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DYNAMIC RESPONSE OF REINFORCED SOIL SYSTEMS: PHASE II

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13. ABSTRACT *(Maximum 200 words)*

Thirty-One 1:30 scale model reinforced soil wall structures were subjected to the dynamic effects of a detonation buried in the backfill behind each wall. The tests were conducted on the Wright Laboratory Geotechnical Centrifuge. The objective of the test was twofold: (i) to quantify the limits of random variation of test results obtained with the geotechnical centrifuge and, (ii) to investigate the effect of individual reinforced soil wall system components on overall wall system response to the blast loading. Each model was instrumented to record free field and wall panel pressures and acceleration data.

14. SUBJECT TERMS

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PREFACE

A. OBJECTIVE

The objective of this study was to investigate specific parameters governing the response of reinforced soil systems subjected to blast loading from a buried high explosive using a geotechnical centrifuge to test a model of a typical prototype wall. Tests evaluating the repeatability of the centrifuge experiment and then evaluating the effect of specific design parameters on reinforced soil structures were conducted. This study is part of a broader program to develop procedures for the design of a modular aircraft shelter using reinforced soil.

B. BACKGROUND

Phase I of this study investigated the feasibility of using a geotechnical centrifuge to model reinforced soil structures subject to blast loading. Based on limited testing, Phase I demonstrated a close agreement between centrifuge test results and numerical predictions, thus providing evidence of the appropriateness of the centrifuge modeling technique. A more comprehensive study of reinforced soil structures was required to identify key parameters contributing to the most stable wall system and provide results for comparison with numerical predictions made by others.

C. TEST DESCRIPTION AND SCOPE

Thirty-one 1:30 scale model reinforced soil wall structures were tested on the Wright Laboratory Geotechnical Centrifuge. The typical model measured 6-inches high by 20-inches long and was constructed using 2-inch by 2-inch wall facing panels, a nylon mesh soil reinforcing grid, and dry sand as the soil component. The model represented a conventional prototype reinforced soil wall measuring 15-feet high by 50-feet long. Each model was instrumented with pressure gages and accelerometers to record events during the blast loading. Wall panel deflections were measured at the end of each test.

The scope of the Phase II study was composed of three test series as described below.

- Conduct five preliminary geotechnical centrifuge tests to verify model construction techniques, test the instrumentation package, and test a new explosive device.

- Conduct ten reliability (repeatability) geotechnical centrifuge tests on a series of nominally identical reinforced soil wall models to quantify the limits of random variation of deflections, pressures, and accelerations when subjected to blast loading.

- Conduct sixteen production geotechnical centrifuge tests to measure the effects of various construction parameters on deflection, pressures and accelerations. The investigated parameters consisted of: soil density and type, panel geometry, reinforcement length, area of reinforcement coverage, use of a roof and overburden soil, weapon location, use of a berm to dissipate energy, and reinforcement strength.

D. CONCLUSIONS

The results of the reliability test series indicate that wall displacements, wave speeds, and crater dimensions are very reproducible in the geotechnical centrifuge, while peak pressures and accelerations are less reproducible.

The results of the production test series are summarized below.

- Soil density was the most significant factor affecting wall response. As soil density increased, wall panel displacements decreased.

- Reinforcement tensile strength significantly affected the wall response at the sides of the wall, but not as much as soil density.

- The addition or reduction of reinforcement surface area in the soil backfill over standard design guidelines does not significantly alter wall panel displacement.

- Facing panel geometry does not affect wall response.

- The soil type significantly affected peak pressure, but did not affect panel displacements.

- The addition of a roof resulted in significantly less panel displacements in the top row of facing panels only.

- The inclusion of a berm behind the wall significantly reduced panel displacements in the top row of panels only.

SECTION 1

INTRODUCTION

1.1 GENERAL

The United States Armed Forces require structures to resist the blast effects of conventional weapons. These blast-resistant structures are constructed to shelter aircraft, ammunition, or personnel. Current blast-resistant structures are typically constructed of heavily reinforced concrete, normally reinforced concrete with soil berms, or normally reinforced concrete constructed underground. However, these protective measures are expensive and time-consuming to construct, and may be sensitive to multiple strikes. For these reasons, the United States Air Force has funded a program to investigate the possibility of using reinforced soil systems to construct blast-resistant structures.

Over the past 2 years, extensive research has been conducted to investigate the response of reinforced soil walls subjected to blast loading. The research has consisted of numerical, geotechnical centrifuge, and full-scale modeling. The goal of this research effort is to assess the feasibility of using reinforced soil structures to resist blast loading, and ultimately to develop a standard design procedure which can be used to design blast-resistant reinforced soil structures.

The geotechnical centrifuge modeling portion of the study was conducted in two phases. This report presents only the findings of Phase 2 of the geotechnical centrifuge modeling program. A brief summary of the objectives and findings of the Phase 1 study is presented below.

1.2 SUMMARY OF PHASE 1 STUDY

The overall objective of the Phase 1 study was to evaluate the response of reinforced soil wall systems subjected to blast loading, and to assess the feasibility of using reinforced soil systems to provide blast resistance. These objectives were achieved by: (i) conducting an extensive review of existing technical literature to evaluate information on the response of soils and the response of reinforced soil systems to blast loading; (ii) developing a laboratory testing apparatus and executing a test program to study the dynamic response of soil reinforcement subjected to impulse loading; (iii) developing a numerical model for the analysis of reinforced soil wall systems subjected to blast loading; and (iv) conducting physical modeling of reinforced soil wall systems subjected to blast loading in a geotechnical centrifuge. This review of the Phase 1 study covers only the geotechnical centrifuge modeling.

The main objectives of the Phase 1 geotechnical centrifuge modeling were to validate the use of the geotechnical centrifuge as a means of studying dynamically loaded reinforced soil structures, and to provide data which could be compared to results obtained from numerical models and full scale tests. Nine geotechnical centrifuge tests were conducted during the Phase 1 study. Peak pressures, accelerations, and wall displacements were measured for each test. The study investigated the effects of several different parameters, including reinforcement type (i.e., steel strips versus geogrid), reinforcement length and width, and overburden pressure on the reinforced soil wall. The results of these tests were compared to each other, to predictions made with existing structural numerical models, to results obtained from numerical modeling using a soil cap model, and to results of full scale tests. Based on analysis of the collected centrifuge data, the following key conclusions were drawn:

- based on the limited testing conducted, the geotechnical centrifuge tests appeared to be reproducible;

- reinforcement type appeared to be a significant factor in reinforced soil wall deformation;

- the effects of blast location on wall response in the geotechnical centrifuge were similar to those seen in the full-scale testing;

- peak pressures, wave velocities, panel displacements, and the effects of wall response to detonator location in the geotechnical centrifuge tests agreed well with the numerical modeling results, while peak panel accelerations did not; and

- peak pressures measured in the geotechnical centrifuge tests agreed well with those predicted with existing structural numerical models.

Complete documentation of the Phase 1 test procedures and results is presented in Bachus et al.

(Reference 1).

1.3 GOALS OF PHASE 2 STUDY

The results of the physical modeling portion of the Phase 1 study indicated that the geotechnical centrifuge can be a valuable tool in assessing the response of reinforced soil walls to blast loading. However, the results obtained in Phase 1 were based on a limited number of tests. In the Phase 2 study, an extensive testing program was developed with the following goals: (i) to further investigate the ability of the geotechnical centrifuge to reproduce data when nominally identical tests are conducted (i.e., to determine the reliability of the geotechnical centrifuge model tests), and (ii) to further investigate the effects of individual reinforced soil wall system components on the overall response of the wall to blast loading. These goals were achieved by conducting two test series in the geotechnical centrifuge. The first, called the reliability test series, investigated the ability of the geotechnical centrifuge to reproduce test data and to quantify the limits of random variation of wall response for nominally identical test models. The second, called the production test series, consisted of a parametric study of reinforced soil wall system components in order to identify key parameters which influence wall response. The results obtained from the Phase 2 study will be used in the development and verification of a design procedure for blast-protective reinforced soil structures.

1.4 ORGANIZATION

The remainder of this report is organized as follows:

- A description of the geotechnical centrifuge model testing program is presented in Section 2.
- Descriptions of the model development methodology and model construction technique are presented in Section 3.
- The preliminary test program and results are presented in Section 4.
- The reliability test program and results are presented in Section 5.

- The production test program and results are presented in Section 6.

- A summary and conclusions of the geotechnical centrifuge modeling program are presented in Section 7.

- Recommendations for further geotechnical centrifuge model testing are presented in Section 8.

SECTION 2

TEST PROGRAM

2.1 GENERAL

Geotechnical centrifuge test results were used to qualitatively identify trends that could be applied to the design of a full-scale structure. The tests were not intended to directly model any particular prototype design. For example, if geotechnical centrifuge modeling results indicate that peak blast pressures at the model wall facing panels tend to increase with increasing backfill soil density, it is expected that this trend will also occur in a prototype system. To accomplish the goals of the Phase 2 study, the test program was executed in three stages: preliminary testing, reliability testing, and production testing.

2.2 CENTRIFUGE DESCRIPTION

All model tests were conducted on the Wright Laboratory geotechnical centrifuge at Tyndall AFB, Florida (see Figure 1). The Genisco Model E-185 centrifuge is a hydraulically-driven rotary accelerator with a payload capacity of up to 500 pounds and maximum g-level of up to 100 g. Payload and g-level must be selected so as not to exceed the maximum in-flight capacity of 15 g-ton at the sample mounting platform. For example, a payload of 500 pounds cannot be accelerated to the full 100 g, but is limited to a maximum of 60 g. The mounting platforms are located at the ends of two 6-foot cantilever arms, and are free to rotate from a horizontal (i.e., parallel to the length of the cantilever arm) to vertical (i.e., perpendicular to the length of the cantilever arm) position, so that the resultant force on the bucket will be radially inward toward the axis of rotation.

The geotechnical centrifuge is housed in a 0.75-feet thick reinforced concrete structure with a height of 6.9 feet and a diameter of 16 feet. It is operated via a remote control console. Additional features include an on-board 16-channel Pacific Instruments Model 5700 Transient Data Recorder (TDR) data acquisition system, shuttered video camera, and 10,000 picture-per-second high-speed camera.

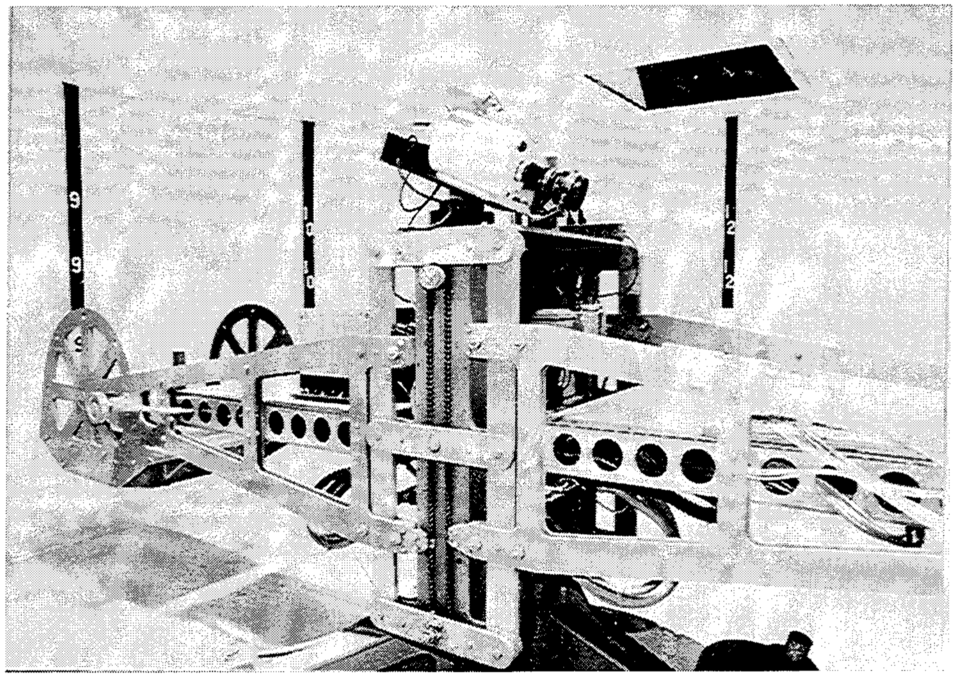


Figure 1. Wright Laboratory Geotechnical Centrifuge

2.3 TEST SERIES

2.3.1 General

Thirty-one tests were conducted, in which small-scale reinforced soil walls were constructed, instrumented, and subjected to blast loading in the geotechnical centrifuge by means of a detonator buried in the soil behind the wall. Collected data consisted of the following:

- wall panel displacements;
- pressure-time histories at the soil-facing panel interfaces;
- pressure-time histories in the free field (i.e., in the backfill soil behind the reinforced soil wall);
- pressure-time histories at the side wall of the centrifuge sample container;

- acceleration-time histories at the wall facing panels;
- acceleration-time histories in the free field;
- blast wave arrival times; and
- crater dimensions.

2.3.2 Preliminary Testing

Five preliminary tests were conducted. The purpose of these tests was to verify the design of the model detonator, assess the accuracy of the instrumentation, and develop a reliable and reproducible wall construction technique. The results of the preliminary tests are presented in Section 4.

2.3.3 Reliability Testing

Ten reliability tests were conducted, in which 10 models with nominally identical construction parameters were tested in the geotechnical centrifuge. The purpose of these tests was to quantify the limits of random variation of wall response to blast loading resulting from slight but unavoidable variations in wall construction, detonator response, instrumentation placement, and instrumentation reliability. The model parameters used in these tests also served as the baseline criteria for the production tests. The results of the reliability tests are presented in Section 5.

The test sample intended to be the first reliability test was not included in this study. During the centrifuge test the firing system failed to detonate the buried charge. The test sample was subsequently accelerated and decelerated several times before the problem was identified and repaired. During this time, the model wall underwent visible deformation due to vibrations during repair and cycles of gravity loading and unloading. Prior to detonating the charge it was agreed to disregard any data from this test. Therefore, although 32 tests were actually conducted, data are reported for 31 tests only.

2.3.4 Production Testing

Sixteen production tests were conducted. The purpose of these tests was to investigate the influence of individual reinforced soil wall system parameters on overall wall response. The parameters investigated in this study were:

- facing panel geometry;
- reinforcement coverage area;
- backfill soil density and type;
- overall system geometry; and
- reinforcement strength.

The results of the production tests are presented in Section 6.

SECTION 3

MODEL DEVELOPMENT

3.1 MODEL DESIGN

3.1.1 General

For each test a 1:30 scale (nominal) model reinforced soil wall, 6-inches high by 20-inches long, was constructed, instrumented and subjected to an explosive charge buried in the backfill behind the wall. The models were accelerated in the geotechnical centrifuge to 30 g (nominal) so that they simulated a 15-foot high by 50-foot long reinforced soil wall subjected to the buried blast from a MK82-500 pound general purpose bomb containing 192 pounds of H6 explosive (Reference 3). The replica scaling relationships used in the design of the models are presented in Table 1 (Reference 4). The development of the model reinforced soil wall system is presented below.

TABLE 1. TYPICAL REPLICA SCALING RELATIONS (Reference 4)

Quantity	Full Scale (prototype)	Model (at g-level n)
Linear Dimension	1	$1/n$
Area	1	$1/n^2$
Volume	1	$1/n^3$
Time	1	$1/n$
Velocity	1	1
Acceleration	1	n
Mass	1	$1/n^3$
Force	1	$1/n^2$
Energy	1	$1/n^3$
Stress	1	1
Strain	1	1
Density	1	1
Frequency	1	n

3.1.2 Facing Panels

The goal of any blast protective shelter is for its contents to survive the effects of detonation of a nearby weapon. The success or failure of such a shelter is determined by the degree of wall spalling, and the wall geometry after weapon detonation. Wall spalling is a function of wall rigidity. A reinforced soil wall is made up of individual facing panels free to move independently, and therefore is a flexible system when compared to a continuously reinforced concrete wall. The Phase 1 testing program (Reference 1) and full-scale tests conducted in 1990 (Reference 5) showed the explosive energy from the weapon displaces the facing panels, and the developed tensile stresses do not exceed the tensile strength of the concrete. The Phase 1 and full-scale testing also showed that bending stiffness of the panels was not a significant factor in wall performance; failures were due primarily to excessive displacement, rather than panel cracking. Therefore, the wall response modeled in this study was displacement.

Reinforced concrete is the most common material used in full-scale reinforced soil wall facing panels. For this study, a generic prototype reinforced concrete facing panel 5-feet long by 5-feet high by 0.5-feet thick was modeled in the geotechnical centrifuge. Exact simulation of this facing panel could be achieved in the geotechnical centrifuge at 30 g with 2-inch long by 2-inch high by 0.2-inch thick reinforced concrete panels. However, construction of such small panels using reinforced concrete is not feasible, and a substitute material was selected.

The material selected for the model facing panels was EXTREN 625, a lightweight, high-strength, composite material made of pultruded vinyl ester resin and fiberglass reinforcement. The model panels were designed to simulate the mass of a 5-foot long by 5-foot high by 0.5-foot thick concrete panel. The unit weights of concrete and EXTREN 625 are not equal (150 and 111 pounds per cubic foot, respectively). Therefore, the scaling was accomplished by modeling the generic prototype panel length and height, and then calculating the required model panel thickness so that the panel mass per unit of face area scaled correctly. The replica scaling equation for this calculation is:

$$\rho_m \, t_m = \frac{\rho_p \, t_p}{n}$$

where:

t_m = model panel thickness

n = g-level

t_p = prototype panel thickness

ρ_p = mass density of prototype material

ρ_m = mass density of model material

The calculated required model panel thickness is 0.27 inch. The actual sheet thickness of EXTREN is 0.253 inches. The model panel mass using this thickness is only 6 percent less than the mass required to simulate the generic prototype panel, and is considered to be a negligible difference.

One-eighth inch wide rabbet cuts were made along the contacting edges of the model panels, allowing them to overlap. The overlapping structure simplified construction of the model and eliminated gaps between panels through which backfill soil could escape. The panel length and height prior to constructing the rabbet cuts were 2-1/8 inches by 2-1/8 inches, so that after the cuts were made and the panels overlapped, the overall panel dimensions were 2 inches by 2 inches. Constructing the panels in this manner did not change their mass.

3.1.3 Reinforcement

A common prototype geogrid reinforcement used in reinforced soil structures is 5-foot wide high density polyethylene (HDPE) strips. Using HDPE reinforcing strips in the geotechnical centrifuge model was not feasible, so geogrid reinforcement was modeled using a woven nylon material. In geotechnical centrifuge modeling, it is desirable that stresses scale according to:

$$\frac{\sigma_p}{\sigma_m} = 1$$

In replica scaling, this relationship between stresses can be achieved by scaling the cross sectional area of a reinforcement material by the same linear scale used in the model. If reinforcement materials are different, an appropriate model material is chosen to simulate a prototype material strength such that:

$$F_p/A_p = F_m/A_m$$

or

$$F_p/b_p t_p = F_m/b_m t_m$$

or

$$\frac{F_p/b_p}{F_m/b_m} = \frac{t_p}{t_m}$$

where:

F_p, F_m = ultimate static tensile force of reinforcement (prototype and model);
A_p, A_m = cross sectional area of reinforcement (prototype and model);
b_p, b_m = reinforcement width (prototype and model); and
t_p, t_m = reinforcement thickness (prototype and model)

To ensure geometric similarity in the model to be tested, it is desirable that dimensions scale according to the inverse gravitational acceleration scale. Therefore,

$$t_p/t_m = g_m/g_p = n$$

and

$$\frac{F_p/b_p}{F_m/b_m} = n$$

The nylon material was selected because the ultimate static tensile strength per unit width of this model material (116 pounds per inch width) approximately scaled that of the prototype material, Tensar UX 1500 geogrid (3,653 pounds per foot) at 30 g. Although the thickness of the nylon mesh was too small to measure with any degree of accuracy, the laws of similitude dictate that if the force per unit width of the nylon mesh simulates the force per unit width of the prototype geogrid, then the nylon mesh thickness simulates the prototype geogrid thickness.

The nylon mesh strips were cut to a width of 2 inches. The length of the strips was generally six inches, but was varied in some tests as part of the parametric study. The strips were epoxyed to the Extren facing panels. A typical reinforced facing panel used in the reliability test series is shown in Figure 2.

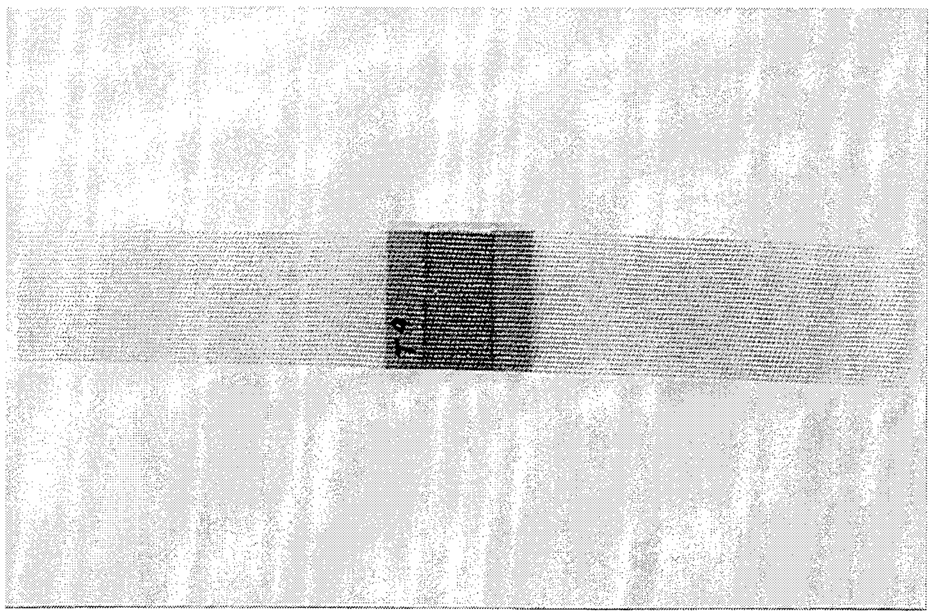


Figure 2. Typical Facing Panel with Reinforcement

3.1.4 Soil Backfill

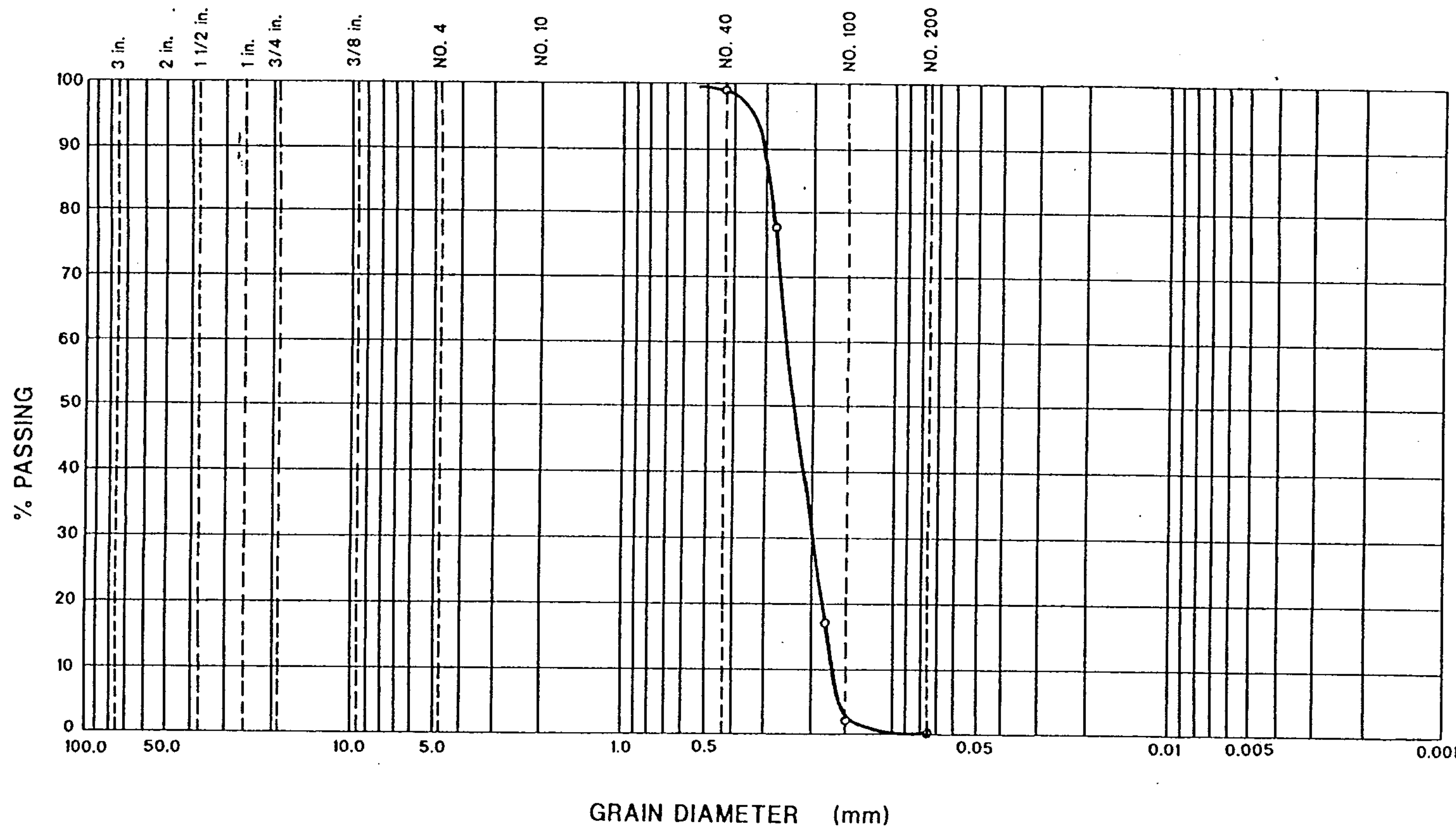
Exact simulation of the prototype soil in the geotechnical centrifuge would require scaling the particle dimensions while maintaining the same soil constitutive properties. However, this is not usually possible and prototype soil is commonly used in geotechnical centrifuge models to ensure constitutive properties are properly modeled. It is important to conserve the constitutive behavior of the prototype soil in order to ensure similar soil performance. The constitutive behavior of soil can undergo significant changes when the grain size is reduced or the grain shape changed. By using the prototype soil in the centrifuge model the same constitutive properties are maintained. There are limits to the use of the prototype soil which are set by the specific prototype event being modeled. In this set of experiments, it is important that the grain sizes of the soil be small compared to the panel dimensions to avoid point loading of the panels. Also, it is important that the soil particles interact through the soil reinforcement openings. These conditions were met by the soils used in these tests.

All 10 reliability tests and 14 of the 16 production tests were conducted with a local sand collected at Tyndall AFB (referred to as Tyndall Beach sand). The Tyndall Beach sand is a white, poorly-graded (C_u=1.43, C_c=0.95), subrounded, fine-grained quartz sand with D_{50}=0.23 mm and no fines. Two production tests were conducted with sand collected from a site on Tyndall AFB (referred to as the Sky X area) where full-scale testing of reinforced soil walls was being conducted. This sand is referred to as Sky X sand. The particle size distribution of the Sky X sand is similar to that of Tyndall Beach sand (C_u=1.57, C_c=0.74, D_{50}=0.23 mm), but contains approximately 2 percent fines. The grain size distribution curves for both soils are presented in Figures 3 and 4. Although the Sky X soil has a silica composition nearly identical to Tyndall Beach sand, it is brown in color, indicating the presence of other materials. A qualitative analysis of the Sky X and Tyndall Beach sands was performed using a scanning electron microscope coupled to an energy dispersive X-ray system (see Figure 5).

3.2 DATA COLLECTION

3.2.1 General

The success of the Phase 2 test program depended on the ability to accurately measure displacement and to collect data obtained from instrumentation buried in the test models. The instrumentation data consisted of: (i) peak pressures on the facing panels and in the free field from the compression wave of the detonation, (ii) peak accelerations of the facing panels and in the soil in the free field from the compression wave of the detonation, and (iii) arrival times of the compression wave at the facing panels and known locations in the free field to calculate compression wave velocities. Pressure gages and accelerometers used in all test models are shown in Figure 6.

