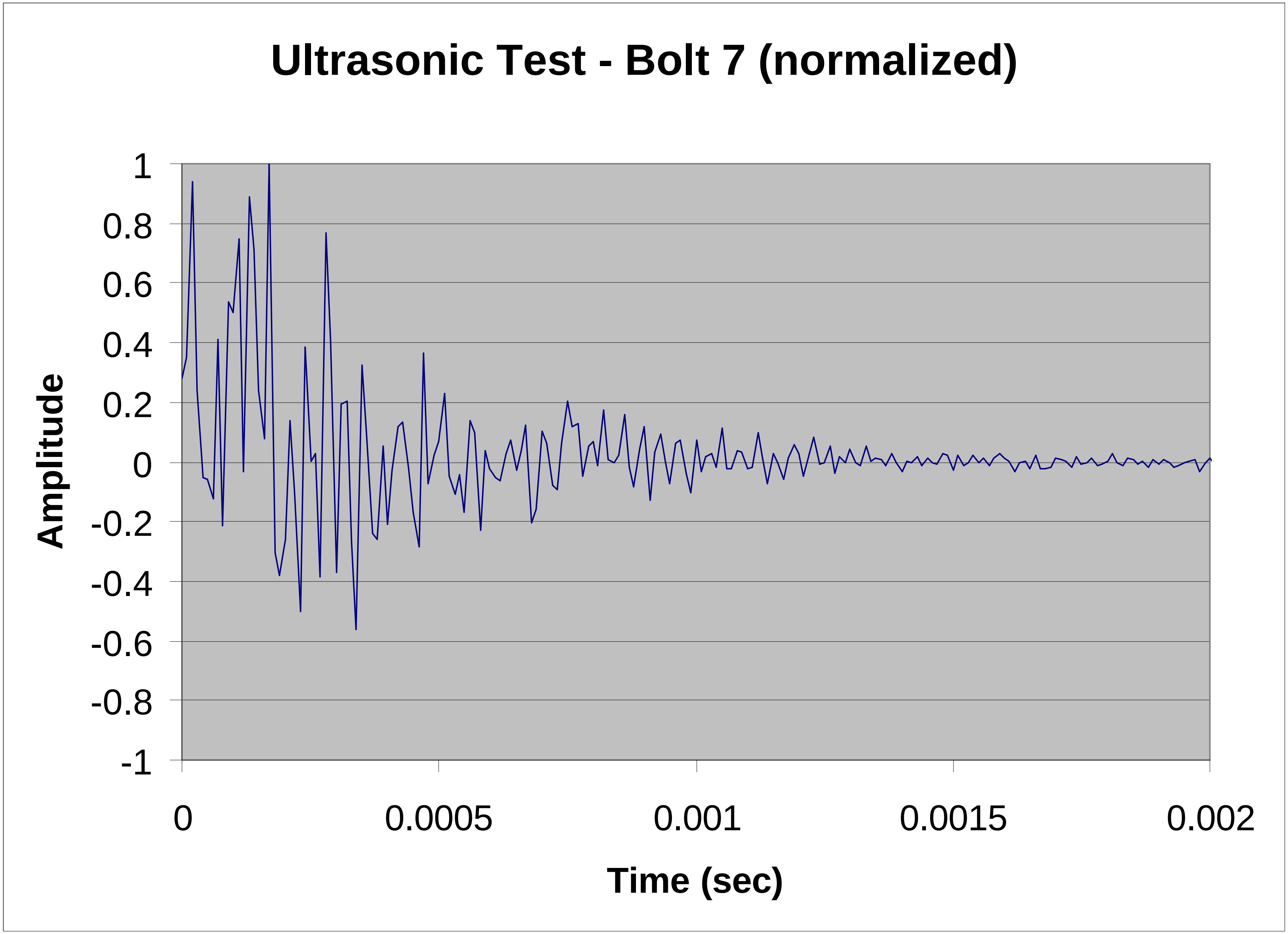
Ultrasonic Test - Bolt 4a (normalized)
Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.0005
0.001
0.0015
0.002
Time (sec)