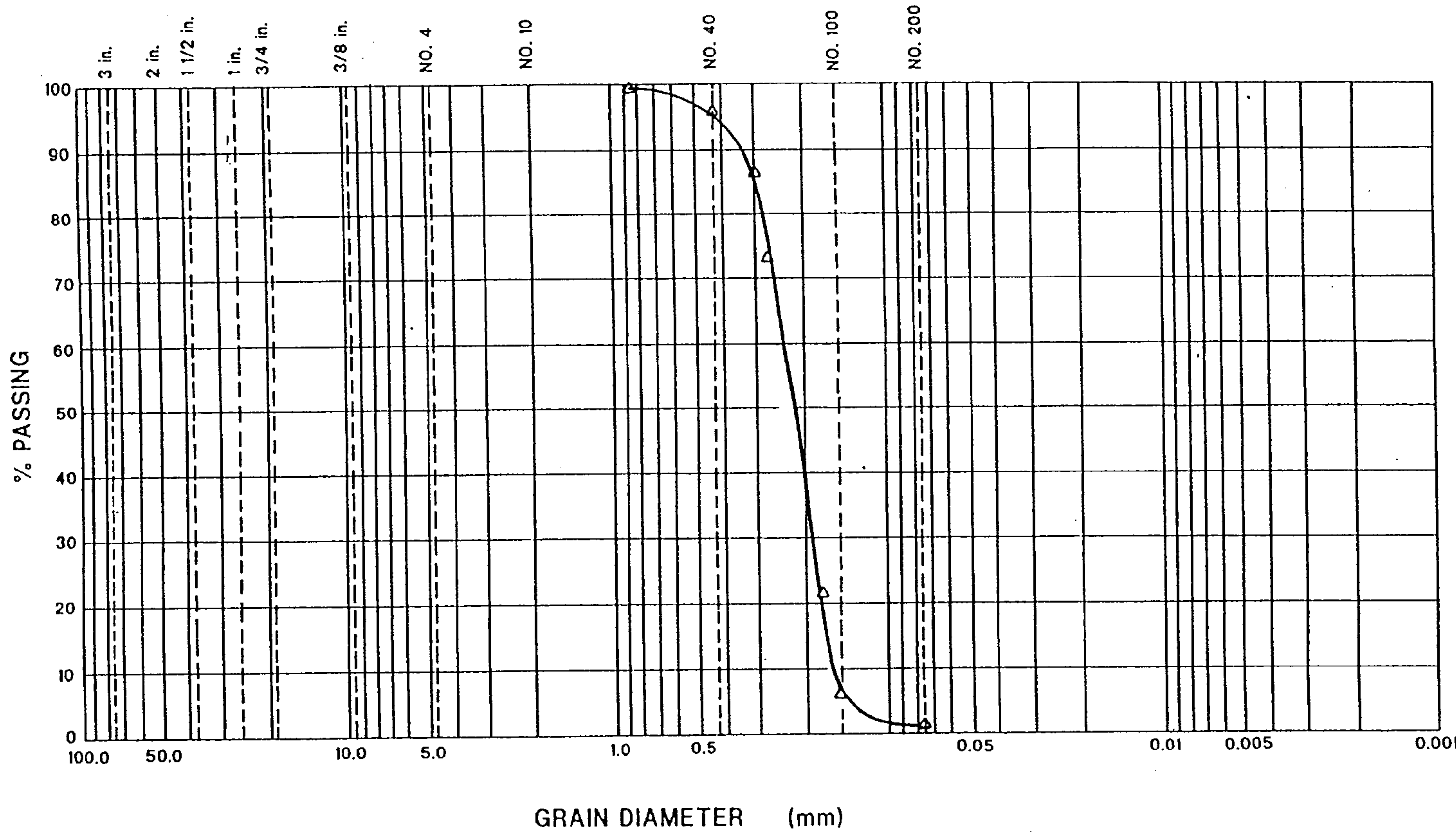


D_{50} = 0.23 mm

C_u = 1.43

C_c = 0.95

Figure 3. Grain Size-Distribution Curve: Tyndall Beach Sand



D_{50} = 0.23 mm

C_u = 1.57

C_c = 0.74

Figure 4. Grain Size-Distribution Curve: Sky X Sand

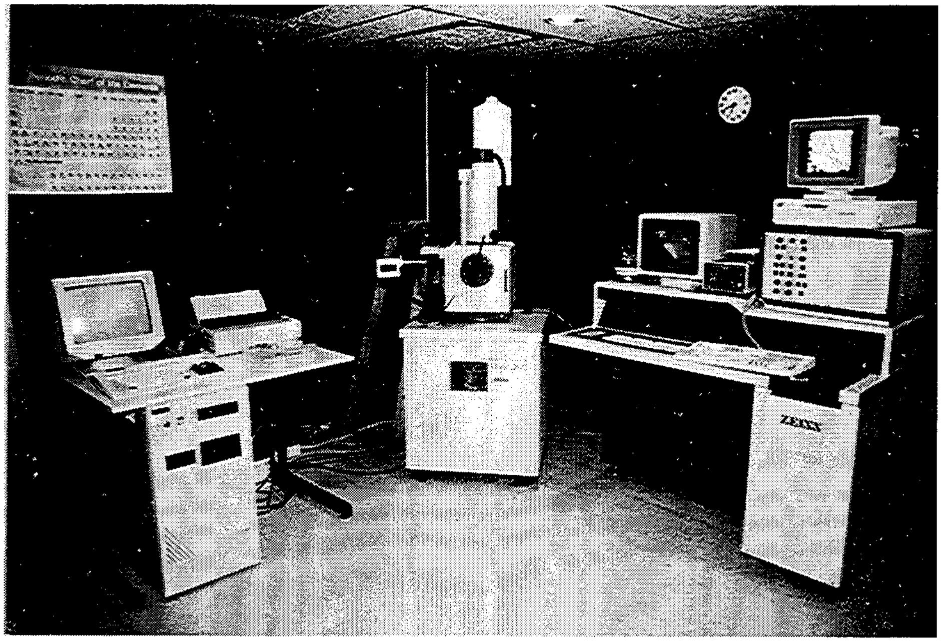


Figure 5. Image Analysis System

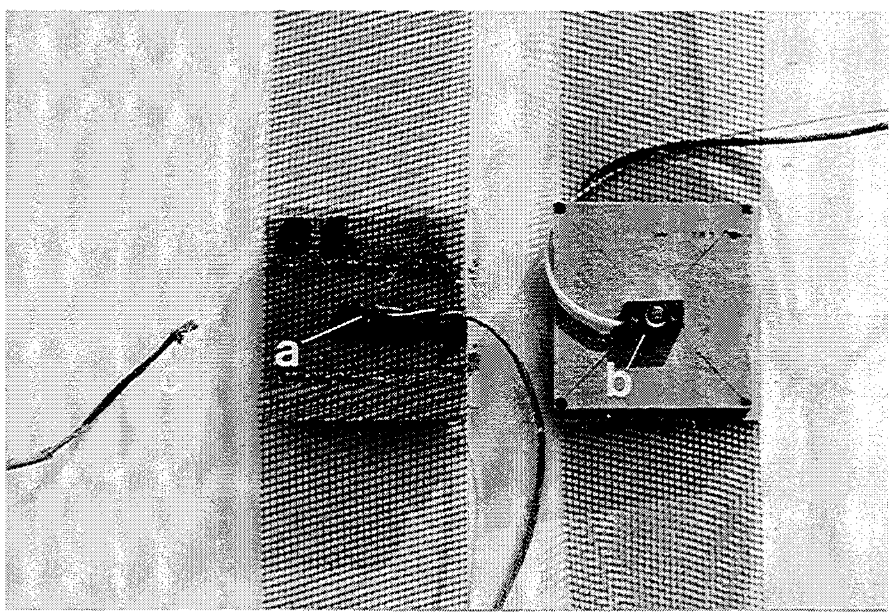


Figure 6. Instrumentation: a) Accelerometer, b) Soil-Interface Pressure Gage, c) Free Field Pressure Gage

3.2.2 Accelerometers

Endevco piezoresistive accelerometers (model 7270A-6K, 6000 g capacity, and model 7270A-20K, 20,000 g capacity) were used in the test models. The dimensions of these accelerometers are 14.22 millimeters (0.560 inches) by 7.1 millimeters (0.28 inches) by 2.79 millimeters (0.110 inches). Three accelerometers were epoxyed to the front face of the top center, middle center, and bottom center facing panels of the model wall to measure acceleration time-domain waveforms for the panels during the explosive event. One accelerometer was placed in the backfill behind the geogrid, on the same horizontal plane as the detonator, to record free field accelerations. Initially, the model 7270A-6K, 6000 g accelerometers were used at all of the above locations. However, after preliminary tests were conducted, large free field accelerations were noted, and the 6000 g free field accelerometer was replaced with the model 7270A-20K, 20,000 g accelerometer.

3.2.3 Pressure Gages

Precision Measurements Co. miniature pressure transducers (model 156F-500, 500 psi) were used to measure pressure time-domain waveforms at the soil-facing panel interface during the explosive event (hereafter referred to as soil-interface pressure gages). The gage dimensions are 9.5 mm (0.312 in) by 3.96 mm (0.156 in) by 1.55 mm (0.062 in). For each test, three pressure transducers were mounted on the back face of the top center, middle center, and bottom center facing panels of the model reinforced soil wall. For some tests, an additional pressure transducer was mounted on the side of the sample containment bucket in the same horizontal plane as that of the detonator.

Precision Measurements Co. miniature pressure transducers (model 105S-500, 500 psi) were used to measure free field pressure time-domain waveforms during the explosive event (hereafter referred to as free-field pressure gages). The gage dimensions are 2.6 mm (0.105 inches) in diameter by 0.45 mm (0.018 inches) thick. For each test, two transducers were placed in the backfill soil behind the geogrid, in the same horizontal plane as that of the detonator.

3.2.4 Data Recording

Data was collected on a Pacific Instrument Model 5700 Transient Data Recorder

(TDR). The TDR was located on the arm of the geotechnical centrifuge and hardwired to the gages located in the sample bucket. The TDR has 16 channels. Each channel consists of a programmable signal conditioner, an amplifier, a filter, and a 12-bit analog-to-digital converter. The digitizer provides sampling rates between 1 MHz and 10 Hz. Each channel contains 256K of nonvolatile memory (Reference 6).

3.2.5 Displacements

Static panel displacements were measured with a digital depth micrometer (Figure 7) with a resolution of +/- 0.005 inches. An aluminum guide track was constructed to guide the micrometer pin and to keep it perpendicular to the facing panels during measurement. The track consisted of a 20-inch long by 3-inch wide aluminum plate with a small slot along the longitudinal centerline. The plate was clamped to the front of the sample bucket after the front wall of the bucket was removed. The micrometer pin was then threaded through the slot until the flat base of the micrometer was in full contact with the face of the guide track. The micrometer pin was then extended or retracted as necessary to obtain displacement measurements. The depth micrometer and aluminum guide track are shown in Figure 8.

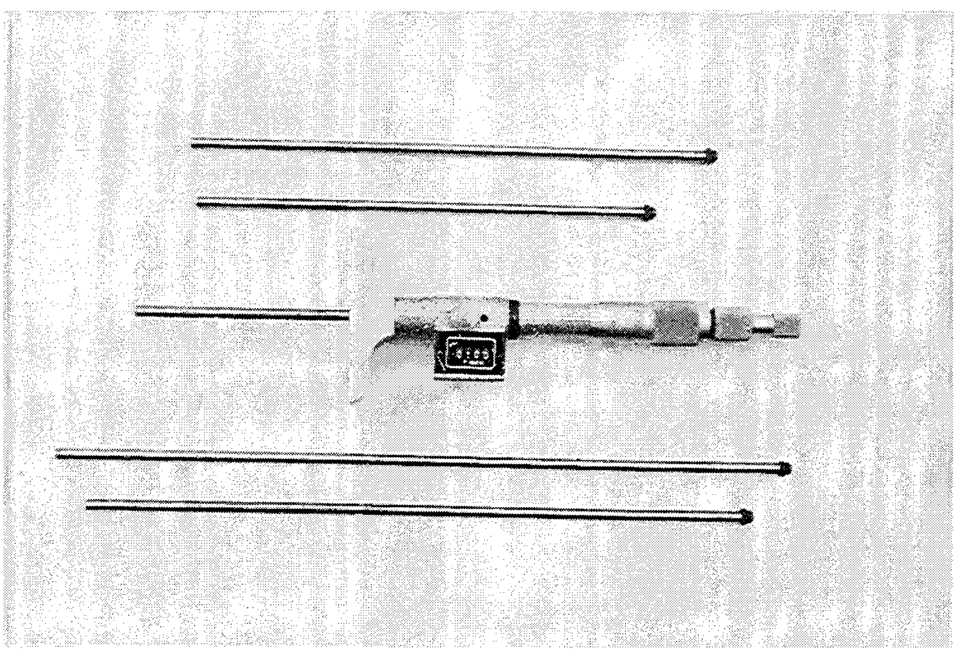


Figure 7. Digital Depth Micrometer

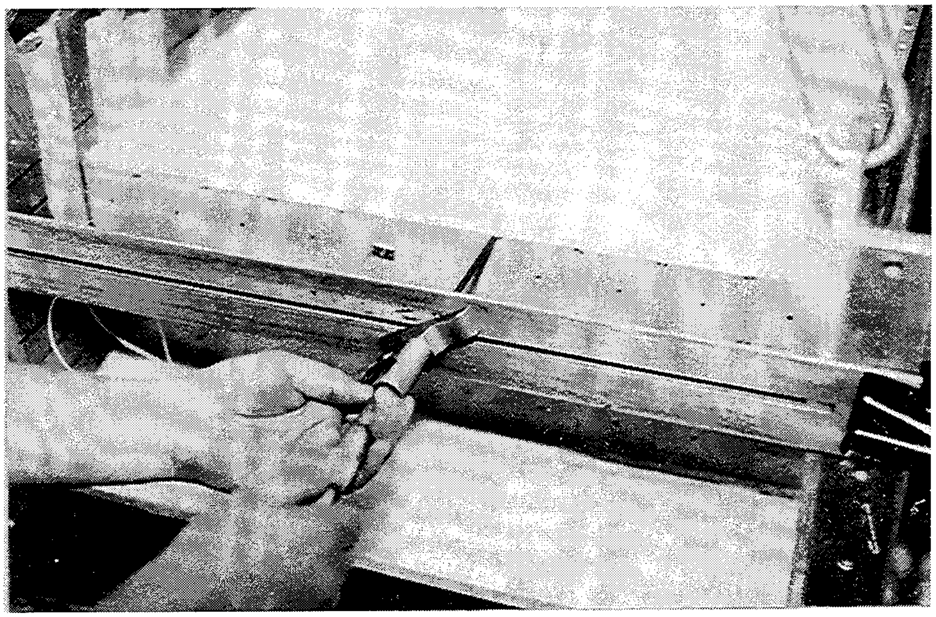


Figure 8. Digital Depth Micrometer and Aluminum Guide Track

3.3 DESIGN OF DETONATOR

3.3.1 General

The modeling of the explosive charge is an important factor in modeling the dynamic loading conditions on a reinforced soil wall in the geotechnical centrifuge. The model reinforced soil wall should experience the same (scaled) explosive event as does the prototype. Most explosive events in a centrifuge involve modeling military high explosive bombs. In recent years, the military began manufacturing bombs using the explosive Composition-H-6 (H6).

Modeling a 500 pound bomb containing 192 pounds of H6 explosive (Reference 3) in a 30 g acceleration field required the design of a new detonator. The Reynolds Industries Systems, Inc. RP-83 detonator is a commonly used explosive charge for modeling high explosive bombs in a geotechnical centrifuge. However, a single RP-83 detonator does not contain enough explosive to model the prototype charge at 1:30 scale. It would have been necessary to tape three detonators together to model the prototype explosive charge. Ideally, the model explosive charge should contain a single pressing of explosive to insure an instantaneous explosive event.

The new detonator was designed by Applied Research Associates, and manufactured by Reynolds Industries Systems Inc. specifically for this study. The detonator is shown in Figure 9, and consisted of an output charge of Plastic Bonded Explosive 9407 (PBX 9407) pressed against an initial pressing of Pentaerythritol Tetranitrate (PETN). PBX 9407 was chosen because of its sensitivity to the shock wave of the initial pressing. This explosive allowed the initial pressing to be reduced to O.0000506 pounds (23 mg), and resulted in a satisfactory total explosive energy release curve.

3.3.2 Explosive Event Scaling

The most common explosive scaling relationship is that of cube root scaling. This scaling method can be used when two explosive charges of the same explosive type are used (Reference 7). The relationship between model and prototype explosive masses follow the equation:

$$\left[\frac{m_p}{n^3}\right] = m_m$$

where m_p is the mass of the prototype, n = the geometric scale factor, and m_m is the mass of the model explosive charge.

In modeling, similitude between the model and prototype must be maintained. It can be shown that an amount of explosive charge modeled in an acceleration field can simulate the explosive effect of a larger prototype explosive charge. Similitude requirements during dynamic modeling can be found in Nielsen (Reference 8), and Tabatabai, et al. (Reference 9). In this simulation technique, the Buckingham Pi theorem can be used to define a relationship between variables when the explosive types are not the same. Energy scaling was used to determine the amount of explosive required to simulate a 500 pound bomb containing 192 pounds of H6 explosive (prototype) in a 1:30 scale model using PBX 9407 explosive. Schmidt and Holsapple (Reference 10) suggest the following Buckingham Pi term for energy scaling:

$$\pi = \left(\frac{g}{Q}\right) \quad \left(\frac{m}{\delta}\right)^{1/3}$$

where: Q = heat (energy) of detonation per unit mass of explosive
δ = initial mass density of the explosive
m = mass of the explosive
g = gravitational acceleration

From the above equation, the model mass may be determined as follows:

$$m_m = \left(\frac{g_p}{g_m}\right)^3 \quad \left(\frac{Q_m}{Q_p}\right)^3 \quad \left(\frac{\delta_m}{\delta_p}\right) \quad m_p$$

Data regarding the heat of detonation (heat liberated during explosive decomposition) and the density of the explosive types was obtained from the Lawrence Livermore National Laboratory Explosives Handbook, (Reference 11), and the Handbook of Land Mines and Military Explosives for Countermine Exploitation, (Reference 12).

Therefore, 192 pounds of H6 may be modeled in the centrifuge at 1:30 scale when the model explosive is 0.00729 pounds (3.3089 grams) of PBX 9407. However, during manufacturing of the model explosive charge, the volume of the canister containing the PBX 9407 was to a small to contain the full weight of PBX9407, and the amount of explosive was limited to 0.00694 pounds (3.1489 grams). This was corrected by conducting all centrifuge tests at 30.5 g. As discussed previously, all model reinforced soil wall components were designed for 30 g. Accelerating the model walls to 30.5 g instead of 30 g resulted in equivalent scaled reinforced soil wall dimensions that were 1.7 percent larger than intended, a negligible difference. For ease of reporting purposes, the term 30 g is presented in this report. The detonator is shown in Figure 9.

3.4 MODEL CONSTRUCTION AND TESTING

3.4.1 Soil Pluviator

In order to obtain uniform, reproducible soil densities, a soil pluviator was used to

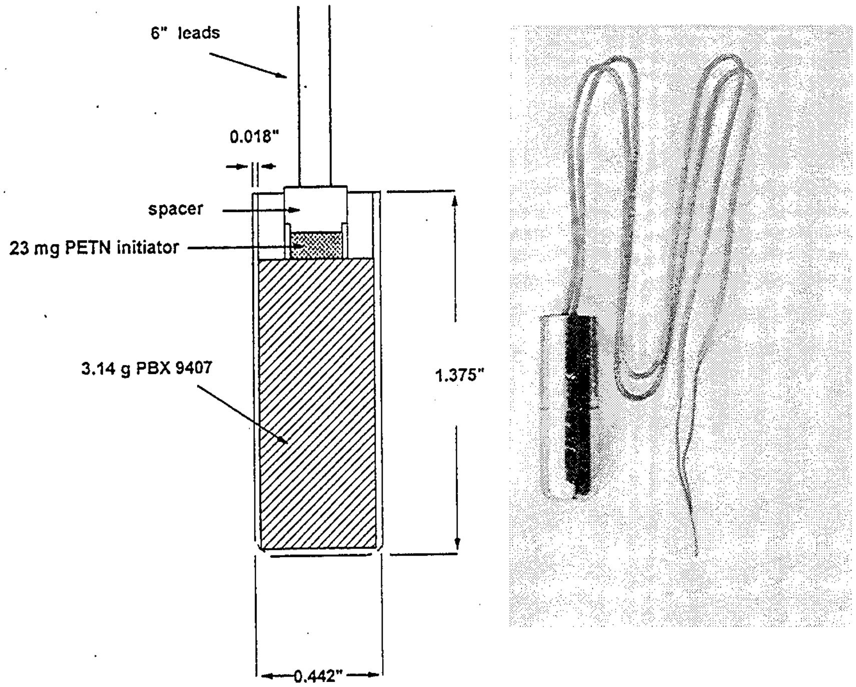


Figure 9. Detonator

place soil during the construction of the model. The pluviator is a sand raining device which disperses sand into a container at a uniform density. Soil is placed in a large hopper at the top of the apparatus. The bottom of the hopper consists of a perforated plate, covered with an aluminum shutter plate. Removal of the shutter plate allows the sand to drop through the perforations and fall freely through a series of diffuser screens which disperses the sand and rain it into the sample preparation bucket. The density of the soil can be altered by adjusting the falling distance between the hopper and sieves (F height), sieves and sample bucket (H height), and the size and spacing of perforations in the hopper base (see Figure 10). Details of the pluviator used for this particular study are presented by Purcell and Hollopeter (Reference 6).

In preparing the model reinforced soil walls, it was crucial to obtain level surfaces on which to place the geogrid reinforcing. When a perfectly level sample was not achieved with the large pluviator, a small hand pluviator was used to even out any small undulations in the soil surface. Through trial and error it was determined that a hand pluviator consisting of a U.S. number 20 sieve sandwiched between two number U.S. 30 sieves produced approximately the same density as the large pluviator.

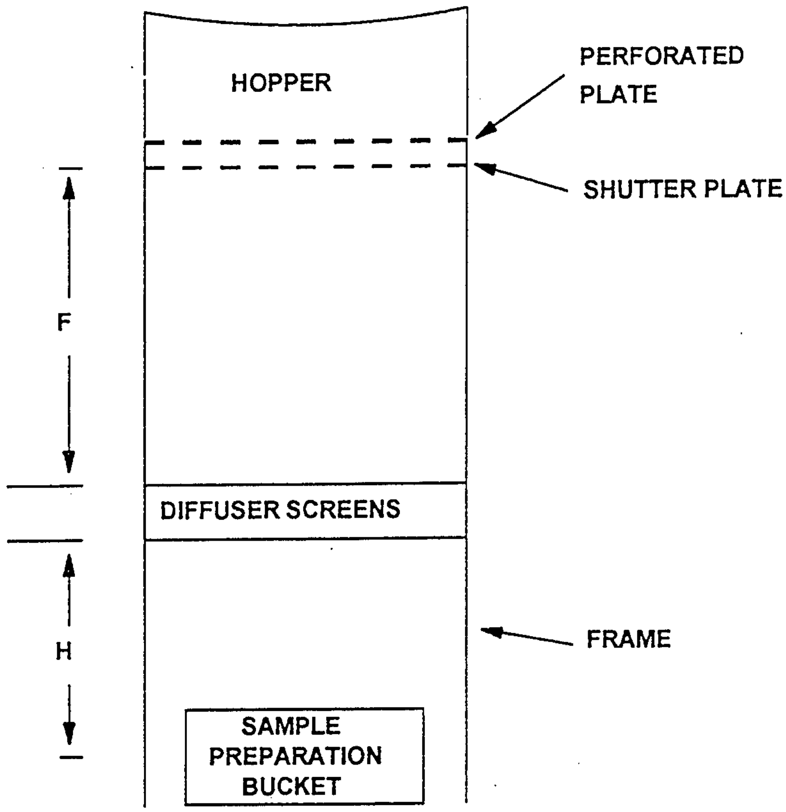


Figure 10. Schematic Diagram: Soil Pluviator

3.4.2 Leveling Pad

Each model wall was constructed on a small leveling pad consisting of a thin wooden strip covered with Number 200 grit sandpaper. The pad was placed in the sample bucket on top of the pluviated base prior to constructing the wall (see Figure 11). The sandpaper was attached to develop a frictional interface between leveling pad and facing panels, qualitatively similar to that which would exist between a concrete panel and a concrete leveling pad.

3.4.3 Sample Container

All samples were constructed in an aluminum sample bucket with dimensions 20-inch long, by 20-inches wide by 16-inches deep. A portion of the front wall of the bucket was removable, which enabled photographing the model wall and measuring panel displacements.

Prior to constructing the model wall, a thin aluminum plate was lowered vertically into the bucket 4 inches from the bucket front wall. The plate was then bolted in place. The model wall was constructed against this plate. The plate had three small openings to accommodate the protruding accelerometers mounted on the front faces of the three center model wall panels. After completing the model, the plate was removed, leaving a 4-inch wide airspace between the model wall and the front wall of the sample bucket. The sample bucket and aluminum plate are shown in Figure 11.

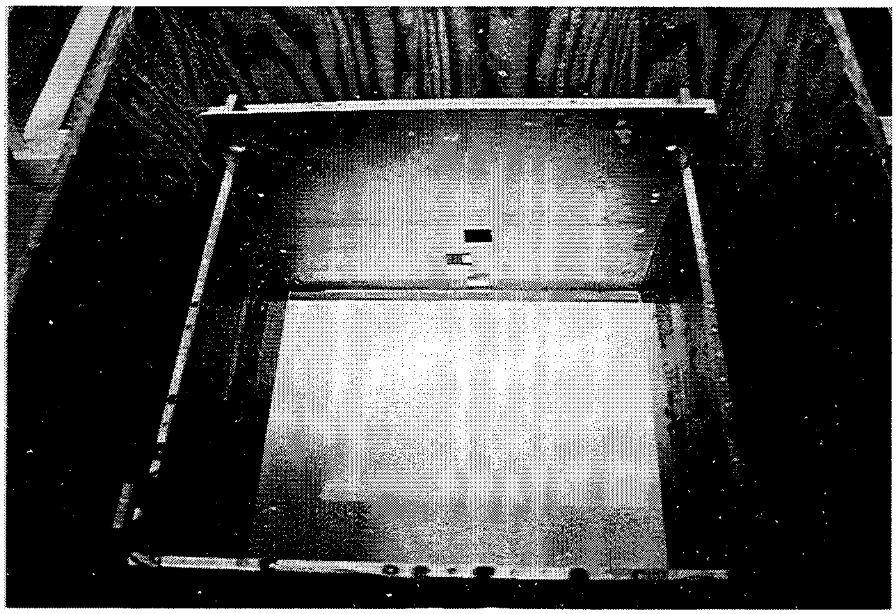


Figure 11. Sample Bucket, Aluminum Plate, and Leveling Pad

3.4.4 Model Construction

The model construction technique discussed below was used to prepare most test models. Any deviations from this technique or any additional construction requirements implemented in the centrifuge tests are summarized at the end of this section. A complete photographic record of the wall construction procedure is included in Appendix A.

- Prior to model construction, all test soil was oven dried overnight, then passed through a #10 sieve to remove large hardened clods, twigs and shells. Except for one test sample (moist Sky X soil), all models were constructed and tested with dry soil.

- A 6-inch thick soil base was pluviated into the sample bucket. The thin aluminum plate was set in the bucket and bolted in place 4 inches behind the front wall of the sample bucket. The airspace between the plate and the front wall of the sample bucket was covered with foil wrap to keep out sand during subsequent pluviations. The levelling pad was then set in place on the pluviated soil surface.

- The first course of facing panels was constructed against the thin aluminum plate, and the geogrid strips were lifted and temporarily taped to the plate to keep them from interfering with future pluviations. A small amount of putty was pressed between the end panels and the sample bucket to seal any small airspaces through which soil might escape.

- Soil was then pluviated into the bucket to the level of the first layer of geogrid strips. The first layer of geogrid strips was detached from the aluminum plate and laid out along the soil surface. A small amount of sand was then pluviated over the strips to anchor them in place until the next soil lift was constructed.

- The sequence of pluviating, placing geogrid strips and constructing panel courses was continued until the pluviated soil surface was 3 inches above the bottom of the wall (9 inches above the bottom of the sample bucket). The detonator was set in place by gently pushing it into the soil mass until the center of mass of the explosive material in the detonator was at the same level as the pluviated surface (i.e., three inches above the base of the wall). The standoff distance (i.e., distance from the detonator to the back face of the middle center facing panel) was a controlled variable in this study.

- The free field gages were set in place at the same elevation as that of the detonator. Gage wires were taped to the side of the sample bucket. Actual gage locations are shown in Figures 12 and 13.

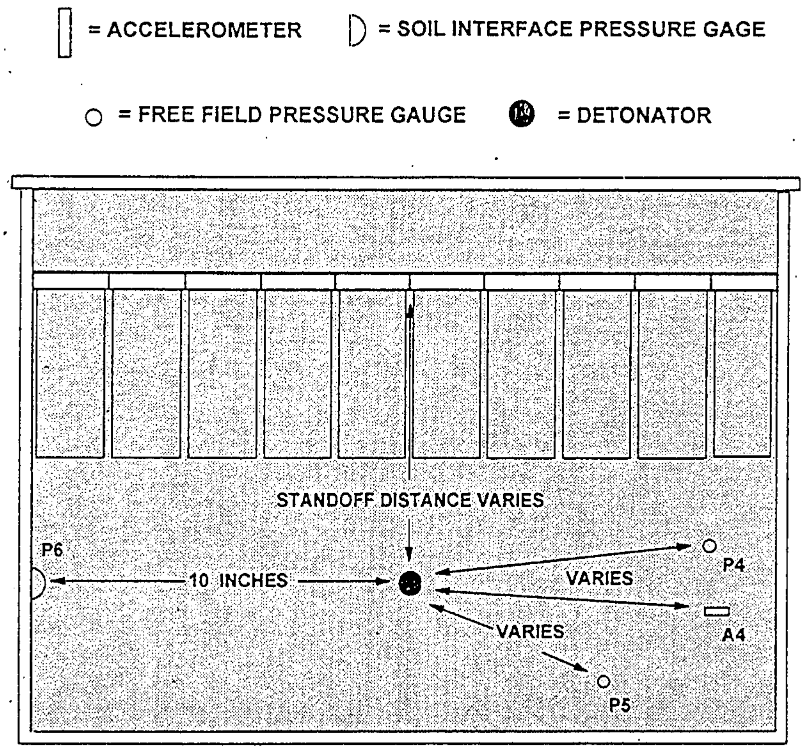


Figure 12. Gage Locations: Plan

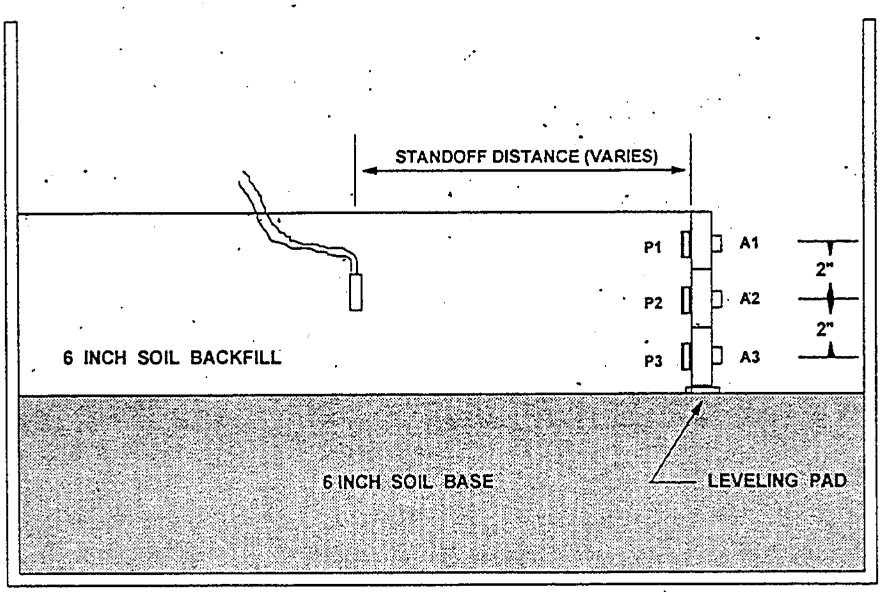


Figure 13. Wall Panel Gage Locations: Profile

- The sequence of pluviating, placing geogrid strips and constructing panel courses was continued until the pluviated soil surface was level with the top of the last course of facing panels. The aluminum plate was then removed and the final sample was weighed. The front panel of the sample containment bucket was then removed and photographs were taken.

3.4.5 Testing Procedure

The testing procedure discussed below was followed for most geotechnical centrifuge tests. Any deviations from this procedure are summarized at the end of this section.

- Initial panel positions were measured with the digital depth micrometer. The panel numbering system is shown in Figure 14. Measurements were taken at each corner of panels 5, 6, 15, 16, 17, 26 and 27. These panels are located in the central portion of the wall where maximum deflection was expected. Only the center point of the remaining panels were measured.

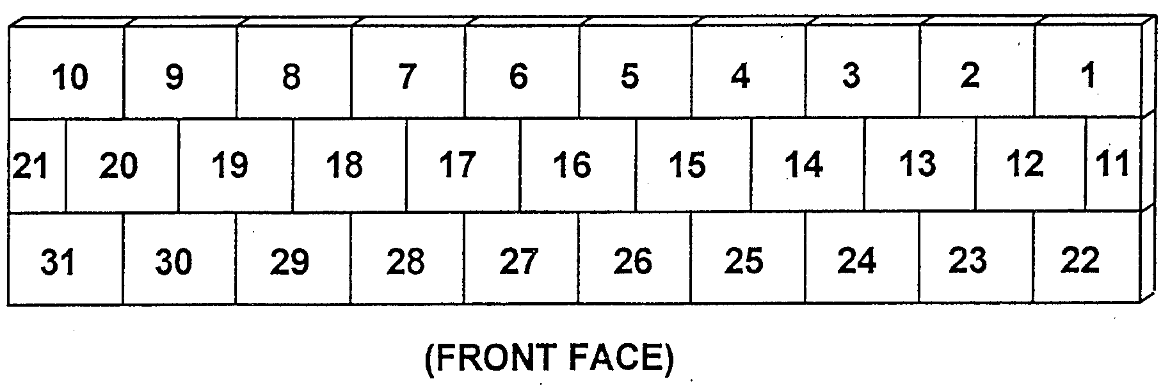


Figure 14. Facing Panel Numbering System

- The completed model was mounted on the geotechnical centrifuge platform and all instrumentation wires were connected to the on-board data acquisition system. The sample was accelerated to 30 g (nominal) for 1 minute and decelerated back to 1 g to expose the model to an initial gravity loading. The sample was then reaccelerated to 30 g (nominal) and the firing system was initiated for detonation. After testing, the measured acceleration and pressure gage data were downloaded from the on-board data acquisition system to a personal computer for later analysis.

- Photographs were taken, and crater dimensions and post-shot panel displacements were measured. For the postshot displacements, the four corners of each panel were measured. Panel displacements were calculated by subtracting the average corner position of each panel prior to detonation (equal to the average of the four corner measurements for each of the seven central panels, and to the panel center point measurement for each of the remaining panels) from the average corner position of each panel after detonation (equal to the average of the four nodal measurements of each panel).

3.4.6 Exceptions to the Model Construction and Testing Procedures

Deviations to the previously described model construction and testing techniques are summarized below.

- An additional set of panel displacements were taken during the reliability test series to investigate the amount of wall displacement due to static loading. These measurements were taken after the initial acceleration to 30 g.

- One test was conducted with moist soil placed by hand in lifts and hand tamped to a pre-determined lift thickness.

- One test included construction of a roof structure and additional soil overburden.

- One test included construction of a soil slope behind the wall.

Detailed explanations of these design variations are presented in Sections 5 and 6.

SECTION 4

PRELIMINARY TEST SERIES

Five preliminary tests were conducted to verify the model construction and testing procedure. The preliminary test series also served to develop the best procedure for measuring displacements of the reinforced soil wall, to evaluate the performance of the newly designed detonator, and to test the performance of the pressure transducers and accelerometers.

It was intended that the preliminary test series be used to confirm the test plan and to gain confidence in the ability of the geotechnical centrifuge to correctly model an explosive event. A computer code CONWEP (Reference 3) for predicting explosive phenomena was used to predict prototype free field peak pressure and peak acceleration values. These predictions were used to evaluate the detonator performance in the geotechnical centrifuge. Prior to any wall construction, the pluviator F height, H height, and perforated plate hole diameter and frequency were adjusted by trial and error to rain uniform Tyndall Beach sand samples at 103 pcf. The preliminary test models were instrumented with accelerometers A1 through A3 and pressure gages P1 through P4. During preliminary testing, soil-interface pressure gages P1 through P3 yielded noisy, unreadable data. The problem was identified and corrected, but not in time to obtain usable soil-interface pressure data for the preliminary testing program. Therefore, usable gage data for the preliminary tests were obtained only from accelerometers A1 through A3 and free field pressure gage P4.

Two preliminary tests were conducted with nominally identical test parameters except the standoff distance of the explosive was 8.5 inches for one test and 9.5 inches for the other. Peak free field pressures and accelerations on the facing panels were within 10 and 22 percent, respectively, of the CONWEP predictions for the 8.5-inch standoff. The free field pressure gage failed during the test with the 9.5-inch standoff, and its output could not be compared to CONWEP predictions. The peak accelerations on the facing panels for this test were within 33 percent of the CONWEP prediction for accelerometers A1 and A3, but the variation was much larger for accelerometer A2 (within 250 percent).

Two tests were conducted with a roof structure and additional overburden soil resting on the reinforced soil wall. Peak free field pressures measured at P4 and P5 were within 25 and 46 percent of the CONWEP predictions. Peak accelerations on the facing panels were within 17 and 54 percent of the CONWEP predictions.

One preliminary test was conducted with a soil unit weight of 101 pcf. The peak free field pressure was within 14 percent of the CONWEP prediction. The peak accelerations at the facing panels were within 50 percent of the Conwep prediction for accelerometer A1, but were only within 81 percent of the Conwep prediction for accelerometers A2 and A3.

With the exception of three accelerometer readings, all gage data fell within approximately 50 percent of the predicted values. This is considered very good correspondence because the gages are extremely sensitive and have often shown a very wide range of results for similar test conditions.

Observed wall displacements indicated that the new detonator was properly designed. Although a final method had not been developed for collecting wall displacement data at the time of preliminary testing, qualitative analysis of wall displacements indicated that the wall displaced differently for different test conditions. For all preliminary tests, no panel breached. This result is consistent with the results of 1990 full-scale tests (Reference 4) and current full-scale tests at Tyndall AFB where standoff distances are similar when appropriately scaled. Visual examination confirmed that the wall displaced less in the test with the 9.5 inch standoff than in the test with the 8.5 inch standoff. No visual difference in wall displacement was noticed in the tests with varying soil unit weights, but this may have been because of the small difference in soil unit weights for the two tests (103 pcf and 101 pcf).

The shape of the deformed walls varied with the constraint condition at the wall top. For the tests conducted with a roof structure, maximum wall displacement occurred at the center of the wall. For the tests conducted with no roof structure, maximum displacement occurred at the top of the wall.

The results of the preliminary test series indicate that pressure transducers, accelerometers, and the new detonator were functioning appropriately. The collected data also indicated that the detonator was functioning properly. Visual comparisons of full-scale and model wall displacements indicated that the detonator simulated the effects of a 500-pound weapon.

SECTION 5

RELIABILITY TEST SERIES

5.1 DESCRIPTION OF TESTS

Exact replication of a physical model is not possible; differences in the outcome of any physical model test are inevitable due to random variation in model construction, test conditions, instrumentation, and other unknown factors. The reliability tests were conducted to identify the range of test results which could be expected due to these random variations. Knowledge of this range would then aid in evaluating test results obtained during the production testing series.

Ten nominally identical test samples were constructed and tested in the centrifuge. The system parameters maintained for all 10 samples are presented in Table 2. Each model wall was prepared by the same individuals, using the same techniques, equipment and instrumentation. The only exceptions to this are noted below, and did not significantly affect the results.

TABLE 2. RELIABILITY TEST PARAMETERS

Reliability Test Parameters	
Facing Panel Geometry:	2 inches x 2 inches
Reinforcement Length:	6.0 inches
Reinforcement Width:	2.0 inches
Reinforcing Layers:	2 strips per facing panel
Soil Type:	Dry Tyndall Sand
Soil Density:	103 pcf
Overall Model Geometry:	No roof Structure 8.5 inch Standoff No Berm

The procedures for constructing and testing the reliability test models were the same as those described in Section 3, except an extra set of displacement measurements was made. These measurements were taken after the model had been cycled in the geotechnical centrifuge from 1 g to 30 g and back to 1 g. The geotechnical centrifuge was stopped and displacement measurements were made to evaluate the displacement caused by static loading to 30 g. The results of these measurements are discussed later in this section.

Displacement measurements for each test were converted to individual panel displacements. Residual panel displacements due to static load only were calculated as the difference between measurements before and after acceleration to 30 g. Residual panel displacements due to dynamic load were calculated as the difference between measurements after acceleration to 30 g and after the explosive event. Total panel displacements were calculated as the sum of static and dynamic displacements. The average wall displacement for each test was calculated by averaging the displacement of all the panels. The average row displacement for each test was calculated by averaging the displacements of all panels in that row.

The mean displacement was calculated for each panel and for each wall, along with the standard deviation and coefficient of variation (standard deviation divided by the mean). In addition, the mean plus and minus two standard deviations were calculated for each panel and for each wall. The range defined by the mean plus and minus two standard deviations covers 95 percent of the expected values for a normally distributed variable. This range was used to determine the significance of the specific parameters which were varied in the production test series.

In addition to wall displacement measurements, time-domain waveform data from the free-field pressure gages and accelerometers, and from the soil interface gages and panel accelerometers were obtained for each reliability test. For each gage, the peak pressure or acceleration was recorded. For each test, the peak interface pressures and peak panel accelerations were averaged to obtain the average peak interface pressure and average peak panel acceleration, respectively. The mean, standard deviation, mean plus and minus two standard deviations, and coefficient of variation were calculated for each gage and for the average peak interface pressure and the average peak panel acceleration.

After completing the fifth reliability test, one additional soil interface pressure gage was added to the instrumentation package. This gage (P6) was placed on the bucket wall, as shown in Figure 15. Because the gage was placed behind the reinforced soil mass, and was too small to affect the density of the backfill, it was concluded that using the gage for half the tests would not affect the behavior of the wall. No other changes were made during the reliability testing, and the 10 tests are considered to be nominally identical.

Test 6 was intended to be the first reliability test. However, several problems developed during testing, including failure of the firing system, and it was decided before completing the test not to include it among the 10 reliability tests. Although it was later determined that the results of this test did fall within the range of the following 10 tests, test 6 results have been excluded from all analyses.

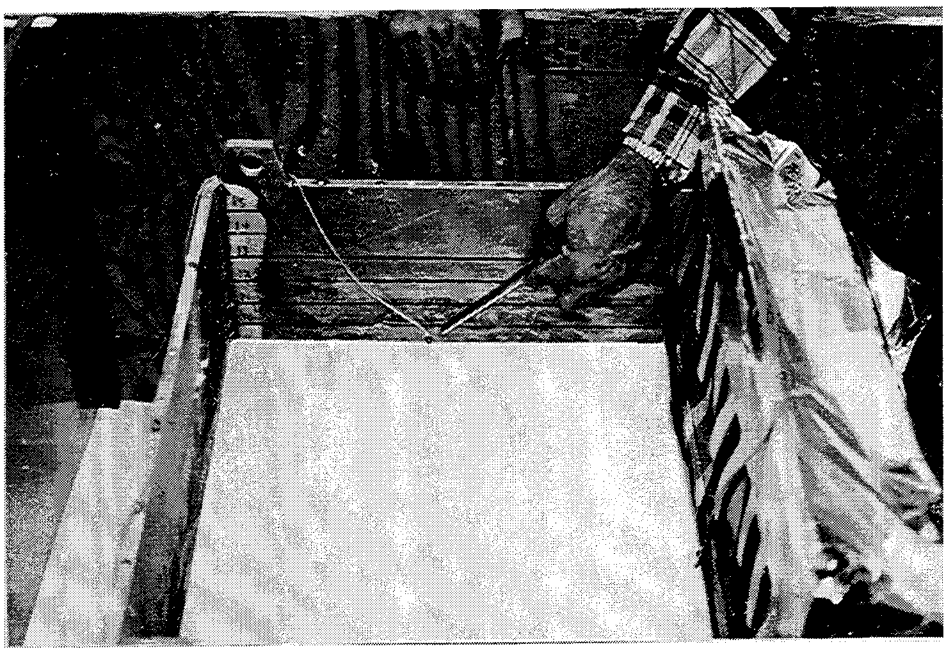


Figure 15. Soil-Interface Pressure Gage P6

5.2 RESULTS

5.2.1 Displacements

Total residual displacements for each panel in the reliability series (tests 7-16) are presented in Table 3. Also presented in this table below the individual panel displacements are the average static, dynamic, and total displacements for each wall and the average total displacements for the top, middle and bottom row of panels for each wall. The mean, standard deviation, coefficient of variation, and the mean plus and minus two standard deviations for all displacements are shown in the last five data columns of the table. The displacements shown in Table 3 are actual model displacements; prototype displacements are obtained by multiplying by the g-level of the test.

The mean and standard deviation for average static wall displacement are 0.018 in. and 0.008 in., respectively. The mean and standard deviation for total (static and dynamic) displacement are 0.324 in. and 0.025 in., respectively. The mean displacement for static loading is less than 6 percent of total displacement. The variability of the static displacement is 0.444 which is quite high compared to the variability of total displacement (0.077). For these reasons it was decided that only total displacement would be used in analyzing the reliability of the production tests. Hence, during production testing, only two sets of displacement measurements were made; one after completing the model construction, and one after detonating the explosive.

Looking first at the repeatability of individual panel displacements, Table 3 shows a range in the coefficient of variation from 6 to 16 percent. Excluding the panels at the ends of each row, where boundary effects are most severe, the range is 3 to 12 percent. These data show that individual panel displacements vary only slightly from test to test. The displacement pattern is also very consistent. For the top and middle row of panels, displacement increases monotonically from the end of the wall (panels 1 and 11) to a peak at the center two panels (panels 5 and 16), then decreases monotonically. This pattern is also followed in the bottom row in all but two tests (9 and 12). In tests 9 and 12 the maximum displacement is in panel 27, rather than panel 26 (0.001 inches greater than panel 26).

The average total displacement of each of the walls is very consistent, considering the complexity of the model. The standard deviation of average total wall displacement (0.025 in.) is only 7.8 percent of the mean wall displacement (0.324 in.). The data appear normally distributed; there are 5 tests above the mean and 5 below; 7 fall within 1 standard deviation, and all 10 tests fall within 2 standard deviations of the mean. The range of total wall displacement for the 95 percent confidence interval (mean minus or plus two standard deviations) is 0.274 to 0.374 in.

TABLE 3. PANEL DISPLACEMENT DATA: RELIABILITY TESTS

PANEL	AVERAGE PANEL DISPLACEMENT (inches) FOR RELIABILITY TEST #										STATISTICAL EVALUATION				
NUMBER	7	8	9	10	11	12	13	14	15	16	Mean (inches)	s (N-1)	Mean + 2s	Mean - 2s	COV (%)
1	0.174	0.137	0.167	0.151	0.149	0.136	0.164	0.156	0.162	0.140	0.154	0.013	0.180	0.127	9
2	0.324	0.290	0.292	0.325	0.283	0.276	0.308	0.301	0.316	0.284	0.300	0.018	0.335	0.264	6
3	0.509	0.491	0.458	0.521	0.454	0.396	0.461	0.458	0.517	0.452	0.472	0.038	0.548	0.395	8
4	0.752	0.728	0.654	0.747	0.637	0.586	0.637	0.640	0.723	0.673	0.678	0.056	0.790	0.565	8
5	0.943	0.914	0.826	0.934	0.837	0.821	0.821	0.793	0.889	0.874	0.865	0.053	0.971	0.759	6
6	1.017	1.137	1.107	1.172	0.947	0.964	0.926	0.964	0.977	0.996	1.021	0.086	1.194	0.848	8
7	0.736	0.689	0.698	0.819	0.599	0.662	0.627	0.621	0.631	0.657	0.674	0.066	0.805	0.543	10
8	0.450	0.396	0.413	0.474	0.387	0.351	0.394	0.396	0.419	0.425	0.410	0.034	0.479	0.341	8
9	0.288	0.261	0.312	0.313	0.243	0.231	0.215	0.266	0.271	0.247	0.265	0.033	0.330	0.199	12
10	0.122	0.099	0.110	0.112	0.101	0.096	0.096	0.110	0.091	0.084	0.102	0.011	0.124	0.080	11
11	0.095	0.103	0.095	0.098	0.097	0.098	0.093	0.087	0.077	0.095	0.094	0.007	0.108	0.079	8
12	0.177	0.178	0.159	0.174	0.167	0.131	0.161	0.164	0.165	0.158	0.163	0.013	0.190	0.136	8
13	0.281	0.281	0.250	0.285	0.255	0.192	0.242	0.261	0.276	0.255	0.258	0.027	0.312	0.203	11
14	0.423	0.416	0.367	0.426	0.366	0.277	0.352	0.382	0.416	0.386	0.381	0.045	0.471	0.291	12
15	0.576	0.568	0.495	0.588	0.491	0.402	0.472	0.503	0.579	0.530	0.520	0.059	0.639	0.402	11
16	0.670	0.642	0.572	0.666	0.551	0.525	0.540	0.549	0.636	0.595	0.595	0.055	0.704	0.485	9
17	0.571	0.546	0.542	0.587	0.497	0.431	0.473	0.490	0.516	0.500	0.515	0.047	0.610	0.421	9
18	0.377	0.343	0.326	0.390	0.338	0.278	0.305	0.315	0.354	0.351	0.338	0.033	0.404	0.271	10
19	0.227	0.223	0.205	0.244	0.211	0.176	0.201	0.197	0.216	0.192	0.209	0.019	0.248	0.171	9
20	0.122	0.145	0.118	0.142	0.117	0.115	0.122	0.118	0.125	0.109	0.123	0.011	0.146	0.100	9
21	0.062	0.081	0.061	0.076	0.061	0.065	0.059	0.050	0.066	0.047	0.063	0.010	0.083	0.042	16
22	0.087	0.108	0.083	0.087	0.090	0.090	0.083	0.079	0.083	0.084	0.087	0.008	0.103	0.071	9
23	0.144	0.156	0.124	0.136	0.137	0.122	0.127	0.130	0.135	0.131	0.134	0.010	0.154	0.114	7
24	0.199	0.205	0.183	0.197	0.165	0.142	0.173	0.183	0.205	0.188	0.184	0.020	0.223	0.144	11
25	0.272	0.268	0.237	0.284	0.236	0.182	0.231	0.235	0.272	0.252	0.247	0.030	0.306	0.187	12
26	0.341	0.331	0.298	0.342	0.272	0.236	0.283	0.271	0.349	0.303	0.303	0.038	0.378	0.227	12
27	0.336	0.322	0.299	0.332	0.269	0.237	0.284	0.254	0.329	0.290	0.295	0.035	0.365	0.226	12
28	0.251	0.241	0.237	0.271	0.216	0.182	0.232	0.205	0.255	0.231	0.232	0.026	0.284	0.180	11
29	0.161	0.176	0.173	0.182	0.155	0.136	0.169	0.152	0.178	0.155	0.163	0.014	0.192	0.134	9
30	0.113	0.127	0.126	0.129	0.122	0.109	0.121	0.111	0.131	0.103	0.119	0.010	0.138	0.100	8
31	0.056	0.079	0.094	0.084	0.085	0.073	0.081	0.071	0.083	0.063	0.077	0.011	0.099	0.054	15
Static	-0.002	-0.006	0.023	0.016	0.025	0.023	0.024	0.019	0.015	0.023	0.016	0.011	0.038	-0.006	69
Dynamic	0.352	0.350	0.302	0.348	0.282	0.258	0.281	0.287	0.322	0.294	0.308	0.033	0.375	0.241	11
Total	0.350	0.344	0.325	0.364	0.308	0.281	0.305	0.307	0.337	0.318	0.324	0.025	0.374	0.274	8
Top	0.531	0.514	0.504	0.557	0.464	0.452	0.465	0.470	0.499	0.483	0.494	0.033	0.561	0.427	7
Middle	0.326	0.320	0.290	0.334	0.286	0.245	0.274	0.283	0.311	0.292	0.296	0.027	0.350	0.242	9
Bottom	0.196	0.201	0.185	0.204	0.175	0.151	0.178	0.169	0.202	0.180	0.184	0.017	0.218	0.150	9

Note: s = standard deviation; COV = coefficient of variation

The average displacement of each row of panels is also very consistent. The coefficient of variation ranges from 7 percent for the top row to 9 percent for the bottom row. In every test, the average displacement for the top row was above, and for the middle and bottom rows was below the mean. A graphic representation of the mean, mean plus two standard deviations, and mean minus two standard deviations panel displacement is shown in Figure 16.

5.2.2 Pressures

The measured peak pressures for the free-field and interface pressure gages are shown in Table 4, along with the mean, standard deviation, coefficient of variation and mean plus and minus two standard deviations. The variability of the peak pressures is considerably higher than it is for the displacements. The coefficient of variation ranges from 14 to 40 percent. There is a consistency in the way variability differs, depending on the type of measurement made.

The soil interface pressures at the wall panels (measured by P1, P2, and P3) are the least consistent (coefficient of variation = 26 to 40 percent). Significant factors influencing the peak pressure at the soil-facing panel interface are the explosive, the soil between the explosive and the gage, the bedding of the soil around the gage, and the wall stiffness. Of these, the wall stiffness is likely to vary the most, because of difficulty in exactly duplicating the wall construction. While the range for the 95 percent confidence interval is large, it is a natural consequence of the inherent variability of the construction process, and a similar, if not larger, range would be expected in full-scale reliability testing.

The free-field gages in the soil (P4 and P5) are affected by all the same variables as listed above, except for wall stiffness. Because these gages are surrounded by soil, the bedding problem is somewhat different for the free-field gages, but not necessarily worse. With the elimination of the wall stiffness as a source of variability, the data from the free-field gages are more consistent than that from the wall interface gages (coefficient of variation = 16 to 20 percent).

The most consistent measurement is the soil interface pressure on the bucket wall (measured by P6) (coefficient of variation = 14 percent). This gage is affected by all the same variables as listed above for the free-field gages, except for the gage location. The increase in reliability when gage location is rigidly fixed (2 to 6 percent reduction in coefficient of variation)

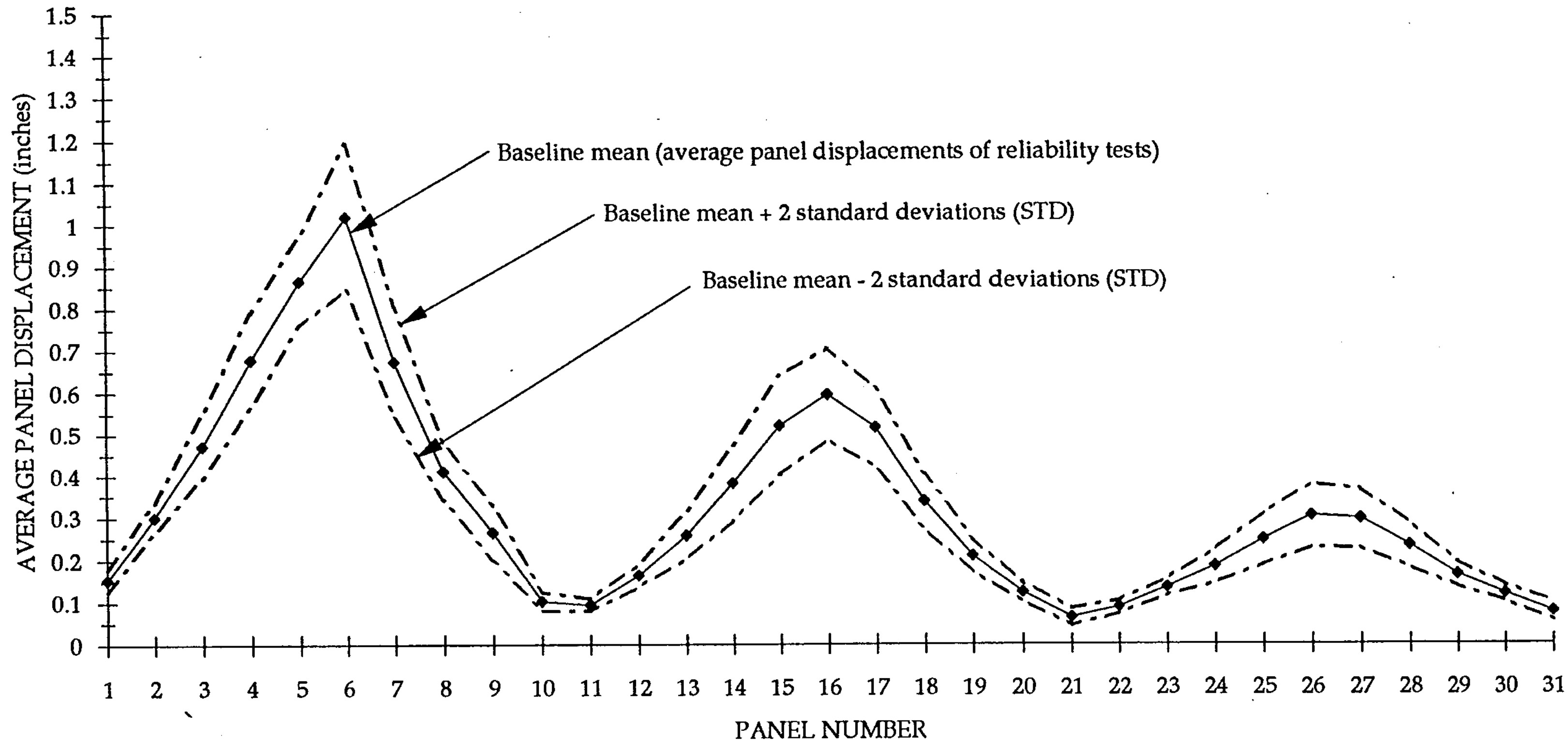


Figure 16. Average Panel Displacement and Statistical Limits of Random Variation: Reliability Test

TABLE 4. GAGE DATA: RELIABILITY TESTS

	GAGE NUMBER	GAGE DATA FOR RELIABILITY TEST #										STATISTICAL EVALUATION				
		7	8	9	10	11	12	13	14	15	16	Mean (inches)	s	Mean + 2s	Mean - 2s	COV (%)
Pk. Accel. (g)	A1	3748	2501	4049	6149	5036	5145	4275	4660	4827	FAIL	4488	1023	6533	2442	23
	A2	4089	3256	7408	8154	9755	FAIL	10072	9947	9187	11655	8169	2825	13819	2519	35
	A3	7921	3497	8101	13424	15820	9560	14676	11291	11252	FAIL	10616	3836	18287	2945	36
	A4	13557	16024	18183	FAIL	18180	18171	12919	13215	12019	13184	15050	2577	20203	9897	17
Pk. Pressure (psi)	P1	187	106	179	197	111	105	142	164	136	212	154	40	233	75	26
	P2	82	40	195	157	191	228	239	200	244	270	184	73	331	38	40
	P3	117	21	119	143	130	189	220	142	141	186	141	54	248	33	38
	P4	82	106	104	FAIL	141	130	136	152	111	151	124	24	172	75	20
	P5	286	329	233	291	354	269	FAIL	FAIL	276	383	303	49	401	205	16
	P6	-	-	-	-	-	137	153	177	198	164	166	23	212	120	14
Arrival Time (uSEC)	A1	447	475	450	420	441	449	438	446	433	FAIL	444	15	474	415	3
	A2	432	458	432	407	413	FAIL	412	435	422	413	425	16	457	393	4
	A3	445	481	445	432	428	432	422	445	434	FAIL	440	17	475	406	4
	A4	335	343	315	BAD	314	317	325	328	302	330	323	12	348	298	4
	P1	452	475	445	424	427	441	438	446	434	424	441	15	472	410	4
	P2	430	468	430	408	410	411	405	427	422	410	422	19	460	385	4
	P3	436	472	450	438	438	432	423	446	435	416	439	15	469	408	3
	P4	339	330	313	FAIL	310	310	314	319	299	318	317	12	340	293	4
	P5	168	160	169	175	172	167	FAIL	FAIL	152	175	167	8	183	152	5
	P6	-	-	-	-	-	407	428	420	399	431	417	14	444	390	3
Stand off (in)	A1	8.98	8.98	8.98	8.98	8.98	8.98	8.98	8.98	8.98	8.98	-	-	-	-	-
	A2	8.75	8.75	8.75	8.75	8.75	8.75	8.75	8.75	8.75	8.75	-	-	-	-	-
	A3	8.98	8.98	8.98	8.98	8.98	8.98	8.98	8.98	8.98	8.98	-	-	-	-	-
	A4	8.50	8.50	8.50	8.50	8.50	8.50	8.50	8.50	8.50	8.50	-	-	-	-	-
	P1	8.67	8.67	8.67	8.67	8.67	8.67	8.67	8.67	8.67	8.67	-	-	-	-	-
	P2	8.44	8.44	8.44	8.44	8.44	8.44	8.44	8.44	8.44	8.44	-	-	-	-	-
	P3	8.67	8.67	8.67	8.67	8.67	8.67	8.67	8.67	8.67	8.67	-	-	-	-	-
	P4	8.50	8.50	8.50	8.50	8.50	8.50	8.50	8.50	8.50	8.50	-	-	-	-	-
	P5	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	-	-	-	-	-
	P6	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	-	-	-	-	-
Wave Speed (fps)	A1	1674	1575	1663	1782	1697	1666	1708	1678	1728	FAIL	1686	56	1574	1797	3
	A2	1688	1593	1688	1792	1766	FAIL	1770	1677	1728	1766	1719	63	1592	1846	4
	A3	1681	1556	1681	1732	1748	1732	1773	1681	1724	FAIL	1701	63	1574	1828	4
	A4	2114	2065	2249	BAD	2256	2234	2179	2160	2345	2146	2194	85	2024	2365	4
	P1	1599	1521	1624	1704	1692	1639	1650	1620	1665	1704	1642	56	1530	1754	3
	P2	1635	1502	1635	1723	1715	1711	1736	1647	1666	1715	1669	70	1529	1808	4
	P3	1657	1531	1606	1650	1650	1673	1708	1620	1661	1737	1649	56	1537	1762	3
	P4	2089	2146	2263	FAIL	2285	2285	2256	2220	2369	2227	2238	82	2075	2401	4
	P5	2976	3125	2959	2857	2907	2994	FAIL	FAIL	3289	2857	2996	147	2702	3289	5
	P6	-	-	-	-	-	2048	1947	1984	2089	1933	2000	66	1868	2133	3
Crater Dim.	Dia. (in)	13	13	13	13	12.5	12.5	13	13	12.75	12	13	0	12	13	3
	Depth (in)	3	3.5	3.125	3.25	3.5	3.25	3.5	3.5	3.25	3	3	0	3	4	6

Note: s = standard deviation; COV = coefficient of variation indicates the importance of even small errors in gage placement.