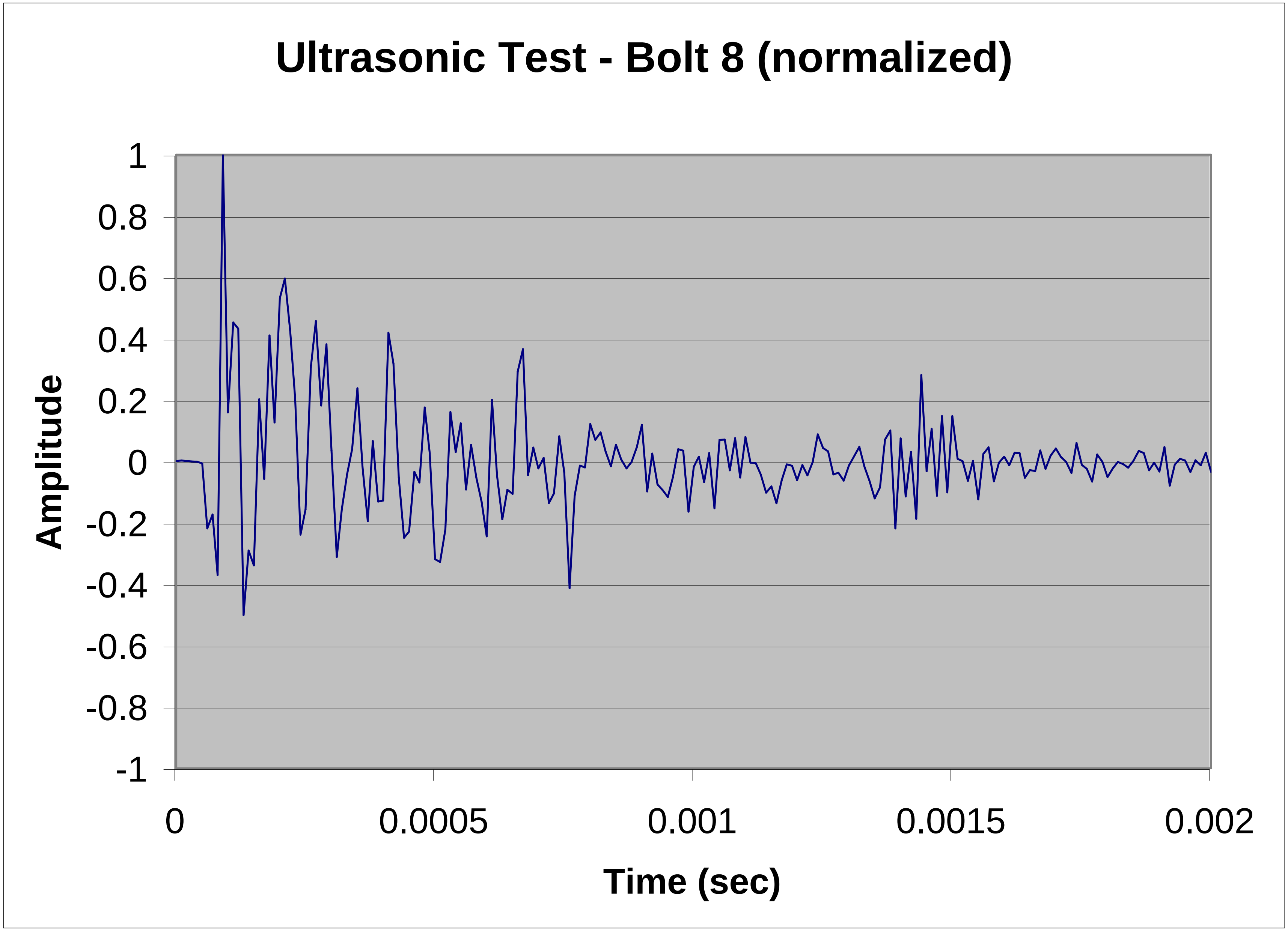


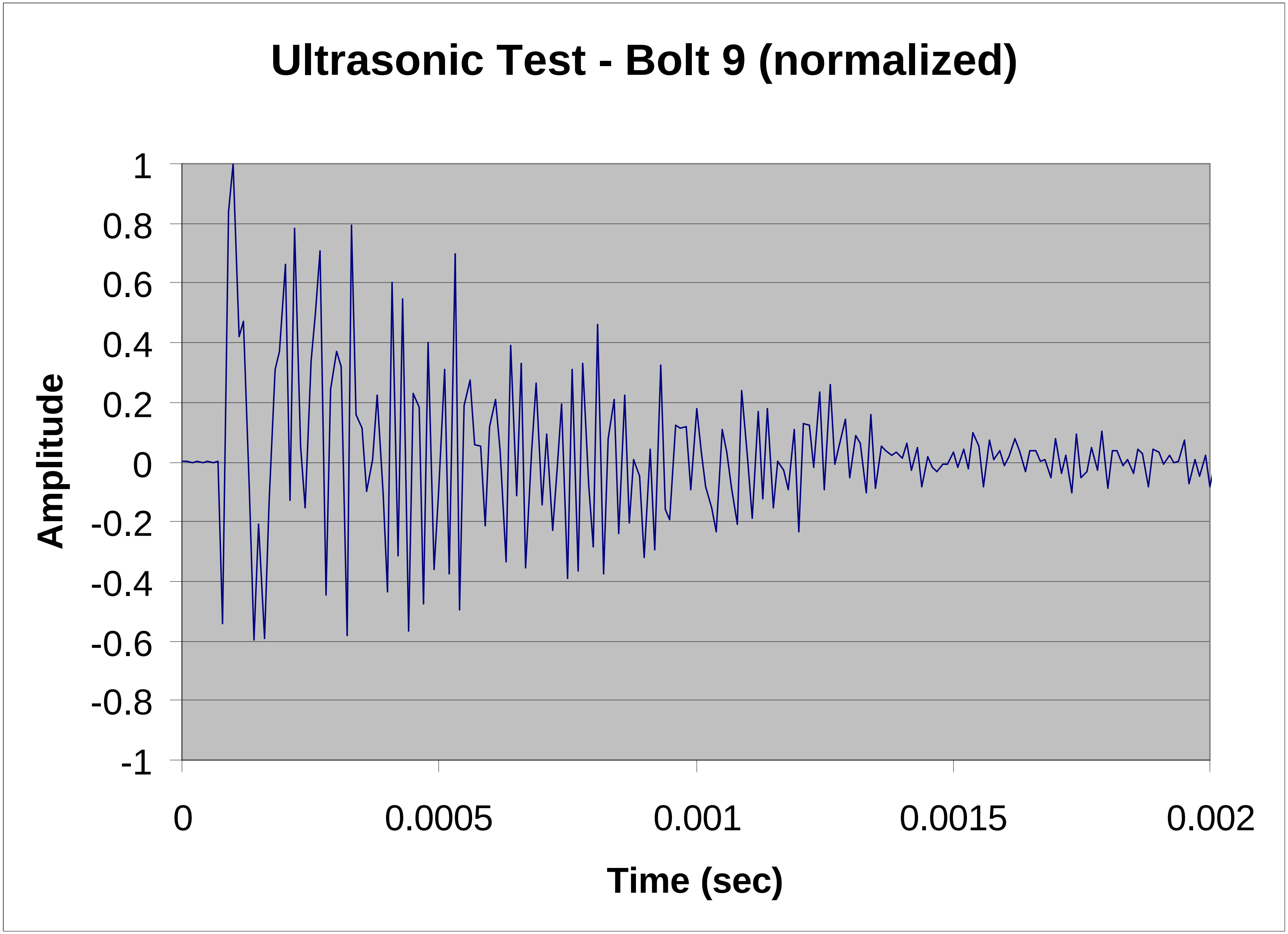
Ultrasonic Test - Bolt 5 (normalized)
Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.0005
0.001
0.0015
0.002
Time (sec)