5.2.3 Accelerations

The peak accelerations measured in the free field and on the center wall panels are shown in Table 4. Prototype accelerations are obtained by dividing model acceleration by 30, the g-level of the tests. As with the pressure data, the most variable measurements are on the wall. The coefficient of variation for the three accelerometers on the wall panels ranges from 23 to 36 percent, with the coefficient of variation for the average peak panel acceleration equal to 32 percent. These coefficients of variation are approximately the same as those for the wall interface pressures. Also, like the pressure data, the most reliable acceleration measurement on the wall is A1, located on the top row. The coefficient of variation for the free-field accelerometer is 17 percent, again almost exactly the same as for the free-field pressure measurements.

5.2.4 Wave Speeds

The wave speeds between the explosive and each of the gages were calculated for each reliability test. These wave speeds are shown in Table 4, along with the mean, standard deviation and mean plus and minus two standard deviations for each gage. Prototype wave speeds are the same as model wave speeds. The variability in wave speed measured to any individual gage is extremely small, as evidenced by coefficients of variation between 3 and 5 percent. Because the major sources of variability in wave speed between specific points are soil density and confined modulus, the minimal variability in wave speed indicates that the pluviation method used to prepare the sand backfill produces very consistent test specimens.

The wave speeds measured in the reinforced mass to the gages on the facing panels (gages A1, A2, A3, P1, P2 and P3) are essentially equal. Similarly, the wave speeds to the free-field accelerometer and pressure gage located 8.5 inches from the explosive (gages A4 and P4, respectively) are also nearly identical. However, the average speed to the free-field gages is approximately 500 fps faster than that to the gages on the facing panels. The average speed to the pressure gage closest to the detonator (P5) is even higher, by approximately an additional 750 fps. The average speed to gage P6, located 10 inches from the explosive, falls between the speed to the wall and the speed measured to A4 and P4.

The large variation in wave speed noted above is not fully understood at this time. Two contributing factors have been identified; however, it does not appear that they can completely explain the observed variation. A reduction in wave speed through the reinforced zone will be caused by friction developed between the reinforcement and the soil. A calculation of the expected magnitude of this reduction may be possible, but is beyond the scope of this investigation. The stiffness of the soil in the test container varies with direction due to boundary effects. The relative rigidity of the bucket side walls compared to that of the reinforced soil wall leads to lower stiffness in the direction toward the reinforced soil wall, thereby causing reduced wave speed. These two factors may explain qualitatively the lower wave speed through the reinforced soil, but do not seem adequate to explain the reduction in wave speed through unreinforced soil. Further centrifuge testing, preferably using a larger test container, will be needed to resolve these questions regarding wave speed.

5.2.5 Crater Dimensions

For each reliability test, crater diameter and depth were measured. In Table 4, the coefficients of variation for crater diameter and depth are three and six percent, respectively. These low values further illustrate the uniformity of the pluviated soil, as well as the reproducibility of the detonator effects. Because crater dimensions proved to be very reproducible, the dimensions were measured throughout the production testing program to aid in evaluating other data that proved to be less reliable in the centrifuge.

5.3 CONCLUSIONS

The results of the reliability study show that it is possible to have a high degree of reproducibility in the centrifuge for a complex dynamic soil-structure interaction test. They also show that within the same test, some system responses can be much more reproducible than others. In this work, wall displacements and measured wave speeds show much less variability than acceleration or pressures on the wall or in the soil.

Statistical analysis of the reliability test data has provided a range of system responses (displacements, wave speeds, pressures and accelerations) which can be expected due entirely to random variation. This information is used to evaluate the results of the production tests in which system parameters are systematically varied to determine which parameters significantly affect wall behavior. Only parameters which produce a system response (displacement, pressure, etc) with a low probability of occurring by random variation (5 percent for this study) will be considered significant.

SECTION 6

PRODUCTION TEST SERIES

6.1 INTRODUCTION

Sixteen production tests were conducted in which individual system parameters were systematically varied to investigate their effects on wall response. As in the reliability test series, the data collected for the production test series consisted of average panel residual displacements, peak pressures (free field and soil-facing panel interface), peak accelerations (free-field and soil-facing panel interface), and wave speeds. In the production test series, only two sets of panel displacement measurements were taken for each test, one set upon completion of model construction, and one set after detonation of the explosive. However, to minimize deviations in test procedure between the production and reliability tests, all production test models were accelerated to 30 g in the centrifuge, decelerated to approximately 1 g, then re-accelerated to 30 g prior to triggering the explosive event.

6.2 PARAMETRIC STUDY

6.2.1 General

The parametric study was conducted using the reliability test parameters as the baseline set of parameters from which variations would be made. For each production test, only one parameter was varied from the baseline so that any significant change in wall response could be attributed to that parameter. The exception to this rule was test 27, in which several parameters were varied from the baseline in an effort to simulate the parameters of a full scale reinforced soil wall that was also being tested. The production test matrix is presented in Table 5. Residual displacement, pressure, acceleration and wave speed measurements for each test are presented in Tables 6 and 7.

TABLE 5. PRODUCTION TEST MATRIX

Study	Test No.	Variation From Baseline
Effects of Reinforcement Coverage Area	22 23 24 28	4.2-inch long reinforcement 3 reinforcing strips/panel 1 reinforcing strip/panel 15-inch long reinforcement
Effects of Facing Panel Geometry	17 30 31	4 inch x 2 inch panels 4 inch x 1 inch panels 2 inch x 1 inch panels
Effects of Soil Type and Density	18 20 26 27	89 pcf Tyndall sand 90 pcf Sky X sand 95 pcf Tyndall sand 98 pcf Sky X sand (moist)
Effects of Overall Model Geometry	19 21 25	includes roof structure 6.5 inch standoff includes berm
Effects of Reinforcement Strength	29 32	reduce tensile strength introduce shear resistence between reinforcing strips

TABLE 6. DISPLACEMENT DATA: PRODUCTION TESTS

PANEL	AVERAGE PANEL DISPLACEMENTS (inches) FOR PRODUCTION TEST #															
NUMBER	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32
1	0.256	0.214	0.131	0.209	0.173	0.189	0.129	0.185	0.123	0.193	0.179	0.098	0.170	0.152	0.110	0.358
2	0.431	0.378	0.242	0.360	0.326	0.358	0.273	0.336	0.251	0.354	0.333	0.177	0.345	0.294	0.277	0.496
3	0.575	0.573	0.380	0.546	0.543	0.546	0.475	0.539	0.400	0.528	0.475	0.306	0.571	0.447	0.472	0.636
4	0.768	0.824	0.525	0.800	0.835	0.732	0.683	0.771	0.562	0.732	0.645	0.489	0.796	0.590	0.750	0.799
5	0.866	1.045	0.613	1.057	2.099	0.909	0.909	1.021	0.708	0.906	0.768	0.656	0.982	0.966	1.007	0.901
6	0.887	1.346	0.623	1.333	2.101	1.034	1.038	1.162	0.757	1.108	0.805	0.788	1.049	0.952	1.028	0.873
7	0.610	0.854	0.505	0.923	2.109	0.783	0.656	0.827	0.586	0.750	0.707	0.485	0.793	0.642	0.708	0.687
8	0.448	0.498	0.339	0.486	0.537	0.545	0.378	0.530	0.358	0.459	0.496	0.221	0.471	0.435	0.419	0.520
9	0.220	0.306	0.227	0.337	0.240	0.352	0.227	0.335	0.231	0.300	0.369	0.137	0.341	0.229	0.287	0.356
10	0.142	0.141	0.136	0.198	0.105	0.148	0.087	0.139	0.102	0.138	0.184	0.077	0.160	0.106	0.122	0.186
11	0.107	0.131	0.087	0.153	0.086	0.144	0.091	0.085	0.079	0.115	0.123	0.085	0.109	0.099	0.083	0.201
12	0.224	0.255	0.145	0.246	0.172	0.209	0.162	0.167	0.137	0.214	0.231	0.111	0.197	0.164	0.158	0.270
13	0.348	0.409	0.230	0.359	0.308	0.311	0.265	0.312	0.230	0.324	0.341	0.223	0.338	0.265	0.293	0.356
14	0.451	0.593	0.346	0.561	0.538	0.430	0.388	0.484	0.349	0.468	0.464	0.346	0.478	0.383	0.452	0.453
15	0.550	0.837	0.475	0.786	0.811	0.548	0.545	0.676	0.482	0.625	0.581	0.463	0.617	0.486	0.643	0.553
16	0.545	0.996	0.536	0.933	2.098	0.606	0.635	0.792	0.540	0.703	0.650	0.547	0.666	0.536	0.724	0.624
17	0.460	0.817	0.478	0.823	2.096	0.576	0.509	0.688	0.470	0.597	0.615	0.449	0.622	0.475	0.600	0.561
18	0.282	0.518	0.321	0.664	0.466	0.399	0.332	0.503	0.319	0.396	0.488	0.234	0.440	0.339	0.401	0.430
19	0.204	0.336	0.208	0.363	0.236	0.275	0.213	0.324	0.193	0.257	0.374	0.128	0.284	0.215	0.242	0.298
20	0.068	0.211	0.135	0.230	0.109	0.158	0.119	0.174	0.104	0.152	0.258	0.081	0.184	0.115	0.129	0.175
21	0.000	0.086	0.094	0.151	0.062	0.092	0.053	0.082	0.057	0.069	0.132	0.052	0.084	0.061	0.052	0.093
22	0.101	0.135	0.079	0.152	0.082	0.102	0.083	0.088	0.073	0.116	0.119	0.067	0.092	0.099	0.085	0.123
23	0.155	0.214	0.119	0.221	0.136	0.164	0.126	0.161	0.121	0.166	0.186	0.108	0.194	0.144	0.150	0.180
24	0.192	0.307	0.182	0.312	0.217	0.224	0.182	0.256	0.179	0.229	0.241	0.169	0.272	0.205	0.221	0.227
25	0.268	0.407	0.254	0.422	0.354	0.282	0.251	0.361	0.250	0.312	0.302	0.244	0.328	0.255	0.311	0.291
26	0.284	0.512	0.304	0.539	0.493	0.318	0.325	0.451	0.311	0.377	0.348	0.260	0.343	0.289	0.386	0.329
27	0.301	0.481	0.290	0.514	0.486	0.313	0.313	0.454	0.305	0.360	0.357	0.286	0.323	0.291	0.384	0.343
28	0.234	0.355	0.227	0.421	0.356	0.267	0.242	0.373	0.247	0.280	0.307	0.214	0.318	0.234	0.296	0.294
29	0.198	0.256	0.163	0.312	0.196	0.200	0.190	0.294	0.172	0.221	0.265	0.136	0.247	0.172	0.198	0.223
30	0.089	0.196	0.115	0.240	0.105	0.146	0.107	0.169	0.112	0.149	0.190	0.082	0.152	0.123	0.122	0.150
31	0.078	0.102	0.081	0.152	0.070	0.087	0.076	0.097	0.069	0.088	0.121	0.059	0.102	0.015	0.084	0.092
Avg Wall	0.345	0.462	0.277	0.477	0.598	0.369	0.324	0.414	0.286	0.377	0.376	0.251	0.389	0.315	0.361	0.389
Top Row	0.520	0.618	0.372	0.625	0.907	0.559	0.485	0.584	0.408	0.547	0.496	0.343	0.568	0.481	0.518	0.581
Mid Row	0.294	0.472	0.278	0.479	0.635	0.341	0.301	0.390	0.269	0.356	0.387	0.247	0.365	0.285	0.343	0.365
Bot Row	0.190	0.297	0.181	0.328	0.250	0.210	0.189	0.270	0.184	0.230	0.244	0.162	0.237	0.183	0.224	0.225

TABLE 7. GAGE DATA: PRODUCTION TESTS

	GAGE NUMBER	GAGE DATA FOR PRODUCTION TEST #															
		17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32
Pk. Accel. (g)	A1	5731	2535	4292	382	13228	3832	FAIL	FAIL	FAIL	3539	295	8107	3589	5288	4158	2041
	A2	6226	4644	6195	777	FAIL	4687	FAIL	FAIL	FAIL	4554	847	9401	3159	7422	3729	1860
	A3	9505	4126	7381	606	15544	9177	FAIL	FAIL	FAIL	-	439	7860	4445	FAIL	FAIL	4776
	A4	13196	5623	12649	1905	FAIL	FAIL	5938	13016	6858	FAIL	FAIL	FAIL	13177	13177	10640	10546
Pk. Pressure (psi)	P1	149	38	149	11	383	117	FAIL	FAIL	FAIL	80	6	103	59	196	93	88
	P2	129	94	162	17	408	124	FAIL	FAIL	FAIL	76	16	124	50	144	127	72
	P3	196	122	123	12	264	FAIL	FAIL	FAIL	FAIL	80	24	171	95	182	191	84
	P4	121	32	79	15	116	141	127	112	135	141	21	146	97	104	92	105
	P5	236	167	291	90	240	367	302	321	369	315	241	257	322	245	340	220
	P6	194	44	152	FAIL	153	-	-	-	-	-	-	-	222	FAIL	207	243
Arrival Time (usec)	A1	422	644	457	842	294	457	416	443	FAIL	436	787	345	420	359	389	422
	A2	414	588	457	749	FAIL	425	399	430	418	364	666	399	396	350	370	433
	A3	418	581	453	665	313	417	424	436	443	-	684	415	456	FAIL	FAIL	456
	A4	305	482	319	1254	FAIL	FAIL	338	329	297	FAIL	FAIL	FAIL	305	311	312	346
	P1	422	635	461	1383	296	459	384	447	FAIL	429	922	366	436	345	391	422
	P2	405	592	437	1282	288	422	376	380	389	413	772	356	414	356	372	430
	P3	420	575	452	1210	288	FAIL	398	415	426	422	660	376	410	359	365	416
	P4	301	489	319	1088	330	323	336	325	295	365	607	315	311	307	313	341
	P5	161	246	177	559	222	187	186	181	157	203	267	180	172	171	157	168
	P6	410	619	418	FAIL	444	-	-	-	-	-	-	-	482	FAIL	478	498
Stand off (in)	A1	8.98	8.98	8.98	8.98	7.04	8.98	8.98	8.98	8.98	8.98	10.45	8.98	8.98	8.98	8.98	8.98
	A2	8.75	8.75	8.75	8.75	6.75	8.75	8.75	8.75	8.75	8.75	10.25	8.75	8.75	8.75	8.75	8.75
	A3	8.98	8.98	8.98	8.98	7.04	8.98	8.98	8.98	8.98	8.98	10.45	8.98	8.98	8.98	8.98	8.98
	A4	8.50	8.50	8.50	8.50	8.50	8.50	8.50	8.50	8.50	8.50	10.00	8.50	8.50	8.50	8.50	8.50
	P1	8.67	8.67	8.67	8.67	6.74	8.67	8.67	8.67	8.67	8.67	10.14	8.67	8.67	8.67	8.67	8.67
	P2	8.44	8.44	8.44	8.44	6.44	8.44	8.44	8.44	8.44	8.44	9.94	8.44	8.44	8.44	8.44	8.44
	P3	8.67	8.67	8.67	8.67	6.74	8.67	8.67	8.67	8.67	8.67	10.14	8.67	8.67	8.67	8.67	8.67
	P4	8.50	8.50	8.50	8.50	8.50	8.50	8.50	8.50	8.50	8.50	10.00	8.50	8.50	8.50	8.50	8.50
	P5	6.00	6.00	6.00	6.00	6.50	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
	P6	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00
Wave Speed (fps)	A1	1773	1162	1637	889	1996	1637	1799	1689	FAIL	1716	1106	2169	1782	2084	1924	1773
	A2	1762	1241	1596	974	FAIL	1716	1828	1696	1745	2004	1283	1828	1842	2084	1971	1685
	A3	1790	1288	1652	1125	1875	1794	1765	1716	1689	-	1273	1803	1641	FAIL	FLM	1641
	A4	2322	1470	2220	565	FAIL	FAIL	2096	2153	2385	FAIL	FAIL	FAIL	2322	2278	2270	2047
	P1	1712	1138	1568	523	1898	1574	1882	1617	FAIL	1685	916	1974	1657	2095	1848	1712
	P2	1736	1188	1609	548	1863	1666	1870	1850	1808	1703	1073	1975	1698	1975	1890	1635
	P3	1721	1257	1599	597	1951	FAIL	FAIL	FAIL	1696	1712	1280	1922	1763	2013	1980	1737
	P4	2353	1449	2220	FAIL	2146	2193	2108	2179	2401	1941	FAIL	2249	2278	2307	2263	2077
	P5	3106	2033	2825	894	2440	2674	2688	2762	3185	2463	1873	2778	2907	2924	3185	2976
	P6	2033	1346	1994	-	1877	-	-	-	-	-	-	-	1729	FAIL	1743	1673
Crater Dim.	Dia. (in)	13	14	13	13.5	13	13	12.5	12	12.5	13	-	-	13	13	13	13
	Depth (in)	3	3	3	3.25	3.5	3.5	3.25	3.25	3.75	4	-	-	3.5	3.5	2.75	3.0

6.2.2 Effects of Reinforcement Length and Spacing

In the design of a reinforced soil wall for blast loading, a reasonable hypothesis is that wall behavior, measured mainly by displacement, will improve with increasing length and/or decreasing spacing of soil reinforcement. To determine how sensitive wall behavior is to total reinforcement frictional area, tests 22, 23, 24 and 28 were conducted. In these tests the total reinforcement frictional area varied from -50 percent to +150 percent of the baseline area. This was accomplished by changing the spacing of the reinforcing strips in tests 23 and 24 (3 strips per panel and 1 strip per panel, respectively) and changing the length of the reinforcement in tests 22 and 28 (4.2 inches and 15 inches, respectively).

The average total wall and row displacements are summarized in Table 8. The data are also presented in Figures 17 and 18, where they are compared to the 95 percent confidence range obtained from the reliability tests. When examining these data it should be noted that test 24 (one reinforcement layer per panel) represents an extreme minimum amount of reinforcement needed for static stability. This wall design does not satisfy standard static design criteria and would not, therefore, be used in any field application. Despite this, the wall behaved extremely well under blast loading. While the displacements exceeded the mean plus 2 standard deviation criterion, especially for the middle and lower rows, the wall was not breached and the wall remained stable after the dynamic loading.

TABLE 8. SUMMARY OF AVERAGE TOTAL WALL AND AVERAGE ROW DISPLACEMENTS: VARIABLE REINFORCEMENT SURFACE AREA

Test #	Change in Reinforcement Surface Area (%)	Average Total Wall Displacement (in)	Average Top Row Displacement (in)	Average Middle Row Displacement (in)	Average Bottom Row Displacement (in)
24	-50	0.414	0.584	0.390	0.270
22	-30	0.366	0.559	0.331	0.210
7-16	0 (Baseline)	0.324	0.494	0.296	0.184
23	+50	0.324	0.485	0.301	0.189
28	+150	0.251	0.343	0.247	0.162

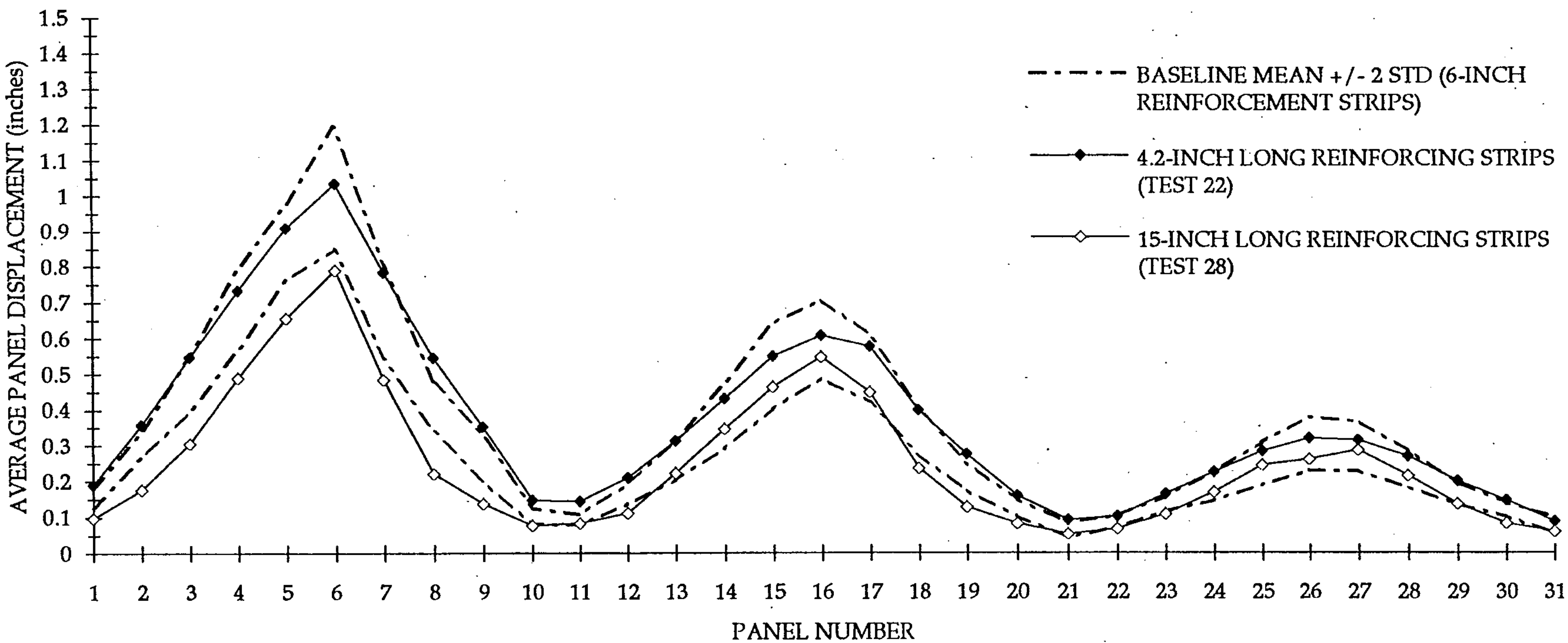


Figure 17. Average Panel Displacement: Reinforcement Surface Area Varied (Reinforcement Strip Length Varied)

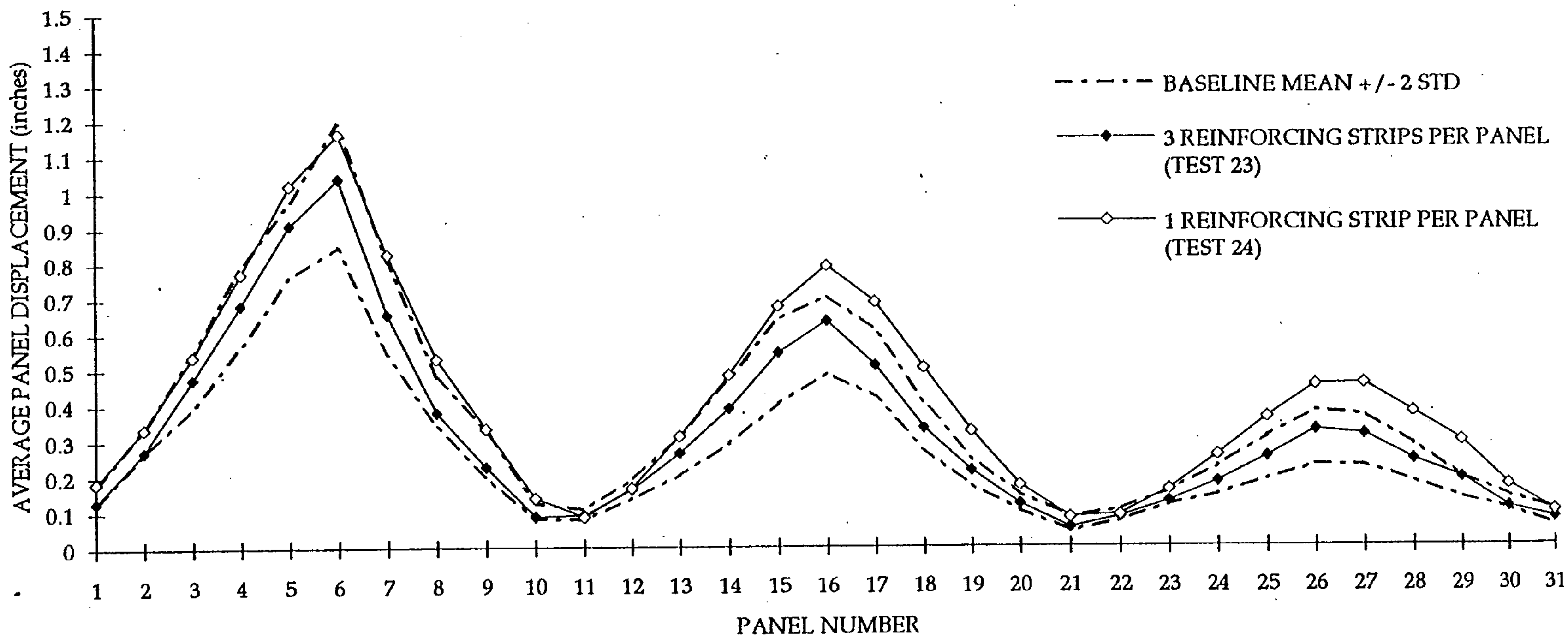


Figure 18. Average Panel Displacement: Reinforcement Surface Area Varied (Number of Reinforcing Strips Varied)

In the range of -30 percent to +50 percent of baseline total reinforcement frictional area, very little difference in wall displacement occurred. In fact, there is virtually no difference between the tests with two layers of reinforcement per panel (baseline) and 3 layers per panel (test 23). When the total reinforcement area is increased to 150 percent of baseline (test 28) the displacement of the upper row of panels is affected significantly; however, the average displacement of the middle and lower rows was only slightly lower than the baseline values, and well within the range for random variation.

It appears from these tests, shown in Figures 17 and 18, that the total frictional area of reinforcement has minimal influence on wall displacement. Only very long reinforcement lengths reduced displacement in the top row of panels and only very short reinforcement lengths increased displacement in the middle and bottom rows of panels. Based on the fact that a wall breach did not occur in the underdesigned test 24, standard static design criteria for designing soil reinforcement appear to be satisfactory for walls built to resist blast loading.

No interface pressure gage or accelerometer data were obtained for tests 23 and 24 due to a malfunction in the data acquisition system. Based on the data obtained from tests 22 and 28, it does not appear that interface pressures or accelerations are affected significantly by reinforcement length or reinforcement frictional area.

6.2.3 Effects of Facing Panel Geometry

Three tests were conducted in which the model facing panel geometry was varied from the baseline panel geometry of 2-inches long by 2-inches high (1 to 1 aspect ratio). Test 17 was conducted with 4-inch long by 2-inch high facing panels (2 to 1 aspect ratio), test 30 was conducted with 4-inch long by 1-inch high facing panels (4 to 1 aspect ratio), and test 31 was conducted with 2-inch long by 1-inch high facing panels (2 to 1 aspect ratio). With the exception of test 31, the results indicated there was no significant change in panel displacement from the baseline tests. This is shown in Figure 19. It was noted that when measuring panel displacements for these tests, the same procedure was followed as for the 2-inch long by 2-inch wide panels. To obtain displacement data that could be directly compared to the baseline panel displacements, the actual panel displacements were averaged as necessary to provide displacement data at the same locations as used in the reliability tests. The average displacements for test 31 were larger along the middle and bottom rows of panels, becoming significantly larger at the center of the middle and bottom rows.

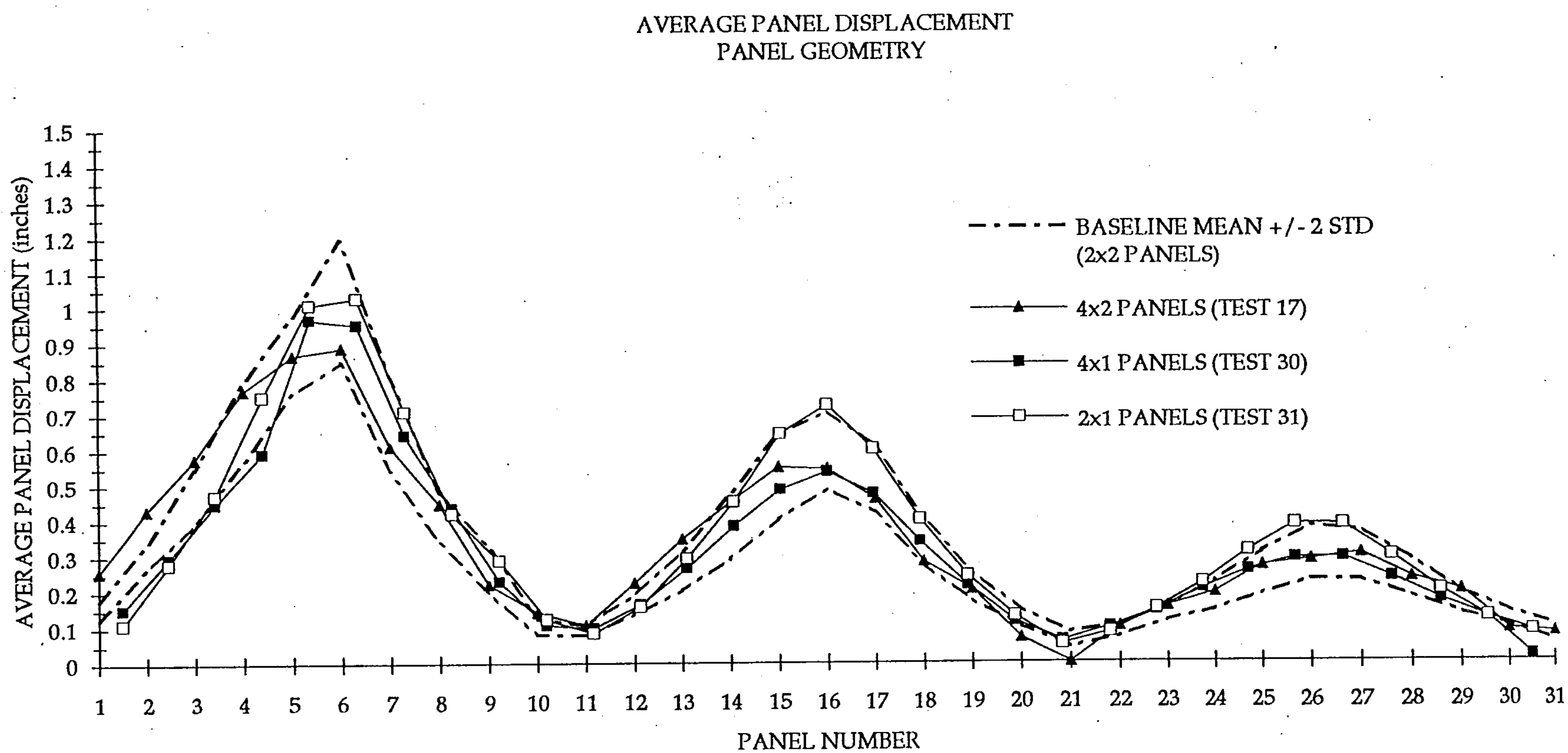


Figure 19. Average Panel Displacement: Panel Geometry Varied

To maintain the same reinforcement tensile area as in the baseline tests, the facing panels for tests 30 and 31 each had only one level of geogrid connected at the center of the panel (i.e., for test 30, two 2-inch wide reinforcing strips were epoxyed adjacent to each other along the horizontal centerline of the 4 inch by 1 inch panel; for test 31, one 2-inch wide strip was epoxyed along the horizontal centerline of the 2 inch by 1 inch panel). A connection of this type reduced the panel potential resisting moment from that of the baseline, where two geogrid strips per panel formed a moment couple. The fact that displacements were not reduced in tests 30 and 31, therefore, indicated that a reduction in the capability of the facing panels to resist rotation out of plane will not significantly reduce wall displacement.

All peak pressures and accelerations measured in tests 17, 30 and 31 were within the range of the reliability tests, indicating that panel geometry had no effect on peak pressures or accelerations.

Calculated wave speeds for all gages of test 17 were within the range of the reliability tests. For tests 30 and 31, wave speeds were within the range of the reliability tests for the free-field gages, but were significantly higher when calculated from the panel gages. Wave speeds calculated from the panel gages of tests 30 and 31 (P1, P2, P3, A1, and A2 [A3 failed during testing]) were greater than the mean plus two standard deviations of the reliability tests by 9 to 19 percent for P1, P2, P3, and 5 to 12 percent for A1 and A2 (A3 failed). The peak pressure at the bucket wall (P6) for test 32 was significantly lower than the mean minus two standard deviations of the reliability tests by 7 percent. The reasons for these wave velocity effects are unclear, especially since the same phenomenon did not occur in test 17.

6.2.4 Effects of Soil Density and Type

Tests 18, 20, 26 and 27 were used to evaluate the effects of soil density and type. Including the baseline tests, three densities of Tyndall Beach sand (89, 95 and 103 pcf) were tested. These tests cover nearly the full range from minimum to maximum relative density for this soil. A dry density as high as 105 pcf is obtainable by pluviation; however, 103 pcf is approximately the upper limit achievable in the field. A second sand, obtained from the Sky X test site at Tyndall AFB (described in Section III), was used to investigate the sensitivity of wall response to small changes in soil composition. These tests also contribute to our understanding of the effects of soil density.

6.2.4.1 Effects of Soil Density

The average of all wall panel residual displacements for tests 18 (89 pcf) and 26 (95 pcf) are 0.462 inches and 0.377 inches, respectively. This compares to a mean displacement for the baseline test of 0.274 inches and a mean plus two standard deviations of 0.374 inches. The effect of soil density on wall displacement is shown graphically in Figure 20. The displacement profile for test 18 (89 pcf) falls completely outside the 95 percent confidence interval for the baseline test and test 26 (95 pcf) falls almost exactly on the mean plus two standard deviation line. When the average panel residual displacements of tests 18 and 26 are compared to the mean of the reliability tests, as shown in Figure 21, it is clear that, as soil density increases, displacements of all 31 panels decrease. These results clearly show that soil density is a significant parameter affecting wall displacement.

The peak panel pressures for tests 18 and 26 are presented in Table 7. The average pressures on the wall are 85 and 79 psi, respectively, compared to a mean of 160 psi for the baseline tests. Because of the large variability in the pressure data, these results do not fall outside the 95 percent confidence interval determined in the reliability tests; however, they are approximately half the baseline mean. These results suggest that as soil density increases peak pressure on the wall panels will increase, although panel displacement will decrease.

The peak panel accelerations for tests 18 and 26 are presented in Table 7. Like the pressure data, panel accelerations for both walls are considerably below the mean of the baseline test, although they are higher than the mean minus two standard deviations. Two accelerometers failed during test 26, limiting the comparison between the 89 and 95 pcf tests.

The wave speed data for tests 18 and 26 are presented in Table 7. As expected, wave speeds measured in test 18 are significantly slower than those measured in the baseline tests, reflecting the difference between a sand compacted near its minimum dry density to the same compacted near its maximum dry density. The results from test 26 indicate the measured wave speeds are generally slightly above the mean wave speeds measured in the baseline tests. Considering the difference in dry density (95 pcf from test 26, 103 pcf for the baseline test series), the wave speeds are higher than expected. Without further testing it is not possible to determine if this unexpected result is due to random variation, or has some other explanation.

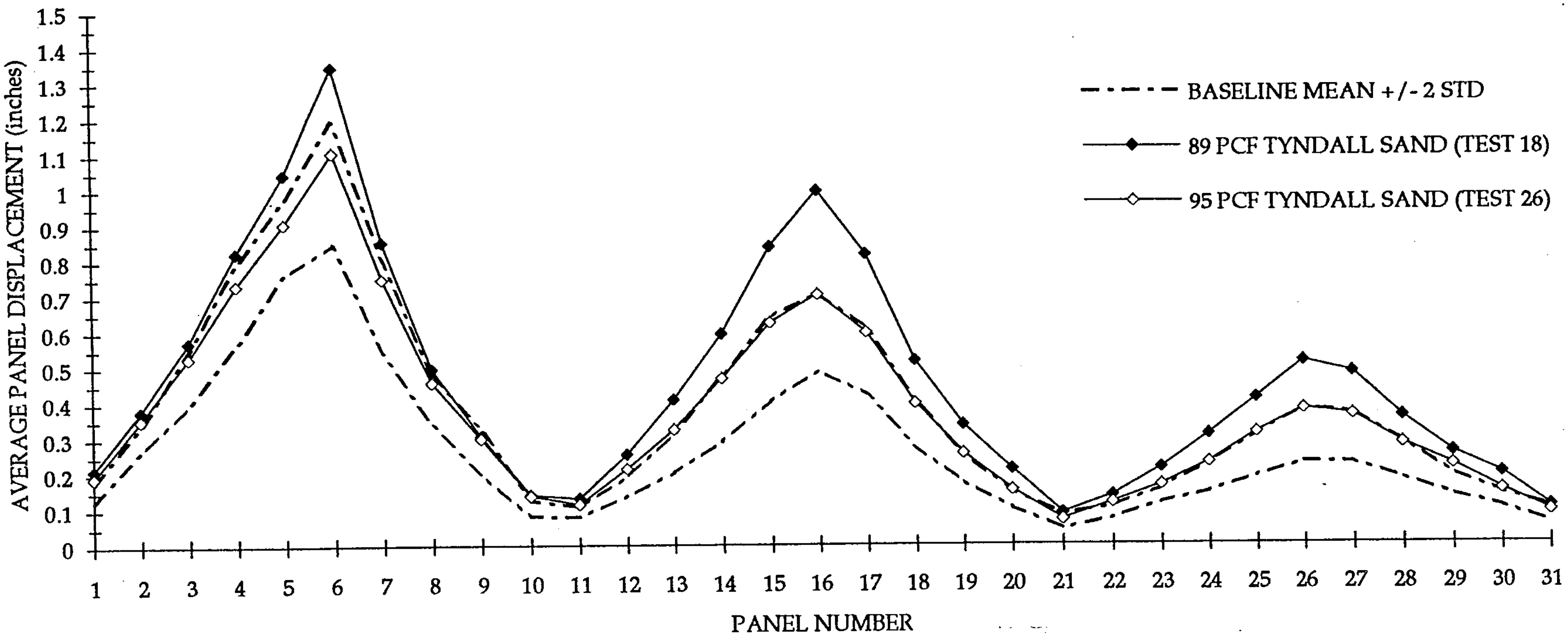


Figure 20. Average Panel Displacement: Soil Density Varied

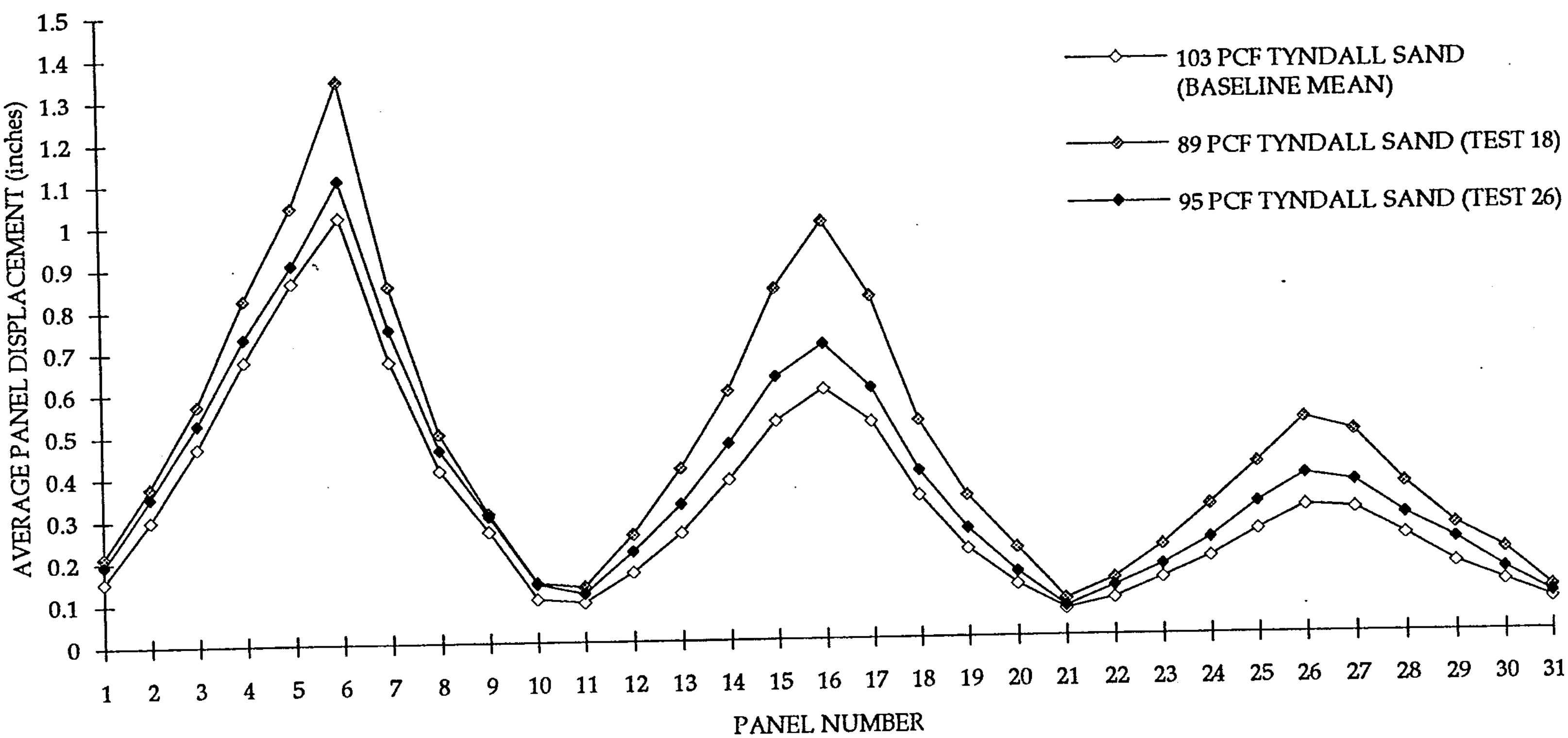


Figure 21. Average Panel Displacement: 89, 95, 103 pcf Tyndall Sand

6.2.4.2 Effects of Soil Type

Tests on Sky X sand (20 and 27) strongly support the results of the Tyndall Beach sand density tests (18 and 26). As shown in Table 6, the average wall displacement for Sky X sand at 90 pcf is almost exactly the same as that for Tyndall Beach sand at 89 pcf (0.477 vs. 0.462). The average row displacements are also nearly identical. Figure 22 presents this comparison graphically.

The comparison between test 27 (moist Sky X sand at a dry density of 98 pcf) and test 26 (Tyndall Beach sand at a dry density of 95 pcf) is shown in Figure 23. The comparison between these two tests is not straightforward because there is a difference in both water content and dry density (test 27 was designed to model a full-scale test in progress). Therefore, the similarity in residual displacements cannot necessarily be attributed to the dry density alone. Additional tests on moist Tyndall Beach sand would be required to determine the effects of moisture.

In Table 7, there is a striking difference in peak pressures measured in the two Sky X tests, compared to those from the tests in Tyndall Beach sand. The peak interface pressures in the Sky X sand are all below approximately 25 psi. This is much lower than the pressures recorded in even the low-density Tyndall Beach sand (test 18). The difference is not quite so dramatic when comparing the free-field pressures; however, three of the four gages in the Sky X tests recorded peak pressures much lower than those recorded in the low-density Tyndall Beach sand. At this time, the reasons for this very low pressure in Sky X sand are not known. The pressures, however, do compare favorably with pressures measured in the full-scale tests conducted at Tyndall AFB. The peak pressures measured on the top center, middle center, and bottom center panels for the full-scale test were 16, 17.5, 16 psi, respectively. The peak pressures measured on the top center, middle center, and bottom center panels for test 27 were 6, 16, and 24 psi, respectively.

The accelerations measured in test 27, shown in Table 7, are also much lower than those measured in any of the Tyndall Beach sand tests. As with the pressure data, it is not clear why such a large difference should occur in two sands which are, by standard measurements, nearly identical. It is possible that the presence of 2 percent fines in the Sky X sand is responsible, although this is not considered probable. A second difference between the two sands can be observed under a scanning electron microscope (SEM). The Sky X sand particles have much rougher surfaces, caused partially by the adherence of a nonquartz material.

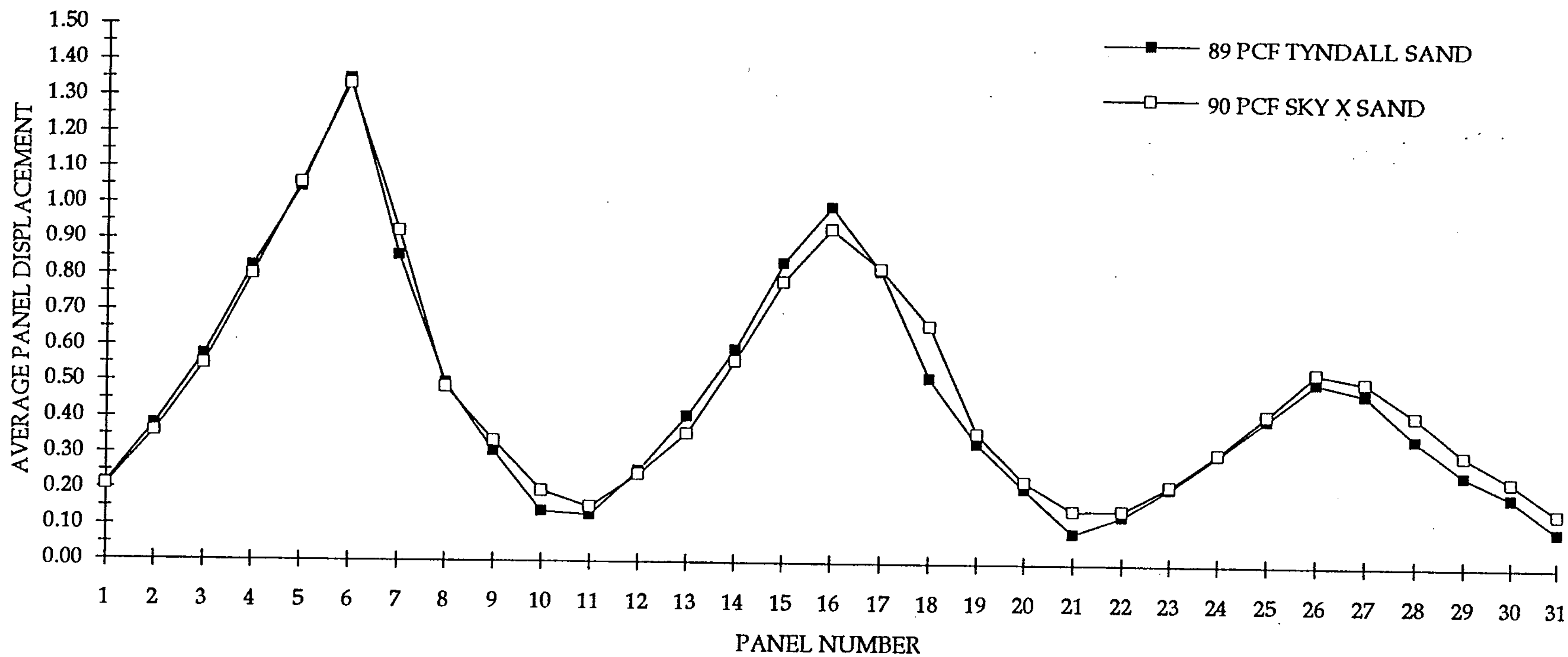


Figure 22: Average Panel Displacement: Soil Type Varied

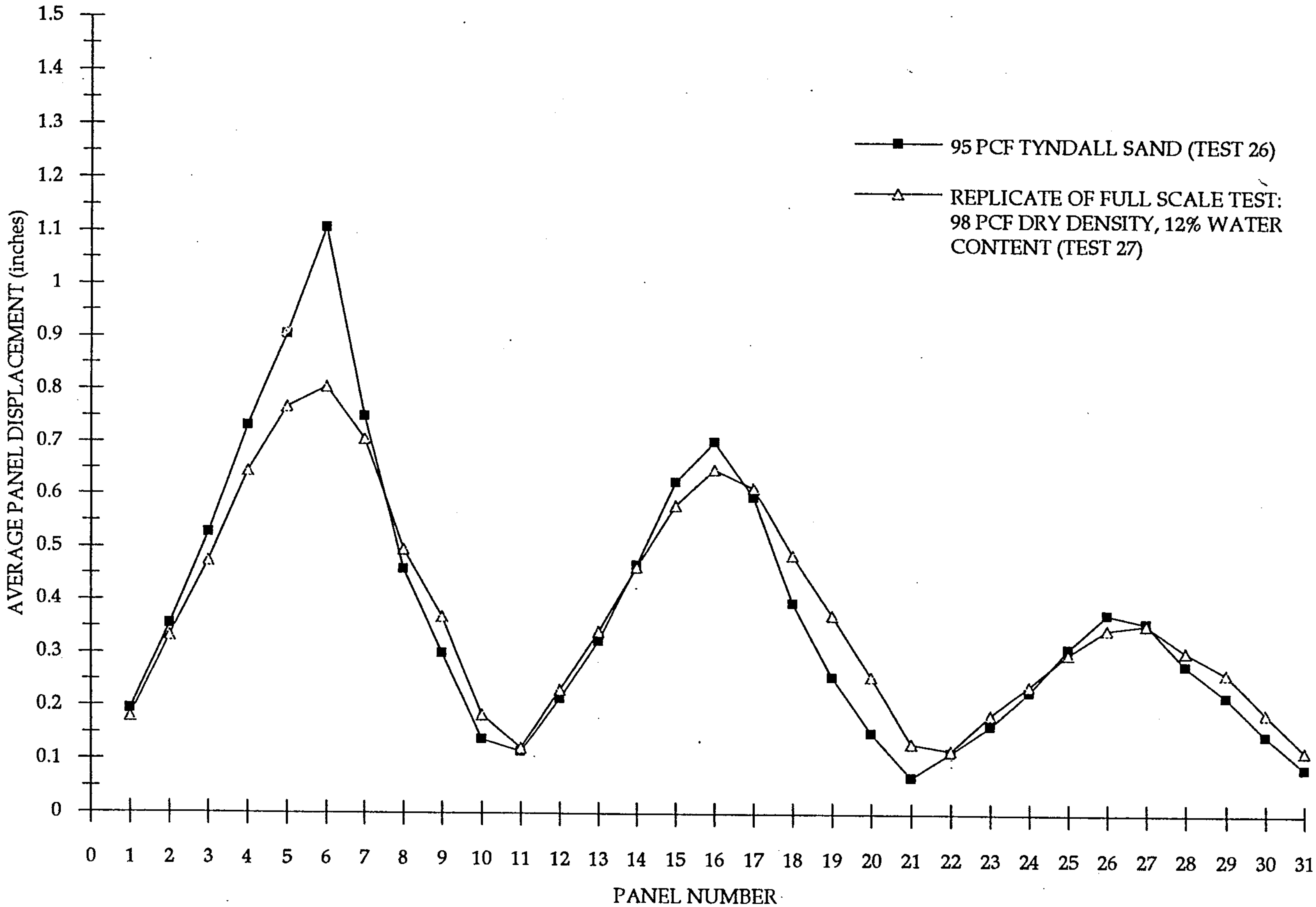


Figure 23. Average Panel Displacement: Soil Desnity and Water Content Varied

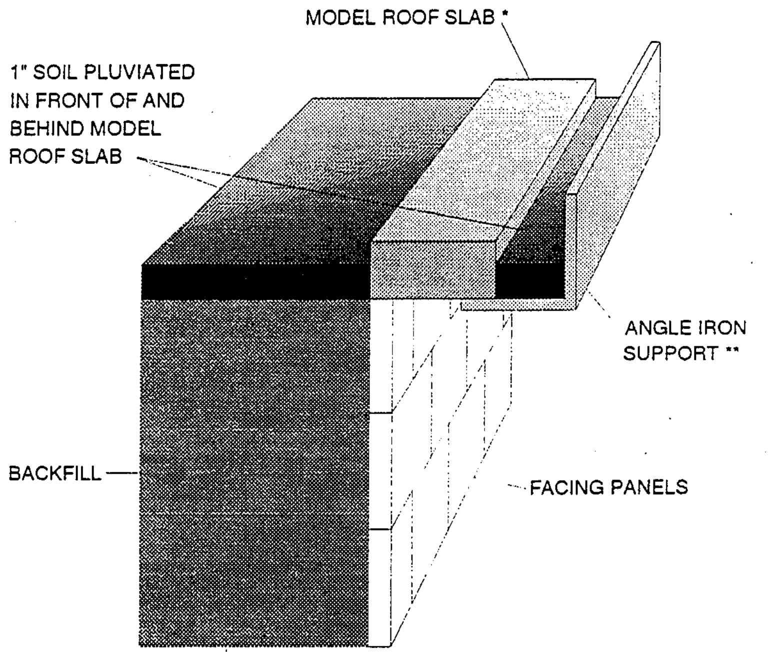
Wave speeds measured in the Sky X tests are presented in Table 7. As with pressure and acceleration measurements, Sky X test 27 wave speeds are significantly lower than Tyndall Beach sand results at approximately the same dry density. Wave speeds as low as 523 fps were measured in the low-density Sky X test, compared to a minimum speed of 1138 fps measured in the lowest-density Tyndall Beach sand test. The wave speeds measured using the soil-interface pressure gages are almost half that measured using panel accelerometer data. Further analysis of the acceleration data is necessary to resolve this discrepancy, and should also be included in follow-up work.

6.2.5 Effects of Overall Model Geometry

Three tests were conducted in which changes were made to the overall geometry of the system. The model for test 19 was constructed with a roof structure above it, as well as additional soil overburden. The model for test 21 was constructed with the detonator standoff at 6.5 inches behind the wall. The model for test 25 was constructed with a partially sloped backfill surface. The effects of these model variations on wall response is discussed below.