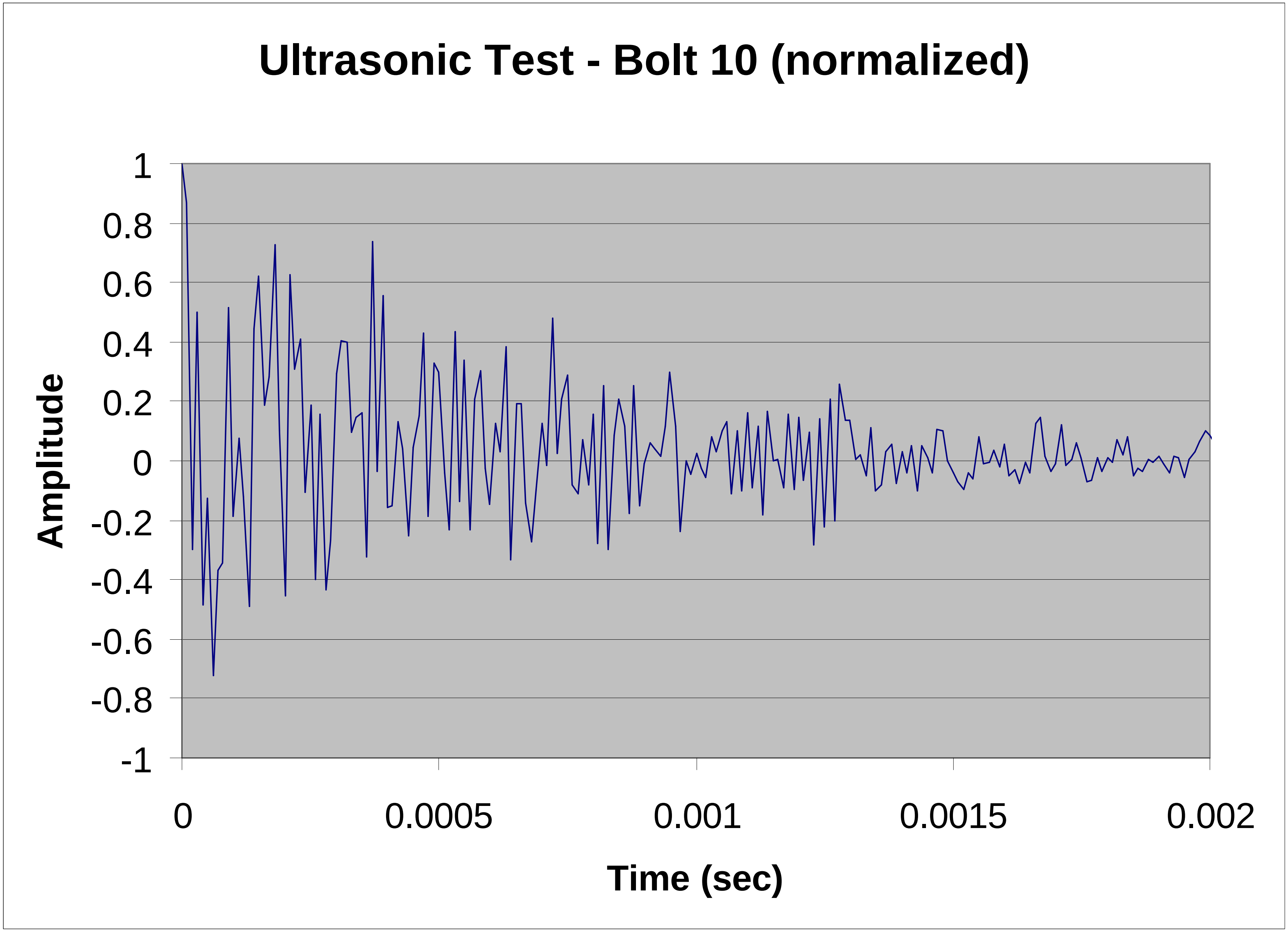


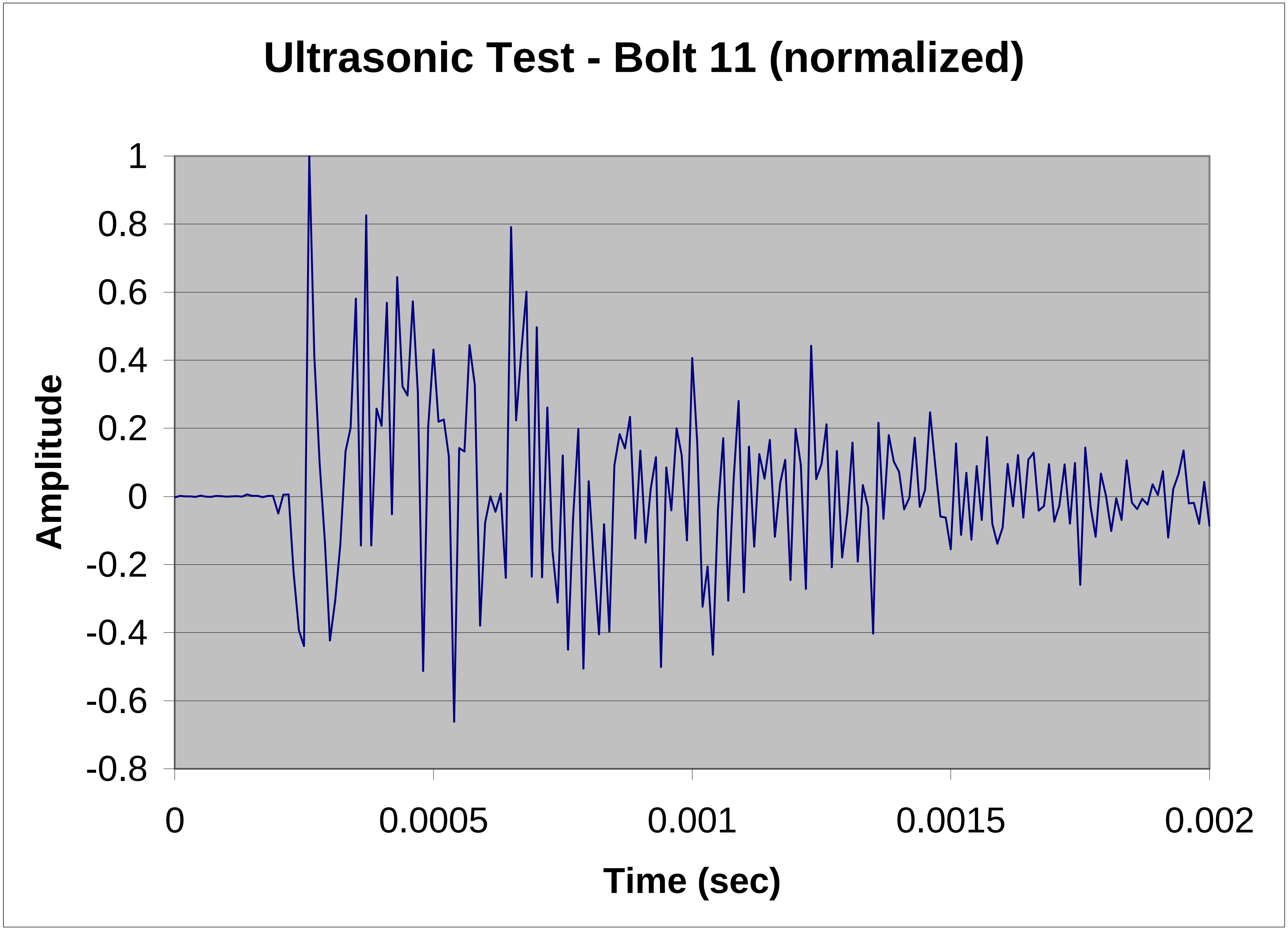
Ultrasonic Test - Bolt 6 (normalized)
Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.0005
0.001
0.0015
0.002
Time (sec)


Ultrasonic Test - Bolt 7 (normalized)
Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.0005
0.001
0.0015
0.002
Time (sec)

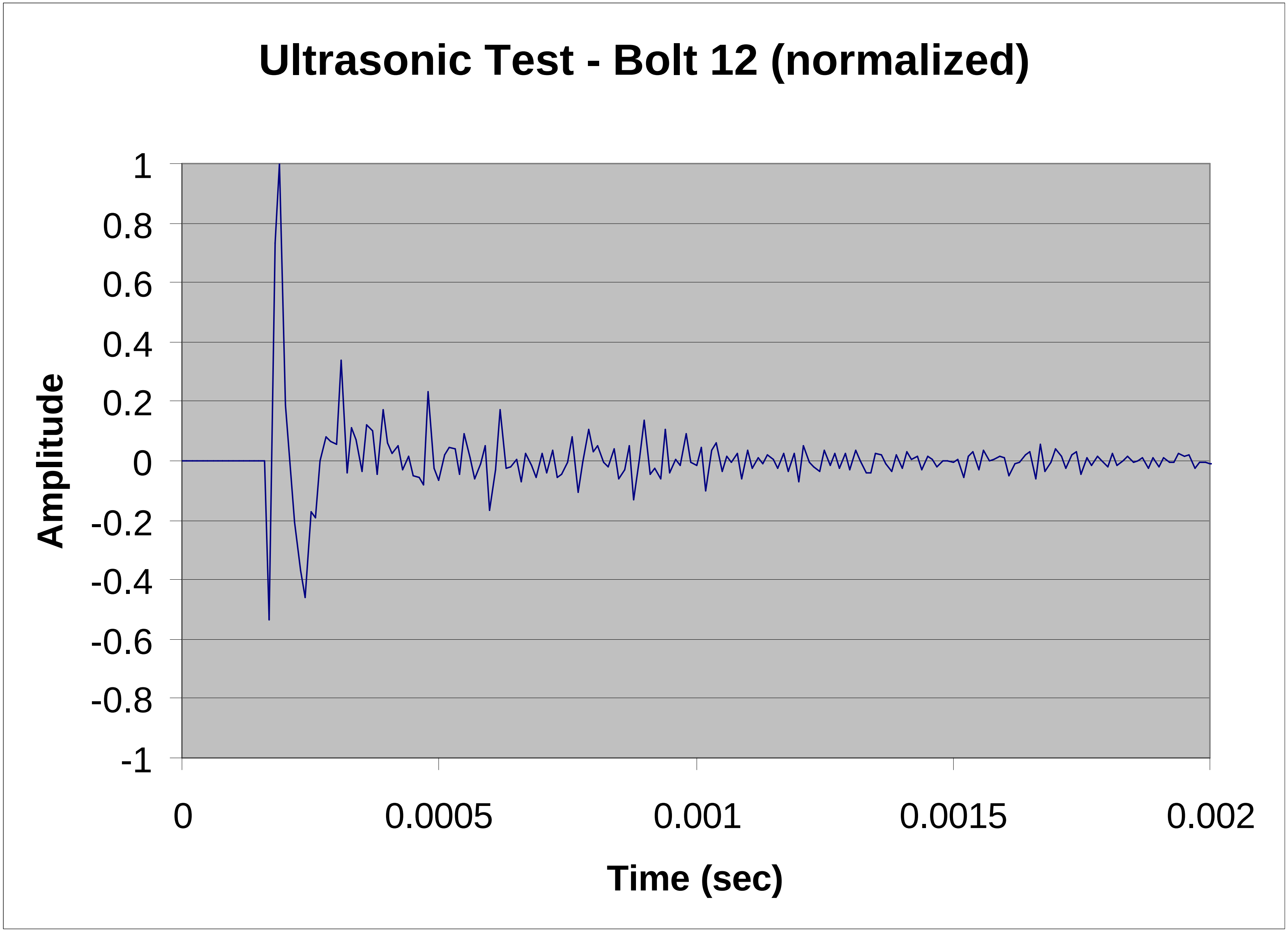

Ultrasonic Test - Bolt 8 (normalized)
Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.0005
0.001
0.0015
0.002
Time (sec)


Ultrasonic Test - Bolt 9 (normalized)
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
Amplitude
0
0.0005
0.001
0.0015
0.002
Time (sec)

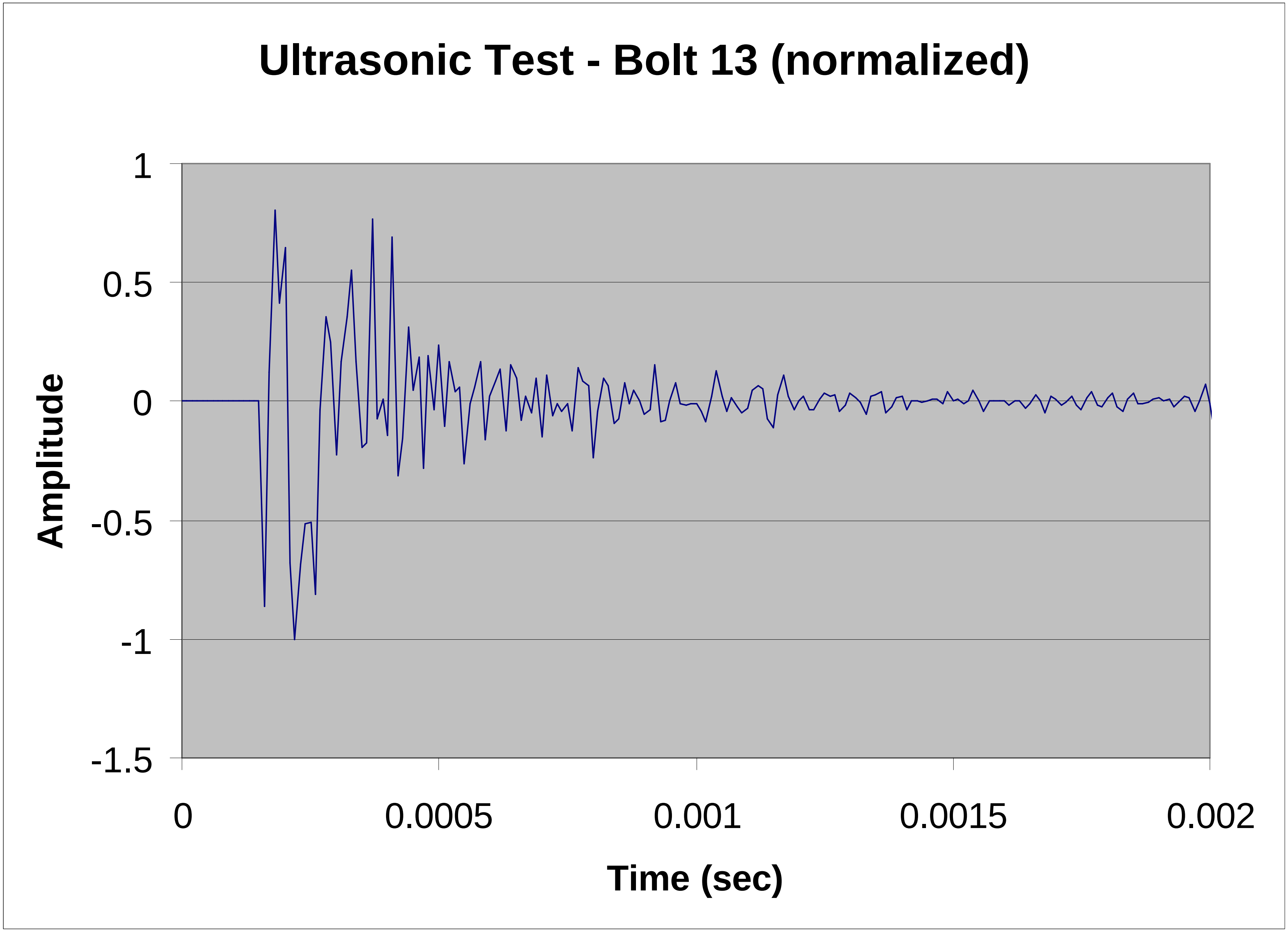

Ultrasonic Test - Bolt 10 (normalized)
Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.0005
0.001
0.0015
0.002
Time (sec)