6.2.5.1 Effects of Roof Structure

The model roof was designed as a simply supported, one-way slab (Figure 24). One-half of the slab weight was supported by the reinforced soil wall, and the other half by a 1-inch angle iron bolted to the sides of the sample bucket, so that the span length from reinforced soil wall to angle iron was 1 inch. The mass of the model roof slab was intended to simulate that of a prototype reinforced concrete roof slab with a span length of 20 feet, width of 50 feet, thickness of 1 foot, and 2.5 feet of soil (at 103 pcf) above the roof. The calculated model roof mass simulating this loading was 6.52 kilograms (14.4 pounds). The model roof consisted of a steel bar 19.75 inches long by 1 inch wide by 2.56 inches thick. The mass of the bar was slightly less than the required mass, and aluminum shims were epoxyed to the bar to increase its mass appropriately. The bar was set in place between the top of the completed reinforced soil wall and angle iron as shown in Figure 24. One inch of soil was pluviated behind and in front of the steel bar. The effect of the roof structure on the wall response is discussed below.



* MODEL ROOF SLAB MASS IS EQUAL TO MASS OF ROOF SLAB PLUS MASS OF 2.5 FOOT-THICK SOIL OVERBURDEN.

** ANGLE IRON SUPPORT IS BOLTED TO THE SIDES OF THE SAMPLE CONTAINMENT BUCKET.

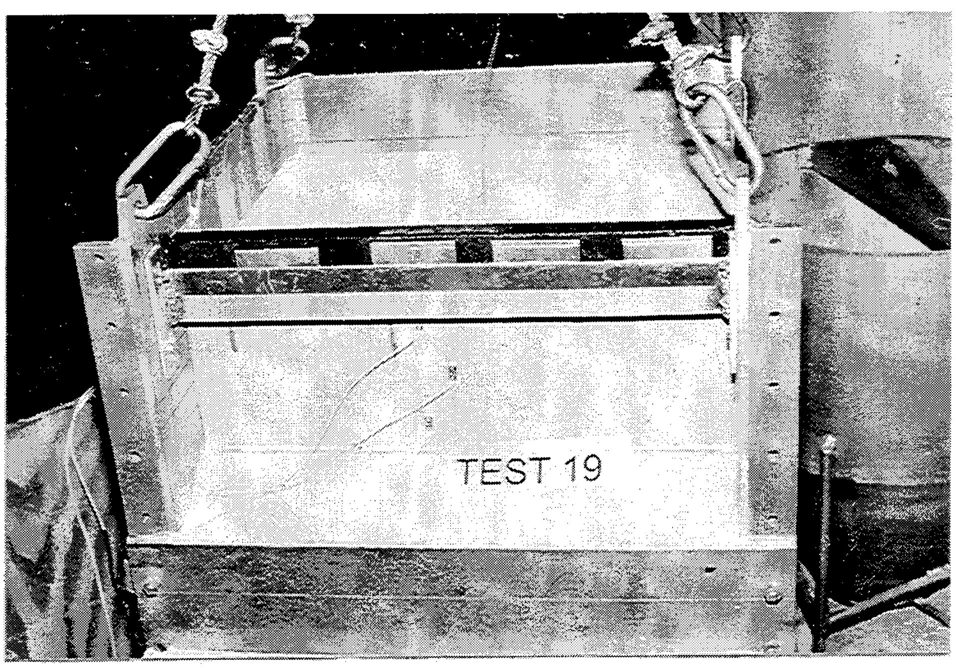


Figure 24. Roof Structure Over Reinforced Soil Wall (Simulated and Actual)

Figure 25 presents the average panel residual displacements for test 19. When compared to the baseline residual displacement data from the reliability test series it is apparent that the residual displacements of the middle and bottom rows of panels were not affected by the presence of the roof structure, and only the top row of panels showed significantly less residual displacement.

Peak pressures on the three central facing panels were 149, 162, and 123 psi at the top, middle and bottom facing panels, respectively. The peak pressure at the top panel was similar to those of the reliability tests, and did not appear to be influenced by the presence of the roof structure.

Peak accelerations of the three central facing panels were 4292, 6195, and 7381 g, for the top, middle and bottom facing panels, respectively. The peak acceleration at the top panel was similar to those of the reliability tests, and did not appear to be influenced by the presence of the roof structure.

Wave velocities calculated from all gages in test 19 were within the range of the reliability tests. Wave speeds calculated from all gages except A4 were slightly lower than the mean wave speeds calculated for the reliability tests. The wave speed calculated from A4 was slightly above the mean for the reliability tests. Slightly higher speeds were anticipated, since the addition of a roof structure and overburden increased the confining pressure in the backfill soil.

6.2.5.2. Effects of Blast Location

Test 21 was conducted with the detonator located 6.5 inches behind the wall facing panels. The reduced standoff distance resulted in greater average panel displacements than those of the baseline tests. For all except seven facing panels (side panels 9, 11, 20, 21, 22, 30, and 31) average panel residual displacements for test 21 were larger than the mean of the average panel residual displacements of the reliability tests. All central facing panels for test 21 (i.e., panels 4, through 8, 14 through 18, and 25 through 29) showed a statistically significant increase in residual displacement over those of the reliability tests (see Figure 26). The wall was breached at panels 5, 6, 7, 15, and 16. The fact that average panel residual displacements of the side facing panels mentioned above were not larger than the mean for the reliability tests is not

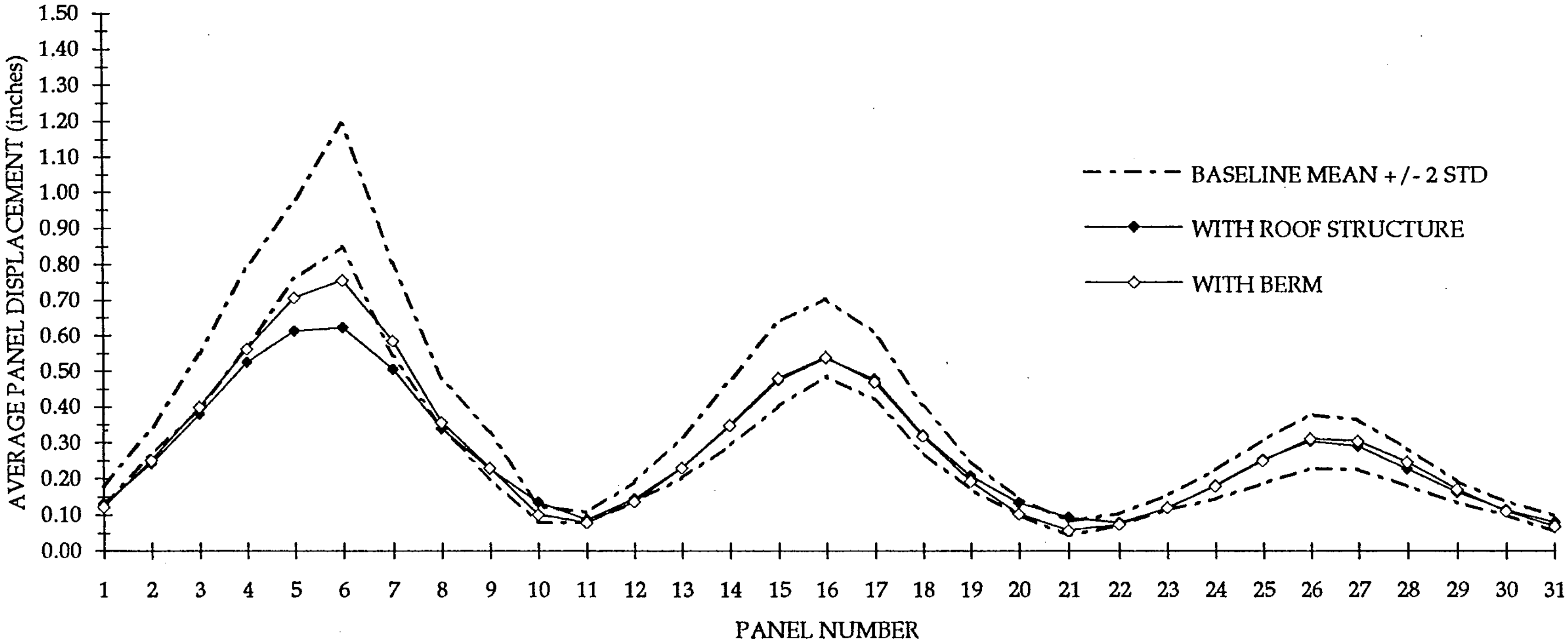


Figure 25. Average Panel Displacement: Overall Model Geometry Varied

significant. Although the detonator was closer to the wall for this test than for the reliability tests, the amplitude of the wave may have decayed sufficiently to cause no significant change in residual displacement over those in the reliability tests at these locations. Also, facing panels near the sample bucket sides are always influenced by boundary effects, specifically by lateral restraint from the sample bucket walls. The amount of lateral restraint varied slightly from test to test, depending on the overall tightness of the constructed wall and the amount of putty that was used to seal the end panels.

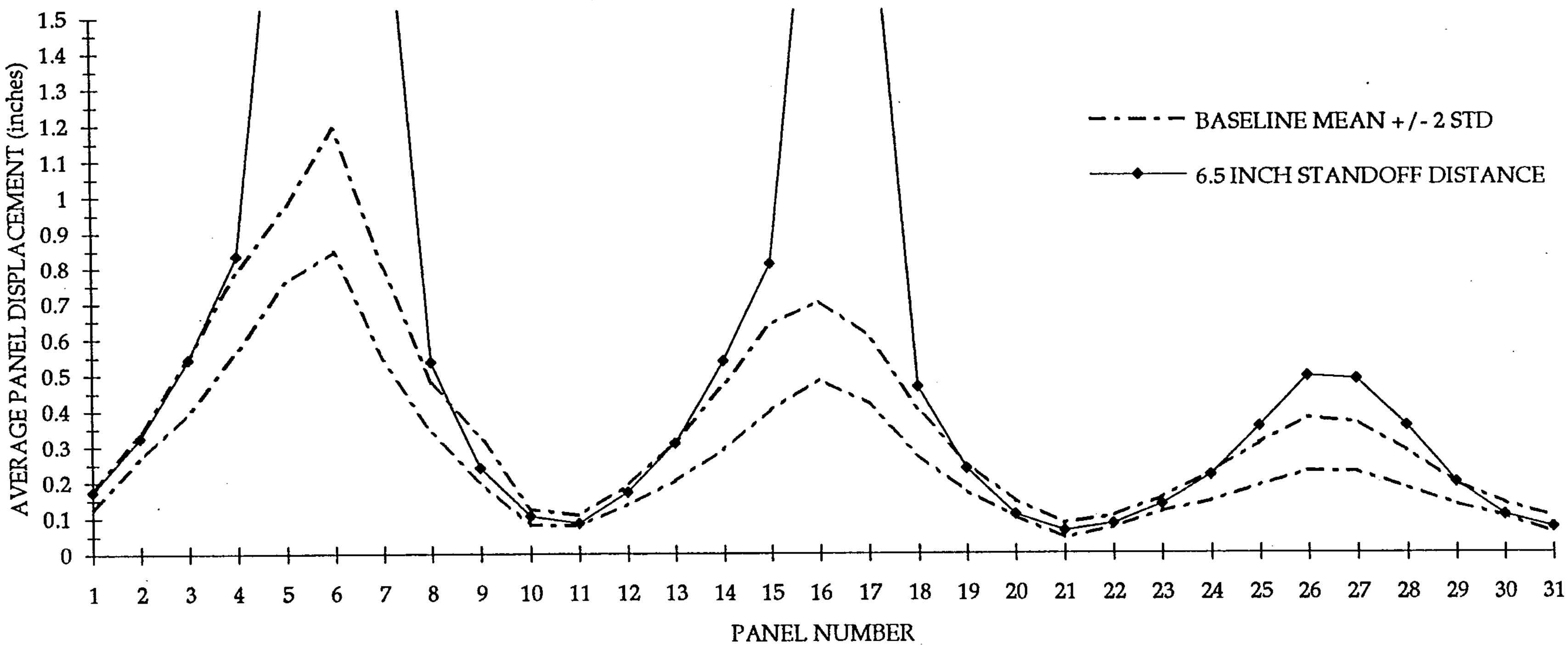
Peak pressures on the center facing panels in test 21 were significantly higher than those of the reliability tests. Peak pressures at the facing panels were 190 percent to 250 percent larger than the mean for the reliability tests. Peak free-field pressure measured at P4, P5, and P6 (located at 8.5, 6.5, and 10.0 inches from the detonator, respectively) showed no significant deviation from the free-field pressures recorded at these distances in the reliability tests.

Peak panel accelerations on the top center and bottom center facing panels (A1 and A3, respectively) in test 21 were both larger than those of the reliability tests by several thousand g. However, due to the large variation in peak accelerations measured in the reliability tests, only the peak panel acceleration at A1 was significantly larger than the reliability tests. Accelerometers A2 and A4 failed during test 21, and comparisons to the reliability tests can not be made.

Wave speeds calculated from the panel gages (P1, P2, P3, A1, A2, and A3) in test 21 were significantly higher than the mean of those calculated from the same gages in the reliability tests. Waves speeds calculated from the free-field gage P4 and soil-interface gage on the sample bucket wall (P6) were close to those for the reliability tests. The wave speed calculated from gage P5 cannot be directly compared to that of the reliability tests because it was located 6.5 inches from the detonator in test 21. However, the wave speed for this gage was slightly lower in test 21 than the mean for the reliability tests, which further indicates that wave speed decreases with increasing distance from the detonator.

6.2.5.3. Effects of Slope

Test 25 was constructed with the backfill surface partially sloped away from the wall. The slope was constructed to reduce the confinement of the blast (i.e., coupling factor) and



Note: Wall breached at panels 5,6,7,16, and 17.

Figure 26. Average Panel Displacement: Standoff Distance Varied

the amount of explosive energy exiting the system. The model was constructed in the same manner as for all other tests. After pluviation of the last lift of soil, the slope was constructed by pushing a thin aluminum plate with a sharpened edge into the backfill at approximately 22° to horizontal, sloping away from the model wall. The soil above the plate was then vacuumed out of the bucket. The slope crest was 1.5 inches behind the location of the detonator (see Figure 27).

Like the roof structure, the presence of the backfill slope reduced the residual displacements of the top row of panels only. The residual displacements of the middle and bottom rows of facing panels were statistically unchanged from those of the reliability tests. This is shown graphically in Figure 25. It is possible that significant reduction in residual displacement occurred for the top row only because the backfill slope height was 2.25 inches, extending only one quarter inch into the zone of the second row of panels. If it had been possible to extend the slope to meet grade at the same elevation as the base of the wall, as would be the case in a real reinforced soil structure, significantly reduced panel residual displacements might have occurred in the middle and bottom rows also. Space limitations prevented such a backfill slope in the centrifuge model.

Peak panel pressures (P1, P2, and P3) could not be measured in test 25 due to a malfunction in the data acquisition system. However, free-field pressures (P4, and P5) showed no statistically significant difference from those at the reliability tests.

Peak panel accelerations (A1, A2, and A3) also could not be measured in test 25 due to the malfunction in the data acquisition system. However, the free-field acceleration measurement (A4) for test 25 was significantly less than that of the reliability tests by approximately 3000 g. This suggests that peak free-field acceleration may be affected by the presence of the backfill slope while peak free-field pressures may not.

Calculated wave speeds for all gages in test 25 fell within the range of those in the reliability tests, indicating that the presence of the backfill slope had no effect on wave speed.

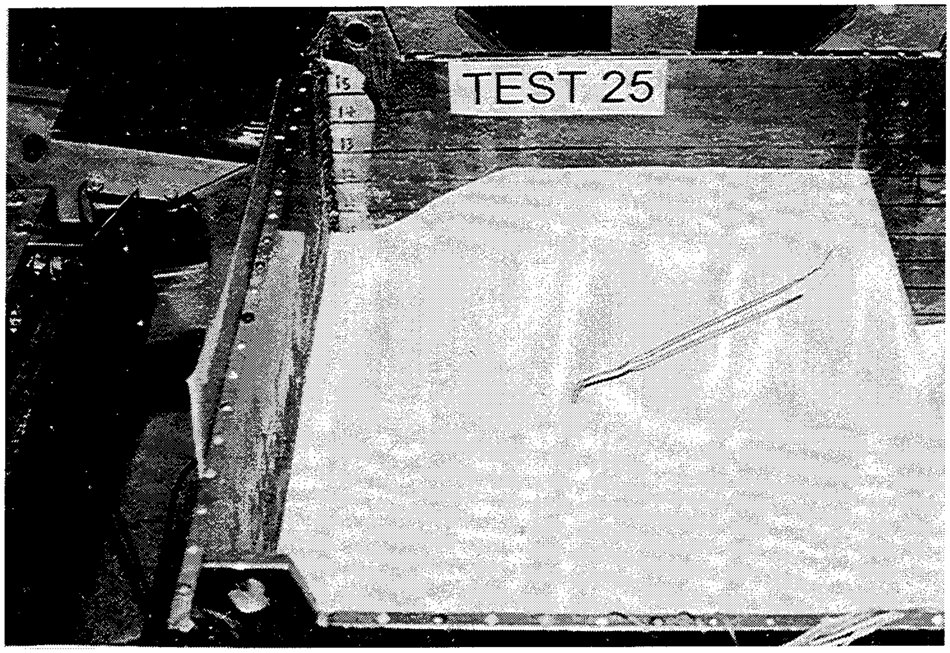


Figure 27. Soil Backfill with Slope

6.2.6 Effects of Reinforcement Strength

Two tests were conducted in which the overall strength of the reinforcement material was qualitatively altered. For test 29, seventy 1/4-inch diameter holes were punched in each reinforcing strip. The holes were punched in a regular pattern to equally reduce the tensile strength of each strip. For test 32, one continuous piece of reinforcing material (20 inches wide by 6 inches long) was used at each reinforcement level. The overall frictional area of reinforcement was conserved (i.e., the baseline reinforcement consisted of ten 2-inch wide strips per reinforcing level). The continuous reinforcing sheet added shear strength to the reinforcing between adjacent panels and possibly distributed the blast loading more uniformly throughout the length of the wall. The results of these two tests are discussed below.

6.2.6.1 Effects of Reduced Tensile Strength

The average panel residual displacements for test 29 are presented in Table 6, and are shown graphically in Figure 28. All centrally located panels (i.e., panels 6, 15, 16, 26, and 27) exhibited residual displacements close to the mean plus two standard deviations of those at the reliability tests, and all remaining panels displaced significantly more than the mean plus two standard deviations of the reliability tests. All panels displaced more in test 29 than the mean of the reliability tests. This implies statistical significance, because if the measured displacements were merely a result of random variation, it would be expected that some points fall above the mean and some below. The average row displacements for test 29 were also statistically larger than for the reliability tests. Although panel displacements were significantly large in test 29, they were not as large as those measured in test 18 (low-density Tyndall Beach sand). The actual reduction in tensile strength of the reinforcement strips for test 29 could not be quantified, but based on the number of holes were punched into each strip, it is reasonable to assume that the strength was severely reduced. Based on this assumption, it appears that wall response is governed more by soil density then by reinforcement strength. This observation is based on very limited data though, and additional testing should be conducted to verify it.

Peak accelerations and peak pressures measured in test 29 were close to those of the reliability tests, except for the pressure measured at soil interface gage P1 (on top center panel), which was slightly low, and soil-interface gage P6 (on side bucket wall), which was slightly high.

All wave speeds measured in test 29 were within the range of random variation when compared to the reliability tests, except for the wave speed measured to soil-interface gage P6, which was significantly low.

6.2.6.2 Effects of Reinforcement Shear Strength Between Panels

The average panel residual displacements from test 32 are presented in Table 6 and shown graphically in Figure 28. These data show that side panels displaced significantly for all three rows of panels. This was expected, since the continuous sheet of reinforcement probably distributed the dynamic loading more evenly throughout the length of the wall. However, if more uniform load distribution results in larger residual displacements at the side panels, it is reasonable to expect residual displacement at the center panels to be less. This was not the case. The four centermost panels for all top, middle, and bottom rows (with the exception of Panel 6) displaced more than the mean of the reliability tests (but within the range of random variation). Panel 6 displaced less than the mean of the reliability tests, (but within the range of random variation). The average wall residual displacement and average residual

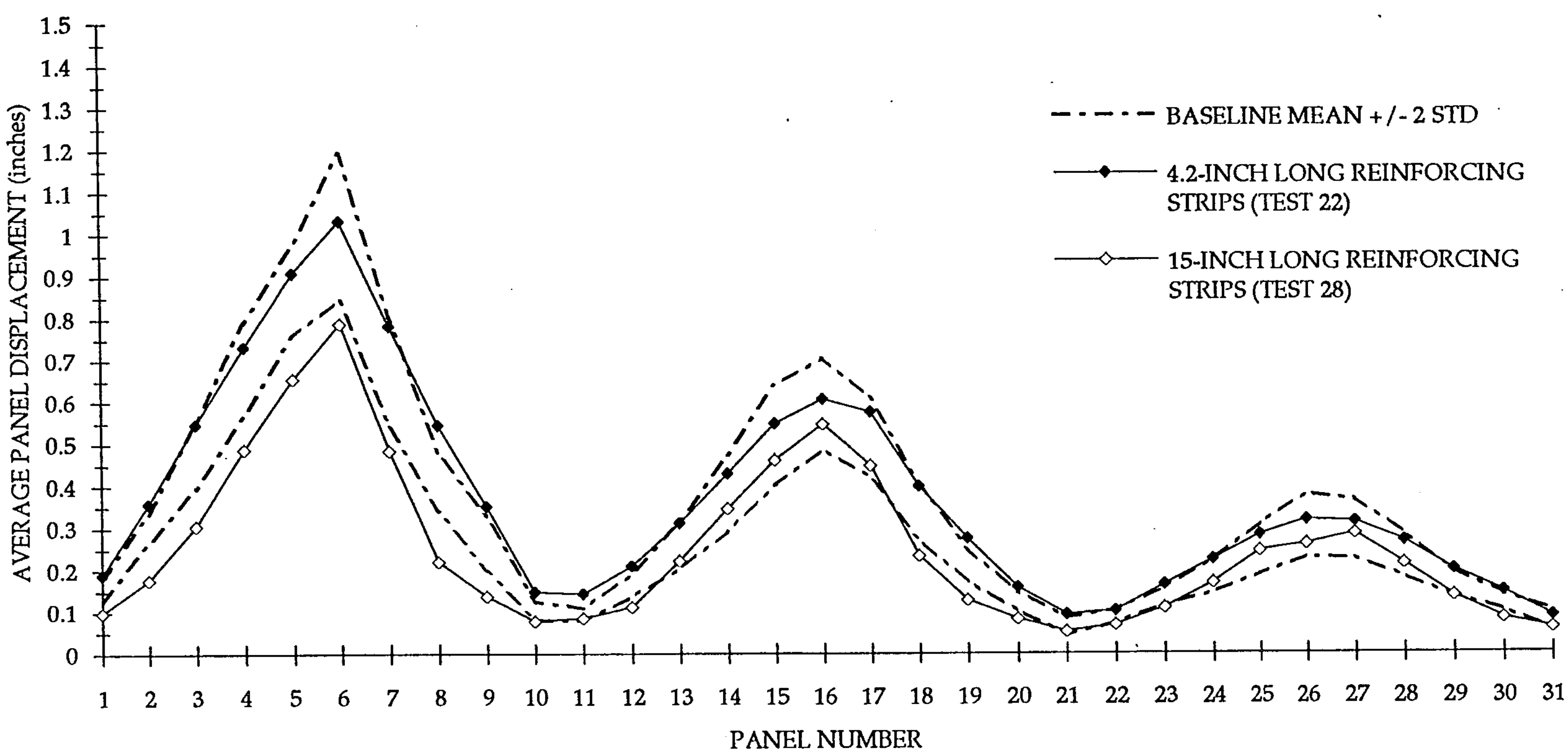


Figure 28. Average Panel Displacement: Reinforcement Properties Varied

displacements for all rows were all significantly larger than for the reliability tests. These data indicate that the continuous reinforcement may significantly increase residual displacements at the sides of the reinforced soil wall, but may not decrease residual displacements at the center of the wall.

All peak accelerations and peak pressures in test 32, except the peak pressure measured at gage P6, were below the mean of the reliability tests. Peak accelerations measured at accelerometers A1 and A2 (top and middle center panels) were significantly lower than those in the reliability tests. The peak pressure measured at soil interface gage P6 was significantly higher than that in the reliability tests.

Wave speeds calculated for test 32 were close to those of the reliability tests except for the wave speed calculated from soil-interface gage P6, which was significantly lower than for the reliability tests. Wave speeds calculated from accelerometers A1 through A4 and pressure gages P1 through P5 were normally distributed, with some wave speeds being slightly above the mean and some being slightly below the mean for the reliability tests.

SECTION 7

SUMMARY AND CONCLUSIONS

7.1 SUMMARY

Thirty-one 1:30 scale reinforced soil wall models were subjected to blast loading in a geotechnical centrifuge. Average wall panel residual displacements, peak pressures, peak accelerations and wave speeds were measured. The purpose of this study was twofold: (i) to investigate the ability of the geotechnical centrifuge to reproduce similar results for nominally identical models, and (ii) to conduct a parametric study to determine which individual model wall parameters significantly affects the overall wall response to blast loading. The test program consisted of the following:

- Five preliminary tests to develop the model construction technique and to evaluate the performance of the blast simulator.

- Ten reliability tests (i.e., ten models with nominally identical parameters) to determine statistical limits of random variation of test results.

- Sixteen production tests to investigate how individual test parameters influence overall wall response. The following test parameters were investigated: (i) friction area of reinforcement, (ii) facing panel geometry, (iii) soil density and type, (iv) overall model geometry, and (v) reinforcement strength.

Measured pressures and accelerations from the preliminary tests were compared to those predicted from the computer program CONWEP. The collected data generally fell within 50 percent of the predictions, which is considered very good for small or full-scale modeling. Qualitative comparisons of wall residual displacement showed good agreement with residual displacements obtained in the Phase I testing program (Reference 1) and in full scale testing (Reference 5).

The results of the reliability test series indicated that facing panel residual displacements and backfill wave velocities measured at the facing panels were extremely reproducible in the centrifuge. The results of a statistical analysis showed that the coefficient of variation (mean divided by standard deviation) ranged from 6 to 16 percent for average facing panel residual displacements and 3 to 5 percent for measured wave velocities. The small variation in wave velocities is an indication of uniform soil density throughout the model. Reproducibility of instrumentation data was slightly less successful. The coefficient of variation for measured pressures ranged from 14 to 40 percent. The soil-interface pressure gage mounted to the side of the sample preparation bucket was most reliable (14 percent), followed by the free-field pressure gages (16 to 20 percent) and the soil-interface pressure gages mounted to the wall facing panels (26 to 40 percent). The coefficient of variation for measured accelerations ranged from 17 to 36 percent. The free-field accelerometer was most reliable (17 percent), followed by the soil-interface accelerometers mounted to the wall facing panels (23 - 36 percent). A large variation in pressures and accelerations was anticipated due to the sensitivity of the gages. Similar results were obtained in the Phase I (Reference 1) study and in full-scale testing (Reference 5).

The results from the reliability test series served as a baseline to which the results of the production test series were compared. For example, if the wall residual displacements due to variation of a given model parameter fell within the mean plus or minus two standard deviations of the reliability test residual displacements, there is a 95 percent degree of confidence that the results were due to random variation and not to the varied parameter. Wall panel residual displacement was the main focus of the production test series, since success or failure of a reinforced soil wall is determined by its geometry after the explosive event. Pressures, accelerations and wave speeds were also collected to investigate their correlations with wall response.

In four production tests, the friction area of reinforcement was varied from -50 percent to +150 percent of the baseline total reinforcement frictional area. This was achieved by lengthening and shortening the reinforcement strips, and by increasing and reducing the number of reinforcing layers in the model. For two of the four tests, panel residual displacements fell within the range of random variation of results. The third test, in which very long reinforcing was used, showed significant reduction in panel residual displacement for the top row of panels only, the residual displacements of the middle and bottom rows of panels falling within the range of random variation. The fourth test, which had only one reinforcing strip per panel, showed a significant increase in residual displacement, but the wall did not breach. This was an interesting occurrence since the amount of reinforcing used in this test was less than the minimum required for static stability. Therefore, provided the minimum design standards for static stability are met, the area of total reinforcement did not appear to significantly affect the wall response. No significant change in pressure, acceleration or wave speed was noted.

Three tests were conducted in which facing panel geometry was altered. Panel dimensions in these tests were: 4-inch long by 2-inch high, 4-inch long by 1-inch high, and 2-inch long by 1-inch high. The reliability test series was conducted with 2-inch long by 2-inch high panels. With the exception of the 2-inch by 1-inch panels, panel geometry did not significantly affect wall response. Panel residual displacements were increased significantly at the center of the middle and bottom rows of panels when 2-inch long by 1-inch high panels were used. No significant change in pressure, acceleration or wave speed was noted.

Two tests were conducted in which soil density was varied. One test had a soil density of 89 pcf, and the other 95 pcf. The soil density for the reliability test series was 103 pcf. The results of these tests indicated that soil density is the most significant parameter affecting wall response. Panel residual displacements fell almost exactly on the upper 95 percent confidence line for the 95 pcf soil, and fell entirely above this range for the 89 pcf soil. Peak pressures and accelerations were considerably lower than the baseline mean, but still higher than the mean minus two standard deviations of the reliability test series. For the 89 pcf test wave speeds were significantly slower than the those measured in the baseline tests. For the 95 pcf soil, wave speeds were not significantly changed.

One test was conducted in which soil type was altered. Sky X sand, which was the soil being used in the full-scale testing at Tyndall AFB, was used instead of Tyndall Beach sand. The wall residual displacements for this test were nearly identical to those measured in the baseline tests; however, measured pressures and accelerations were lower than those measured in the baseline tests by almost an order of magnitude. Measured wave speeds were also significantly lower than those measured in the baseline tests. The grain size distribution curves of these two soils are almost identical, but the Sky X sand contains approximately 2 percent fines where as the Tyndall Beach sand contains no fines. Also, the coloring of the Sky X sand suggests the presence of something other than silica. These test results show that small differences in soil properties can significantly affect pressure, acceleration, and wave speed, but not necessarily wall residual displacement. Further testing with SKY X sand and other soils is necessary to confirm this.

Three tests were conducted in which the overall model geometry was altered. One test included a roof structure above the wall, one test contained a slope in the backfill behind the wall to aid in dissipating explosive energy, and one test was conducted with a 6.5 inch standoff. Both the roof structure and the backfill slope significantly reduced residual displacements in the top row of panels only. No significant change was noted in pressure, acceleration, or wave speed. As expected, reducing the standoff distance from 8.5 inches to 6.5 inches (a difference of 5 feet in prototype terms) significantly increased panel residual displacement. The wall was breached at five panel locations. Pressures on the wall panels were significantly increased, as were backfill wave speeds measured to the panel gages. Accelerations of the panels were much higher than the baseline mean but due to the large variation in peak acceleration measured in the reliability test series, this was not considered significant. Pressures, accelerations, and wave speeds measured by the free field gages, which had the same standoff distances as in the reliability tests, showed no significant change.

Two tests were conducted in which reinforcement tensile strength was qualitatively altered. One test contained reinforcing strips which had holes punched in them, reducing the material tensile strength. One test was conducted with continuous sheets of reinforcing material as opposed to individual strips, introducing continuous tensile stress distribution in the reinforcement and shear strength between adjacent facing panels. The results of both tests showed significant residual displacement at the sides of the wall, while residual displacements at the center of the wall were within the range of random variation. Most pressures and accelerations were within the range of random variation, with a few gages yielding data that were slightly higher or lower than the mean plus or minus two standard deviations of the reliability test series. With the exception of data collected at one gage, the calculated wave speeds for both tests showed no significant change.

7.2 CONCLUSIONS

Based on the results of the Phase 2 test program, the following conclusions were drawn and are briefly summarized below.

- Wall residual displacements and wave velocities are very reproducible in the geotechnical centrifuge, while pressures and accelerations are not as reproducible.

- Soil density was the most significant factor influencing wall response. As soil density increased, wall displacements decreased.

- Two soil types with only slight property differences can produce different results. Although wall residual displacements were similar, peak pressures and accelerations measured in the Sky X sand were significantly lower than those measured in the Tyndall Beach sand at similar densities.

- Reducing reinforcement tensile strength did significantly increase wall residual displacement at the sides of the wall, but not as much as would probably be expected for a large reduction in reinforcement strength.

- Using one continuous sheet of soil reinforcement, as opposed to several strips, significantly increases wall residual displacements at the sides of the walls, but did not reduced them at the wall center.

- Reducing standoff distance by 2 inches (5 feet for prototype) significantly increases wall residual displacements, peak pressures, peak accelerations, and wave speeds.

- Facing panel geometry does not significantly affect wall response.

- The amount of reinforcing used in the wall does not significantly affect wall response (provided the minimum amount of soil reinforcement was used to maintain static stability).

- The roof structure significantly reduces panel residual displacements in the top row of panels only.

- The berm significantly reduces wall residual displacements in the top row of panels only. Smaller panel residual displacements might have been experienced in the middle and bottom rows of panels if the backfill slope had been extended below the top row of panels.

SECTION 8

RECOMMENDATIONS

Results of the geotechnical centrifuge modeling study have provided valuable insight into the behavior of reinforced soil structures exposed to blast loadings. Specific parameters can be qualitatively evaluated in comparison to limits of random variation established in the reliability study. The following recommendations are made:

- Further geotechnical centrifuge modeling and SEM analyses should be conducted on the Sky X sand to determine why measured peak pressures, accelerations, and wave speeds are small compared to those measured in the Tyndall Beach sand.

- Reliability-type test series should be conducted for each of several soil densities (covering the range of possible densities for Tyndall Beach sand) to quantify the effects of soil density on wall response for each density. The results should be studied to determine whether a mathematical or empirical relationship exists between soil density and wall response. If such a relationship is identified, similar testing programs should be conducted for various soil types (i.e., sand, silty sand, silt, silty clay, clay) and any corresponding mathematical or empirical relationships between soil density and wall response for each soil type should be identified. Finally, all results should be studied to determine whether relationships affecting wall response exist between soil types. This extensive study may ultimately lead to the development of a general equation or set of equations that could be used to predict wall response given the soil composition and density.

- Geotechnical centrifuge testing of a variety of modern miniature pressure transducers and accelerometers should be conducted to identify whether the large variation in test results is a function of gage quality, or to gage sensitivity to small, unavoidable differences between test models.

- Geotechnical centrifuge tests should be conducted in a larger sample bucket to reduce boundary effects from nearby rigid sample bucket walls.

- Further testing should be conducted to investigate the effects of reinforcement strength on wall response.

SECTION 9

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Appendix A

Photographic Series of Model Construction

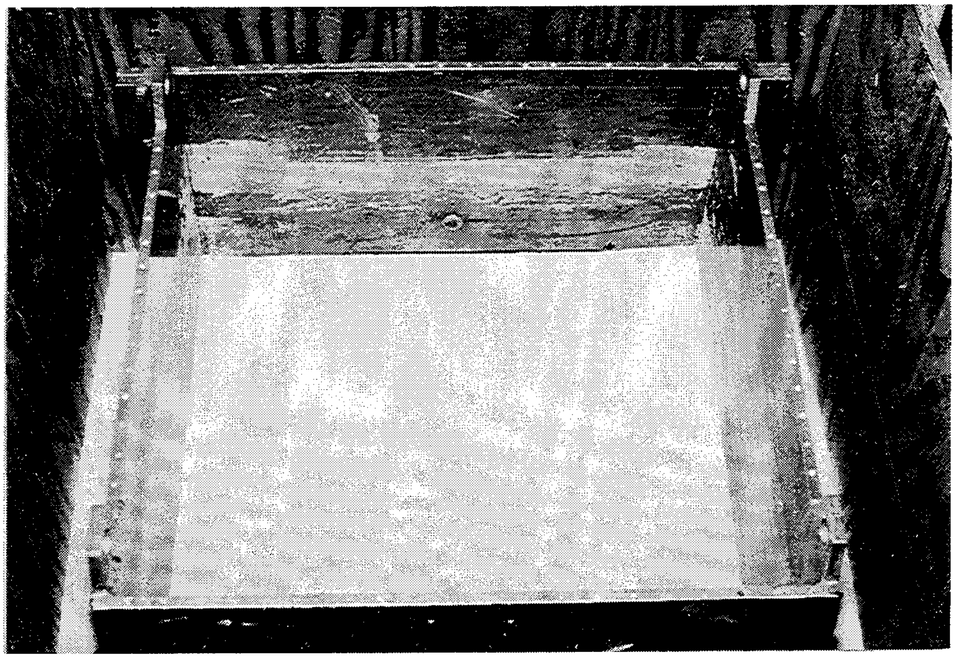

Figure A-1. Sample Bucket with Pluviated Base

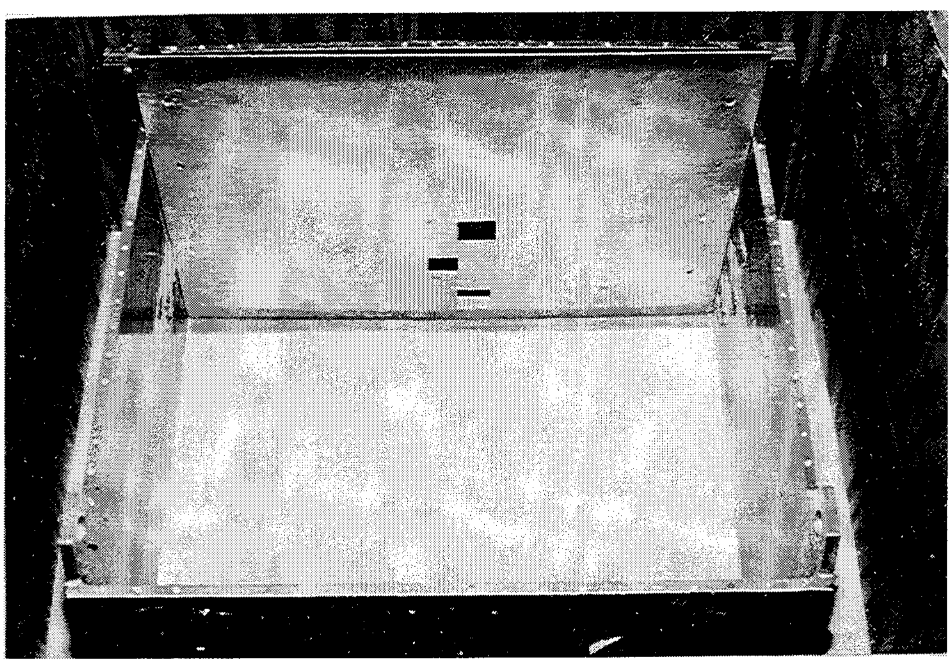

Figure A-2. Aluminum Plate and Levelling Pad

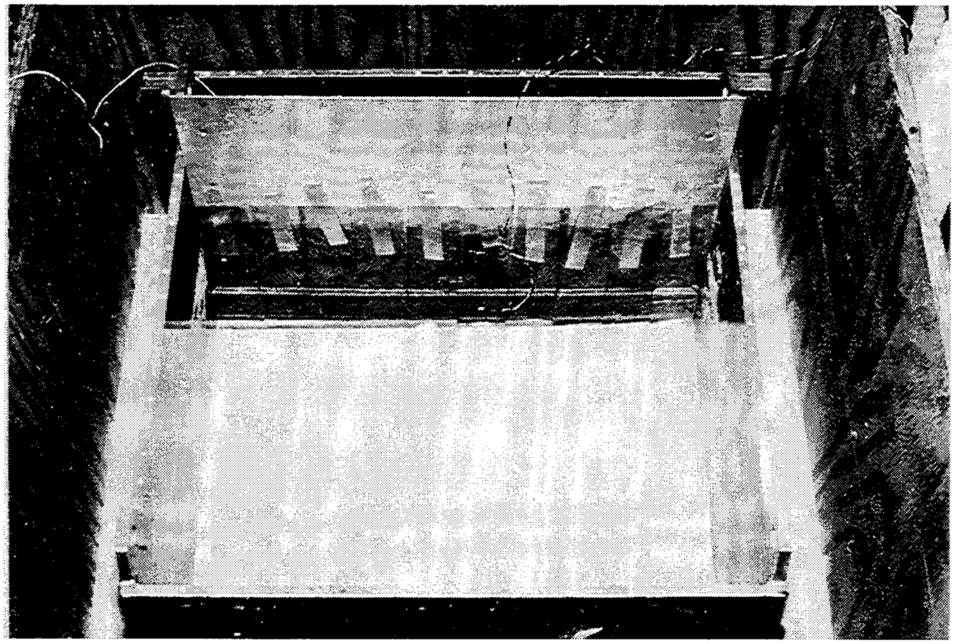

Figure A-3. First Row of Facing Panels

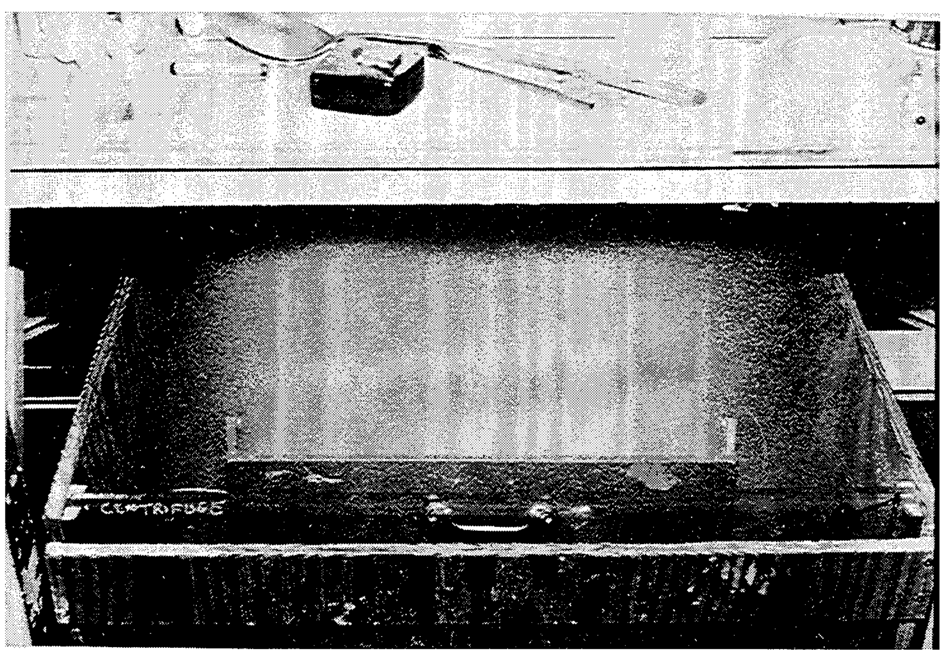

Figure A-4. Soil Pluviation (Typical)