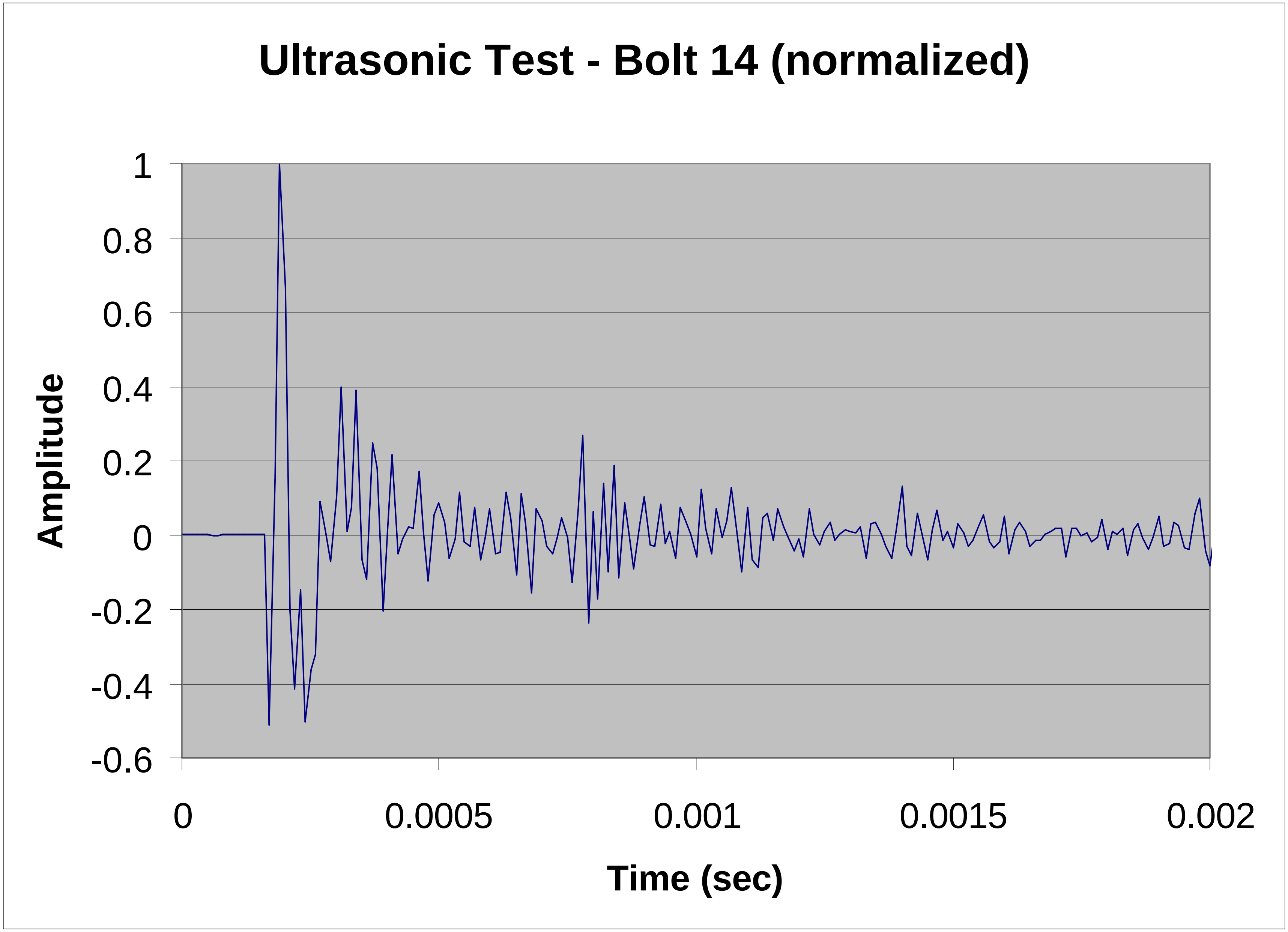


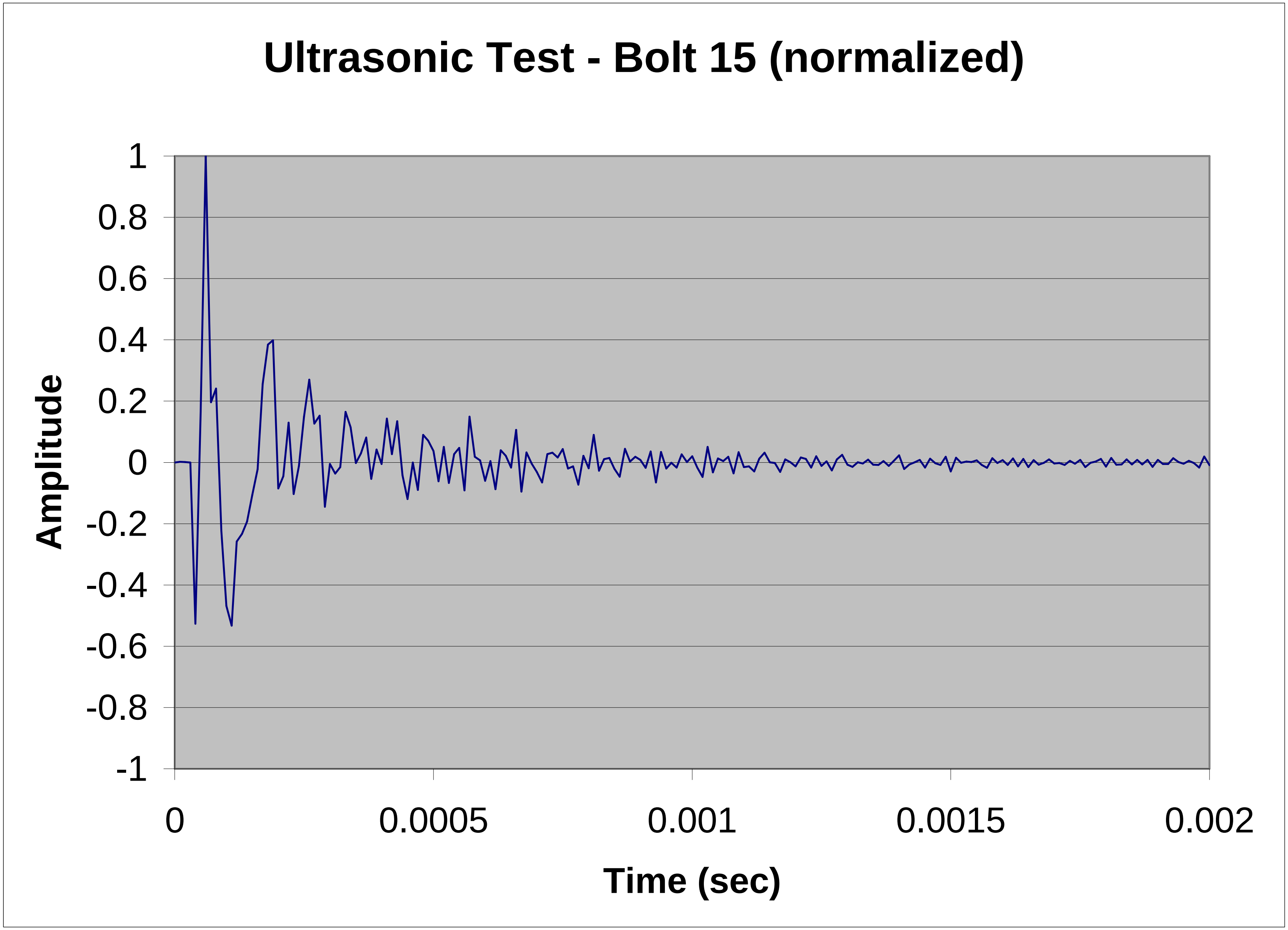
Ultrasonic Test - Bolt 11 (normalized)
Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
0
0.0005
0.001
0.0015
0.002
Time (sec)



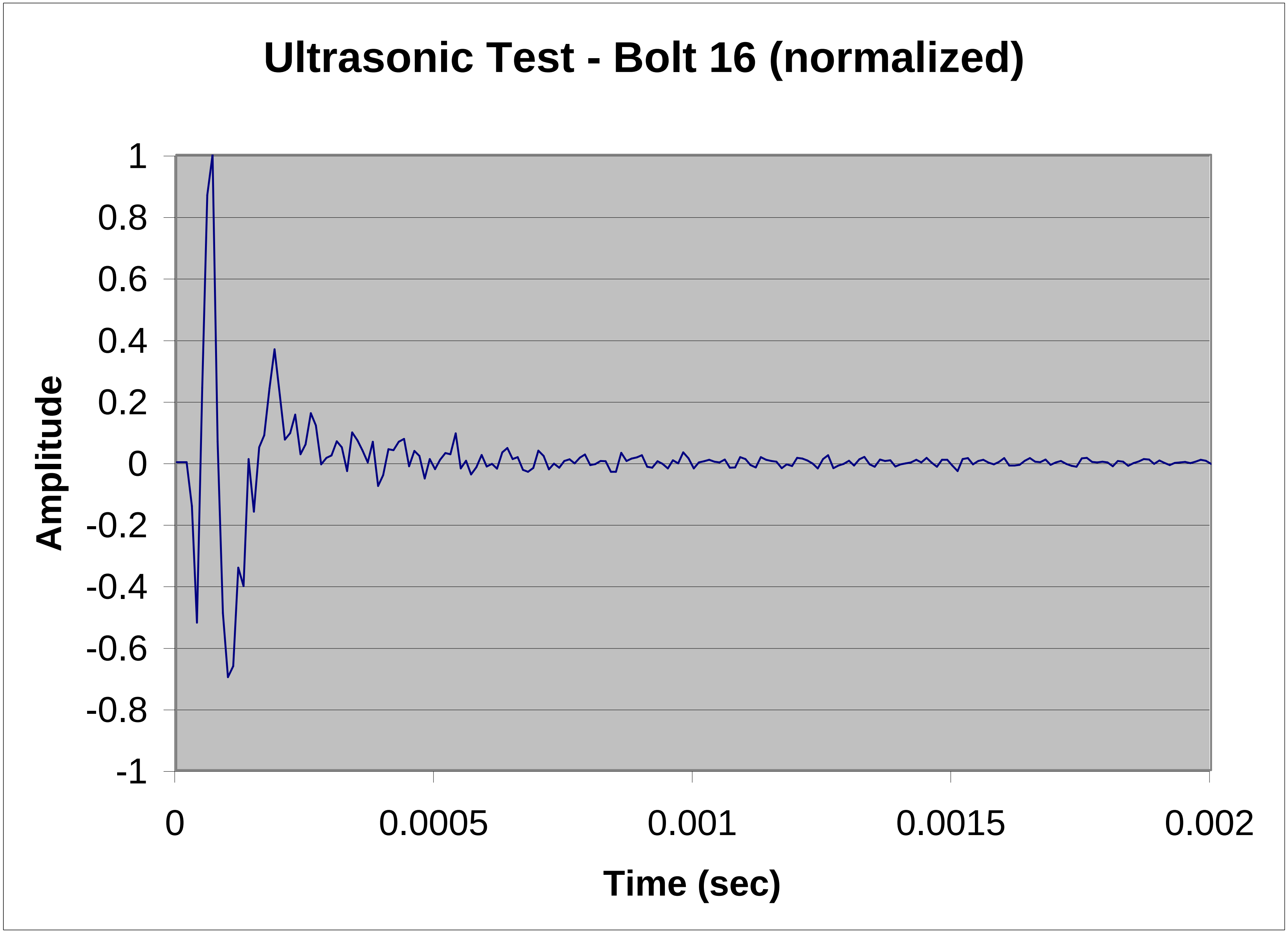
Ultrasonic Test - Bolt 12 (normalized)
Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.0005
0.001
0.0015
0.002
Time (sec)


Ultrasonic Test - Bolt 13 (normalized)
1
0.5
0
-0.5
-1
-1.5
Amplitude
0
0.0005
0.001
0.0015
0.002
Time (sec)

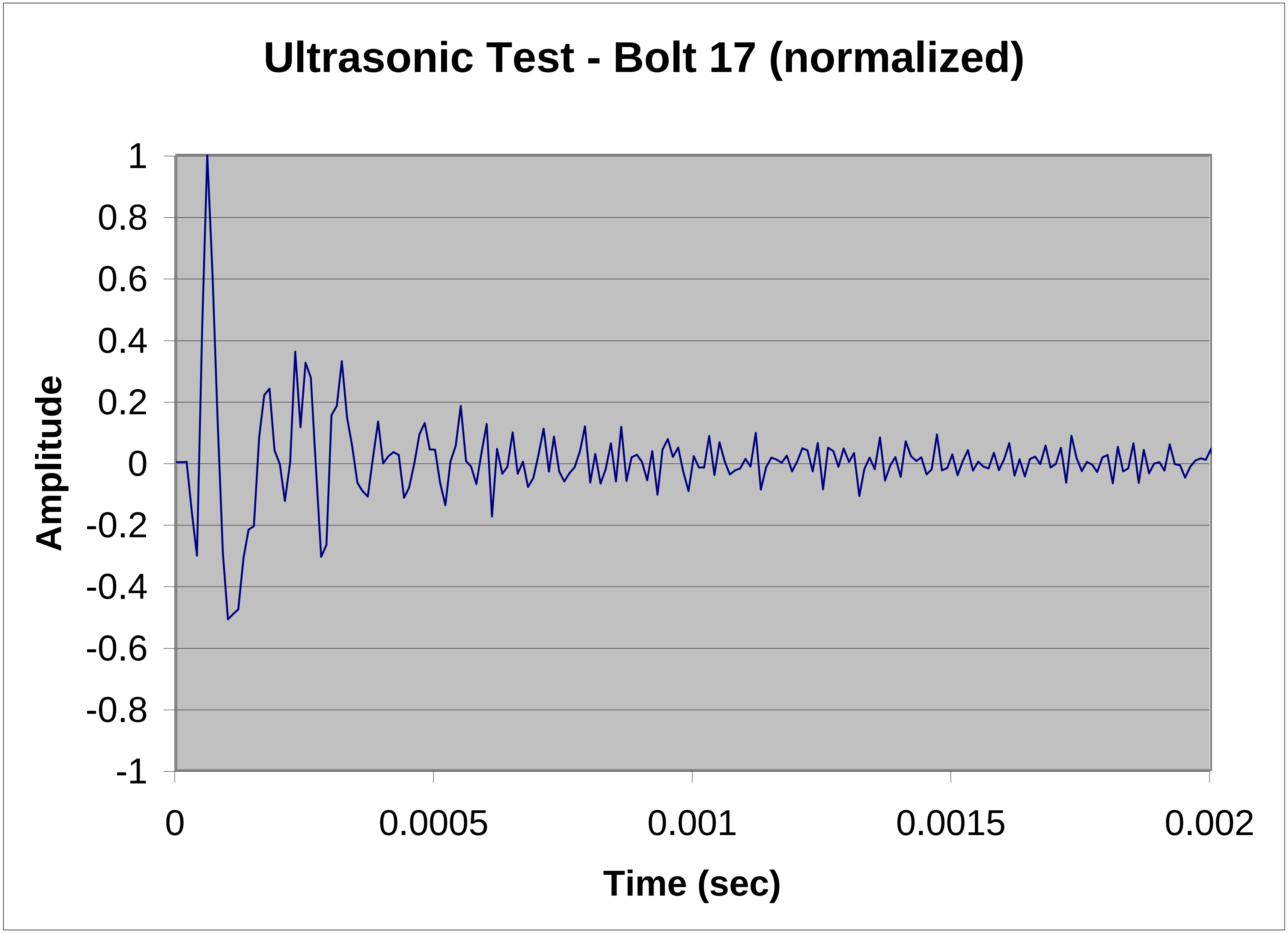

Ultrasonic Test - Bolt 14 (normalized)
Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
0
0.0005
0.001
0.0015
0.002
Time (sec)

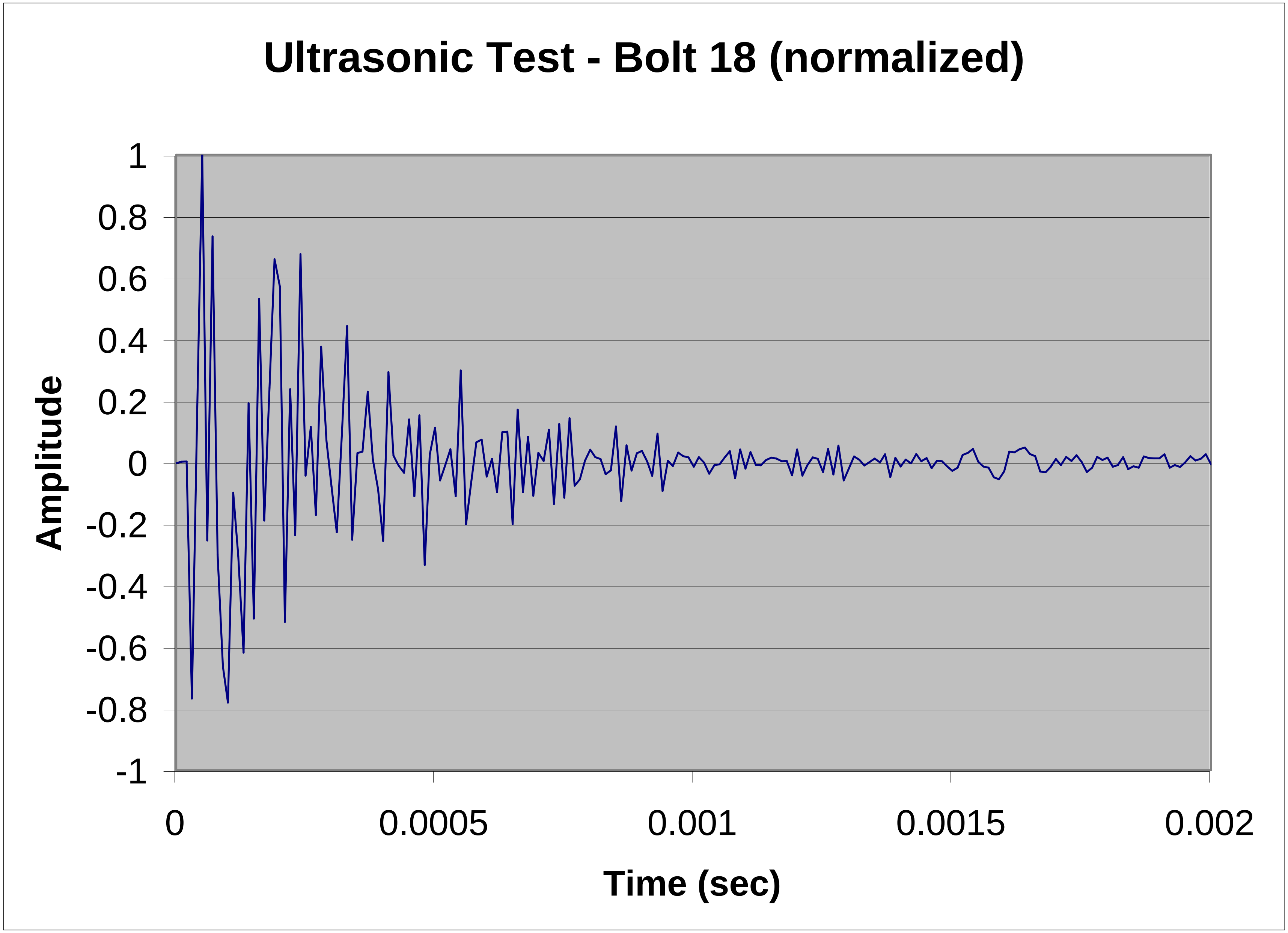


Ultrasonic Test - Bolt 15 (normalized)
Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.0005
0.001
0.0015
0.002
Time (sec)

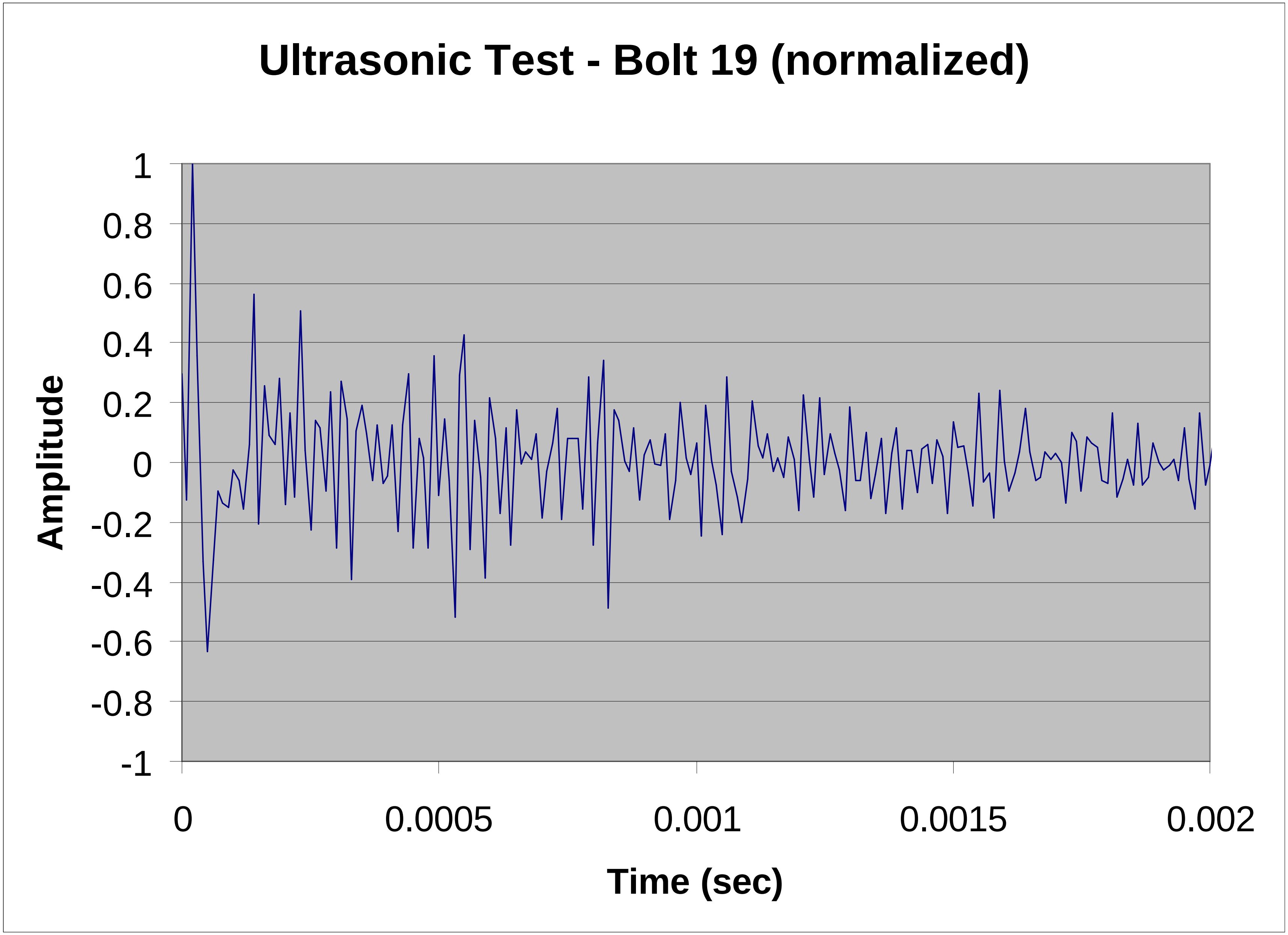


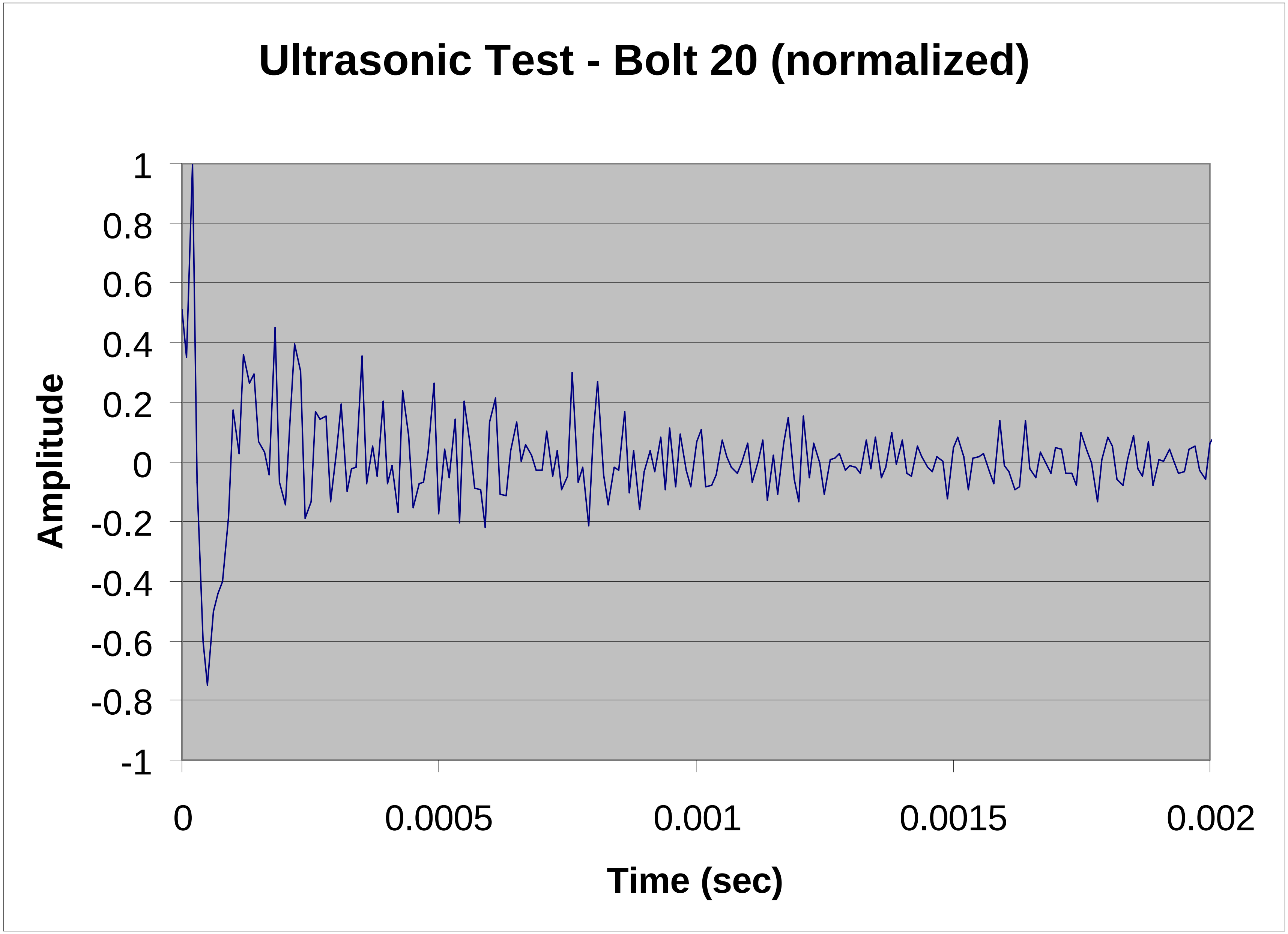
Ultrasonic Test - Bolt 16 (normalized)
Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.0005
0.001
0.0015
0.002
Time (sec)


Ultrasonic Test - Bolt 17 (normalized)
Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.0005
0.001
0.0015
0.002
Time (sec)



Ultrasonic Test - Bolt 18 (normalized)
Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.0005
0.001
0.0015
0.002
Time (sec)


Ultrasonic Test - Bolt 19 (normalized)
Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.0005
0.001
0.0015
0.002
Time (sec)


Ultrasonic Test - Bolt 20 (normalized)
Amplitude
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.0005
0.001
0.0015
0.002
Time (sec)