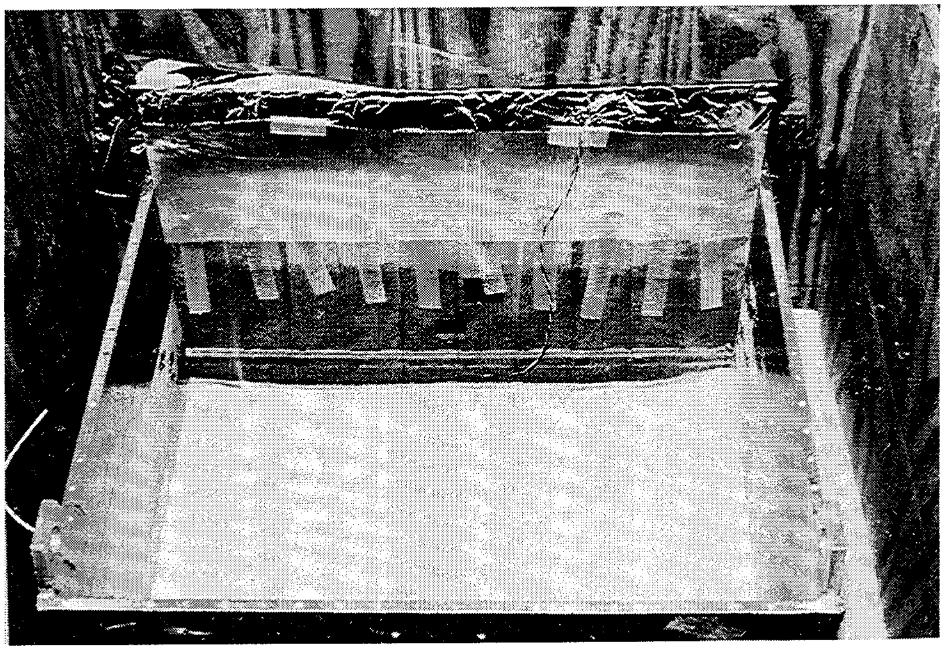

Figure A-5. Soil Pluviated to First Level of Reinforcing

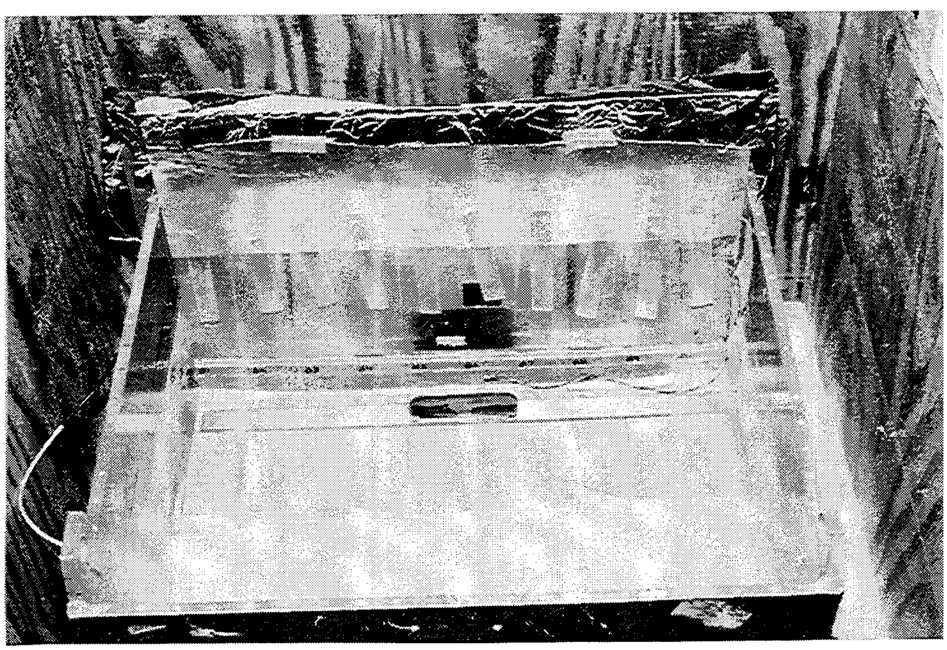

Figure A-6. First Level of Reinforcing Strips

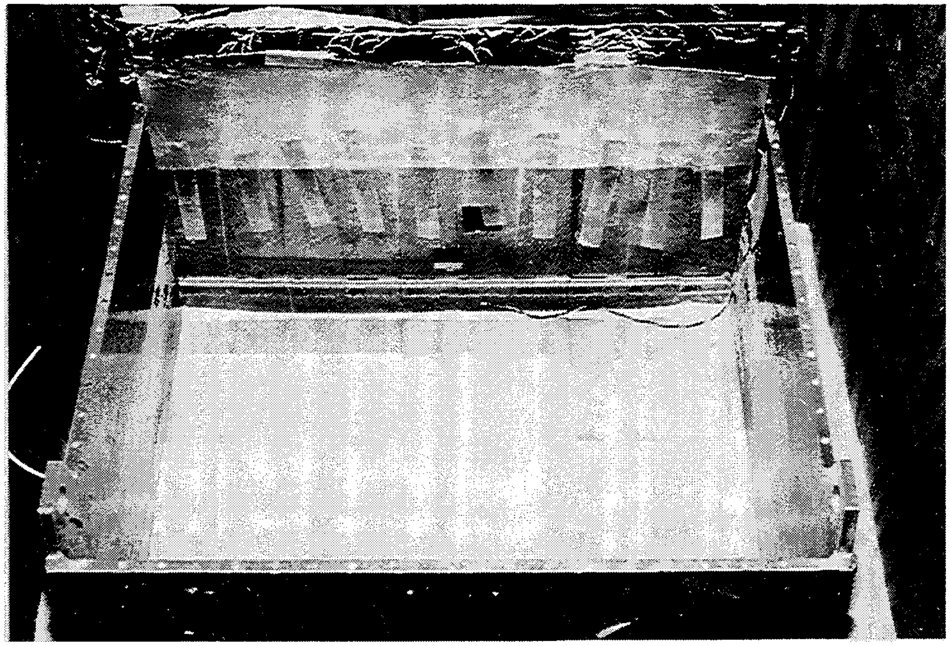

Figure A-7. Soil Pluviated on Reinforcing Strips

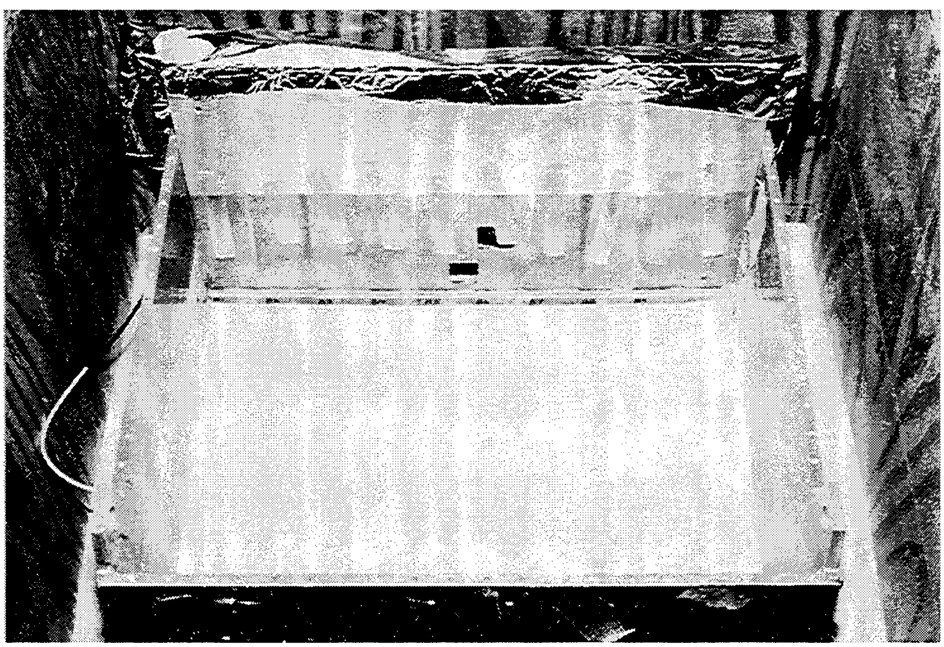

Figure A-8. Soil Pluviated to Second Level of Reinforcing

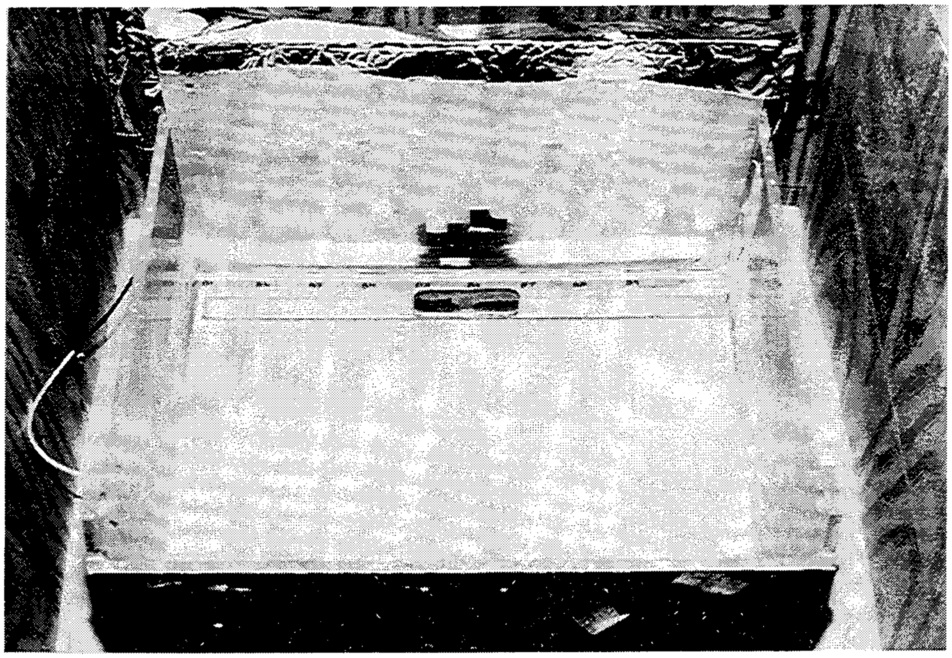

Figure A-9. Second Level of Reinforcing Strips

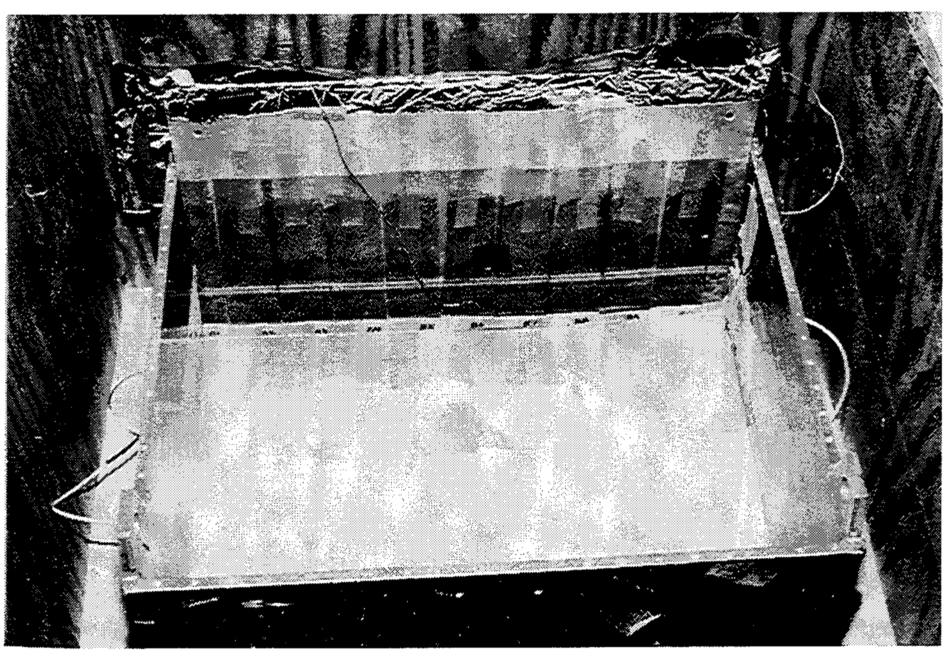

Figure A-10. Soil Pluviated on Reinforcing Strips

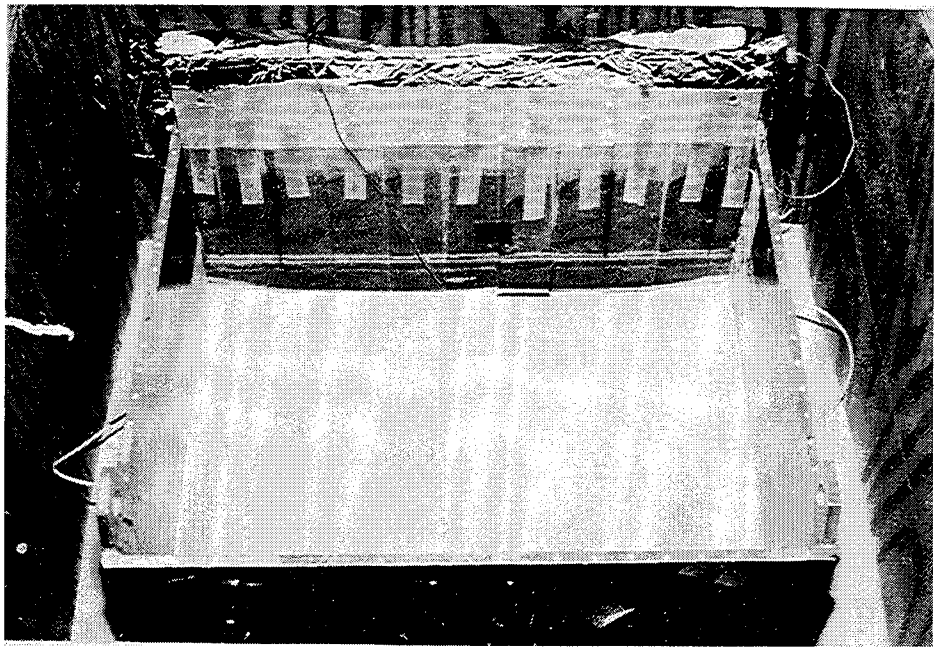

Figure A-11. Second Row of Facing Panels

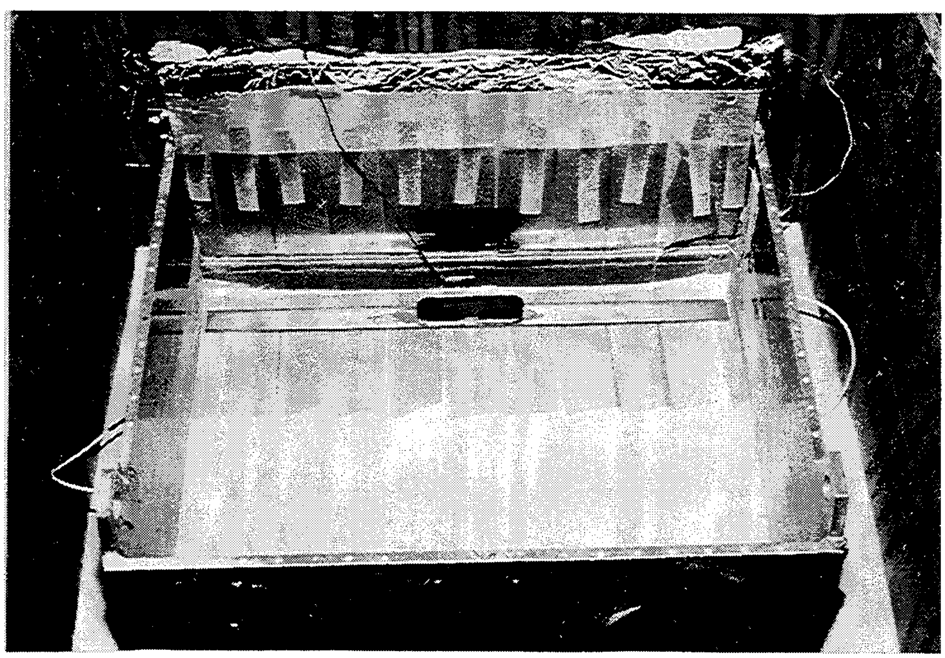

Figure A-12. Third Level of Reinforcing Strips

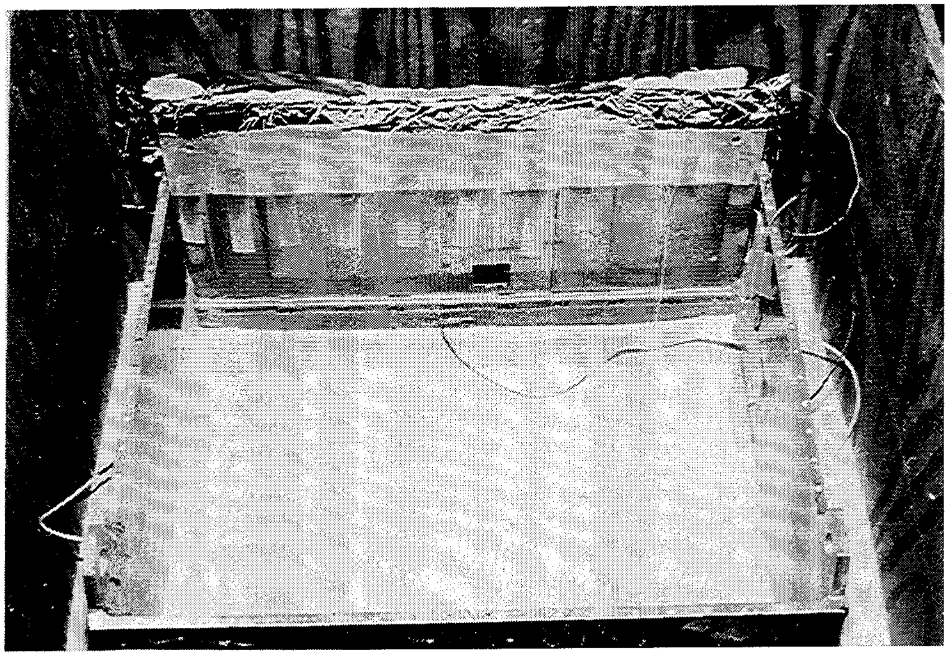

Figure A-13. Soil Pluviated on Reinforcing Strips

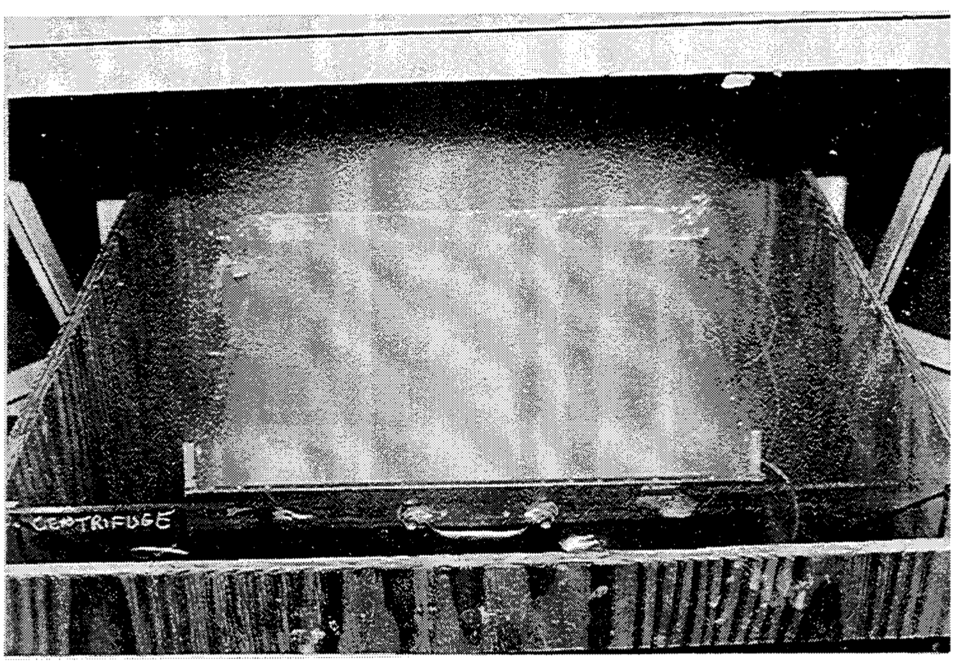

Figure A-14. Soil Pluviated to Three Inches Above the Base of the Wall

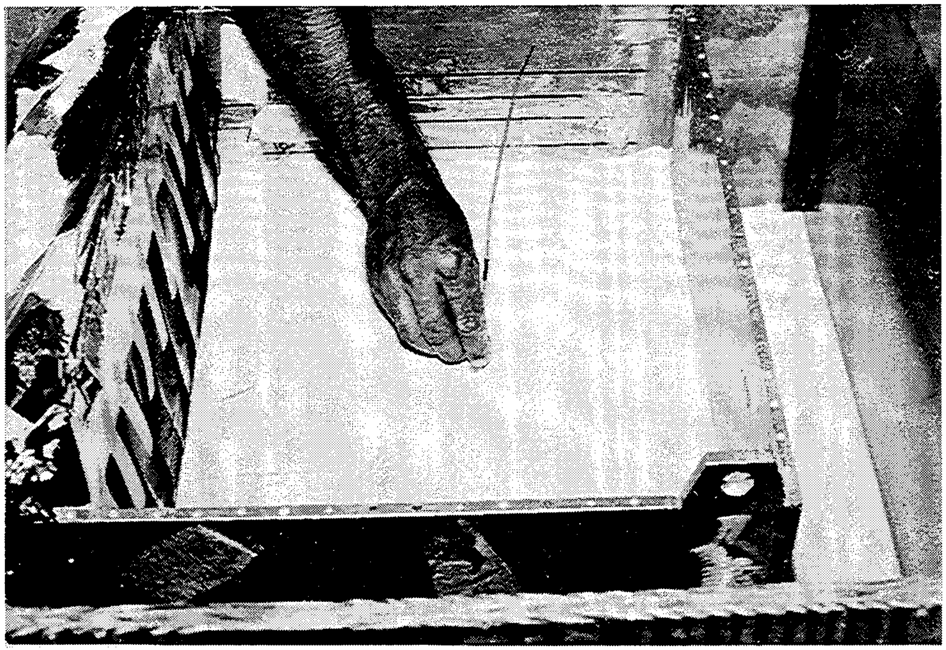

Figure A-15. Placing the Detonator

Figure A-16. Hand Pluviating Soil Above the Detonator

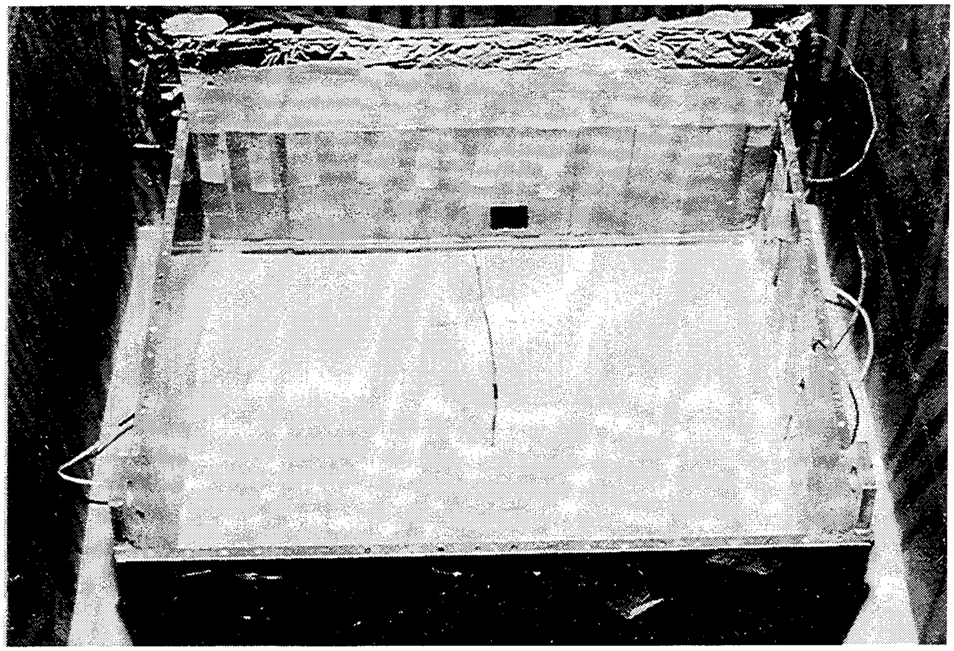

Figure A-17. Soil Pluviated to Fourth Level of Reinforcing

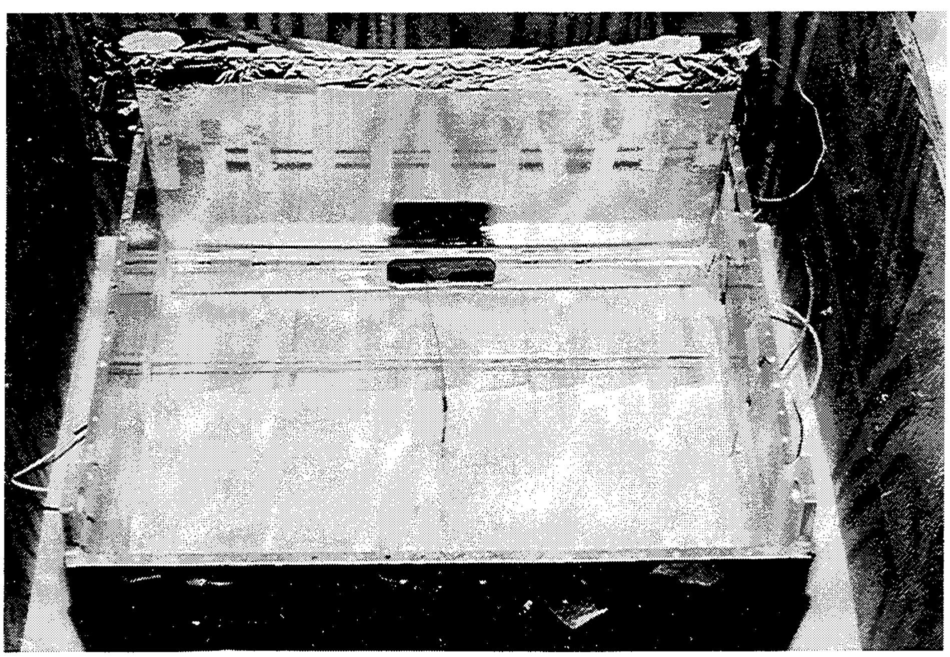

Figure A-18. Fourth Level of Reinforcing

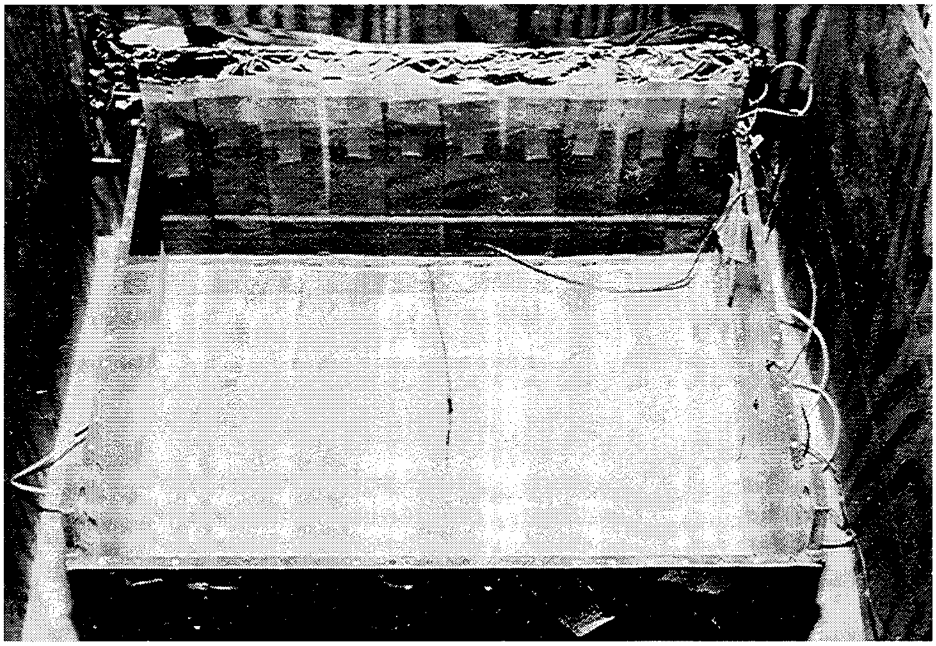

Figure A-19. Third Row of Facing Panels

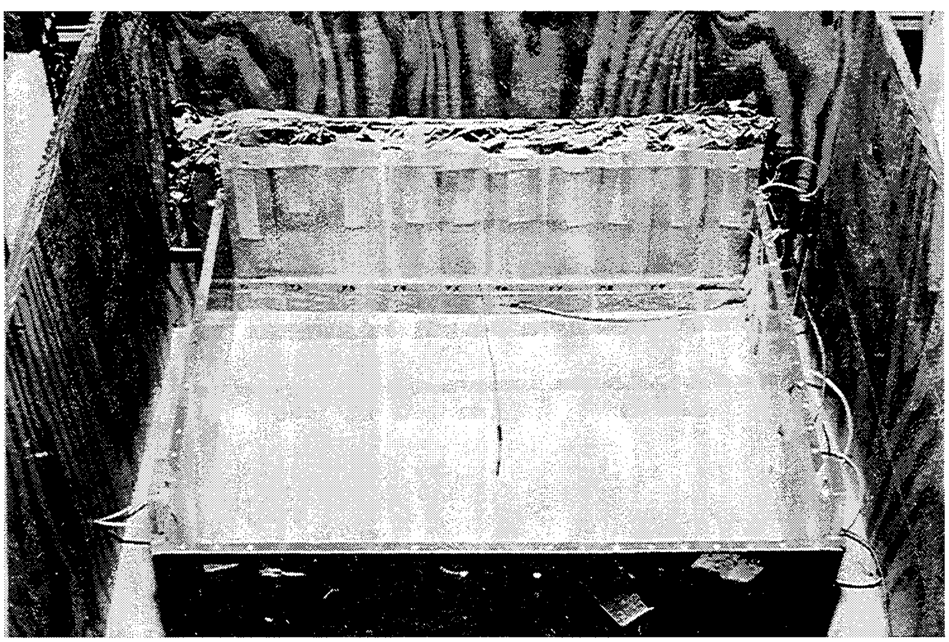

Figure A-20. Fifth Level of Reinforcing

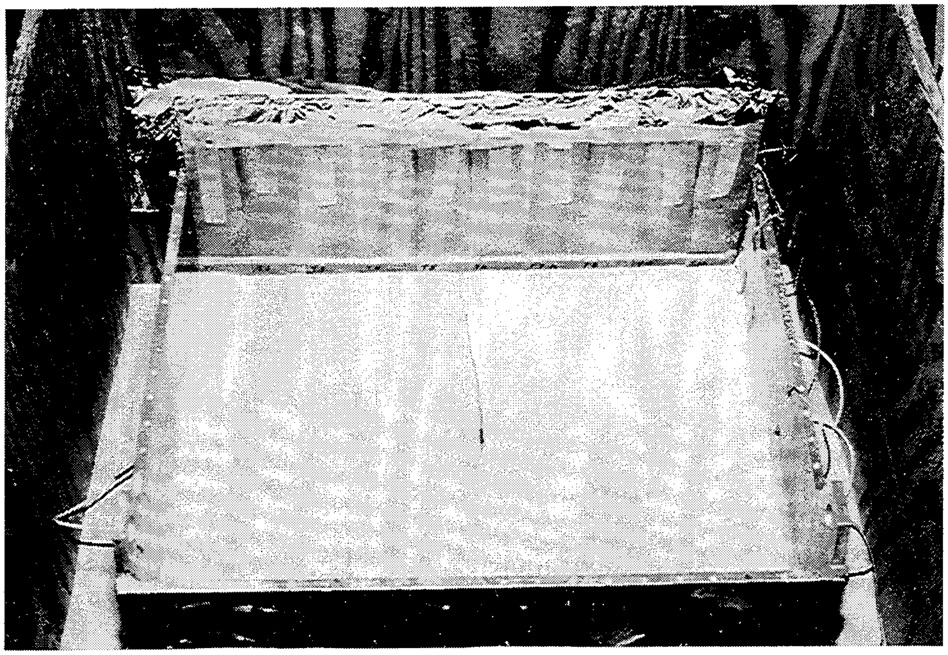

Figure A-21. Soil Pluviated to Sixth Level of Reinforcing

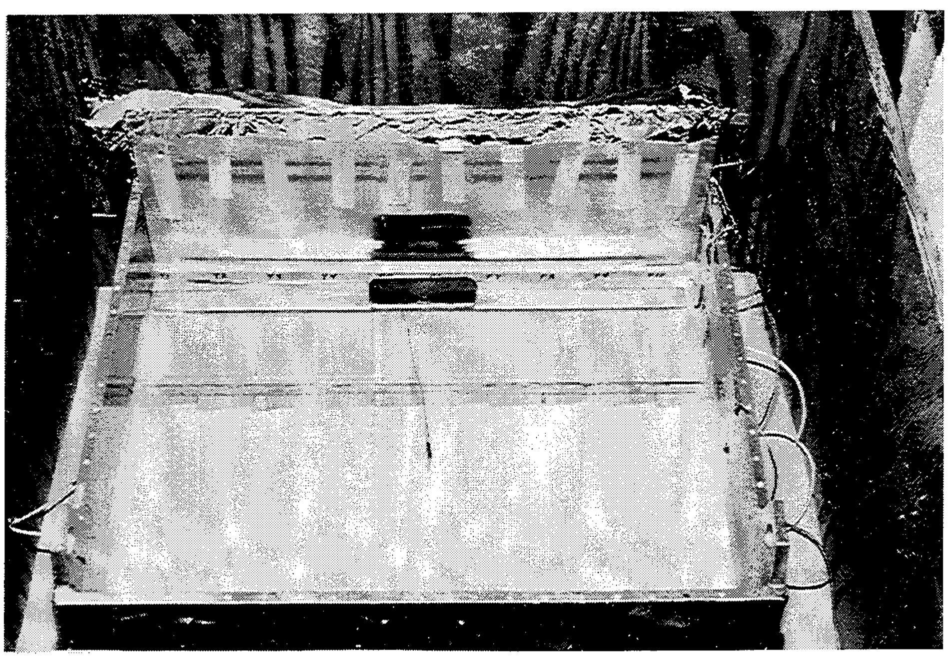

Figure A-22. Sixth Level of Reinforcing

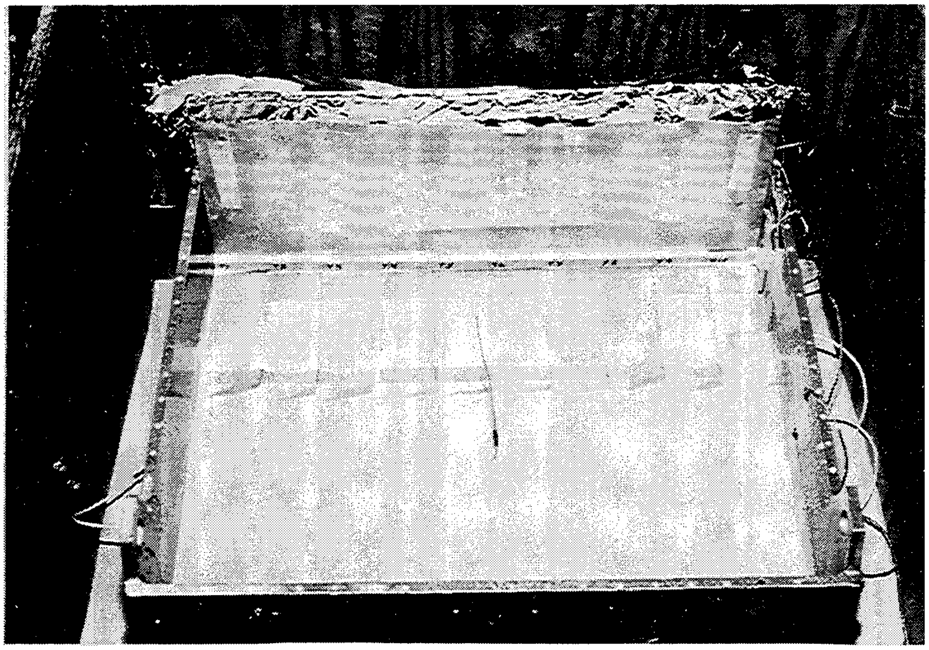

Figure A-23. Soil Pluviated on Reinforcing Strips

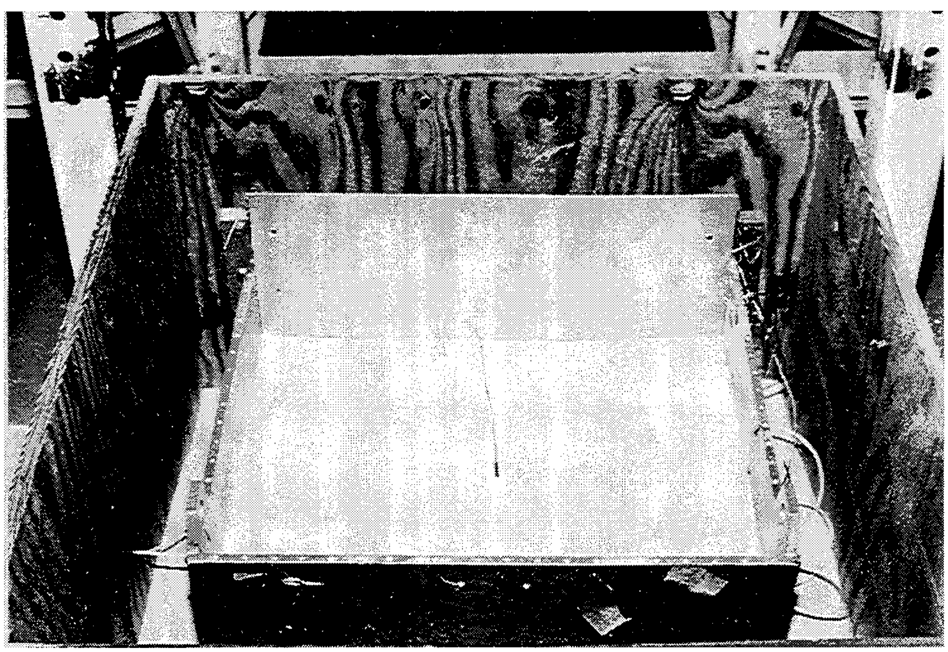

Figure A-24. Soil Pluviated to Top of Wall

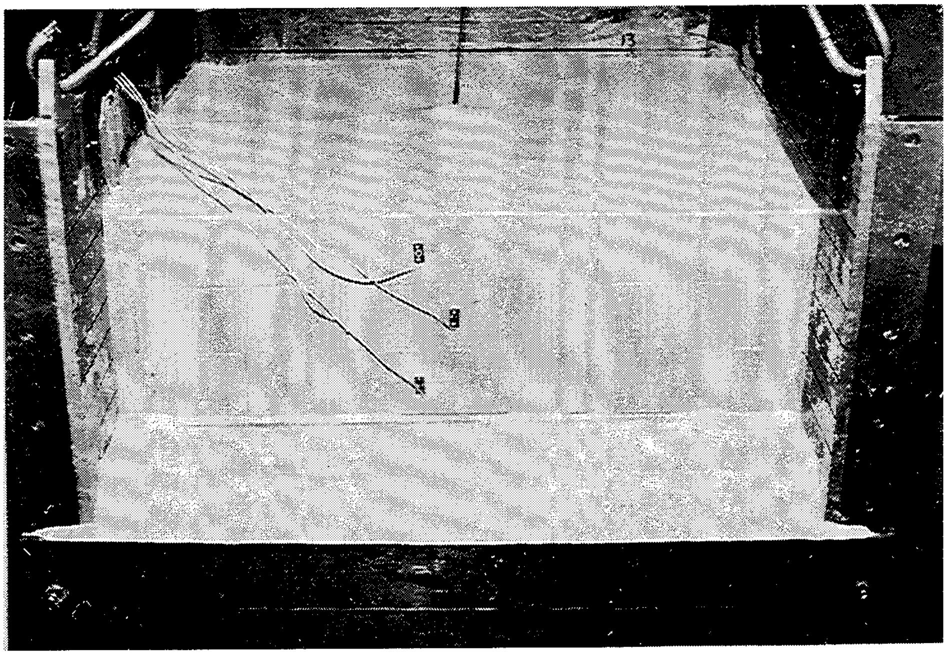

Figure A-25. Front View of Completed Wall

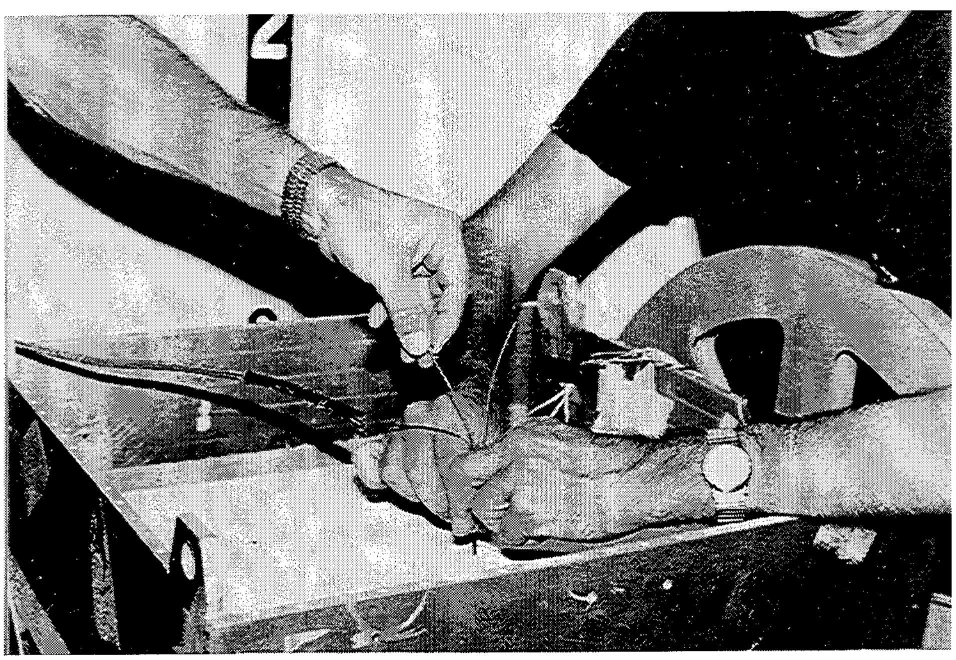

Figure A-26. Wiring the Detonator

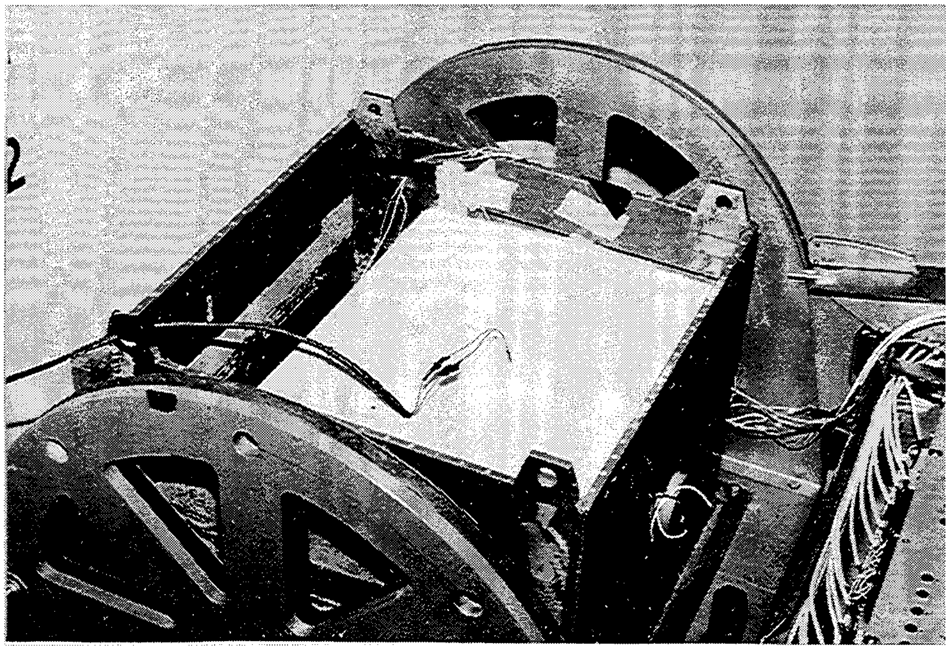

Figure A-27. Completed Sample Ready for Testing

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Appendix B

Post-Shot Photographs of Selected Models

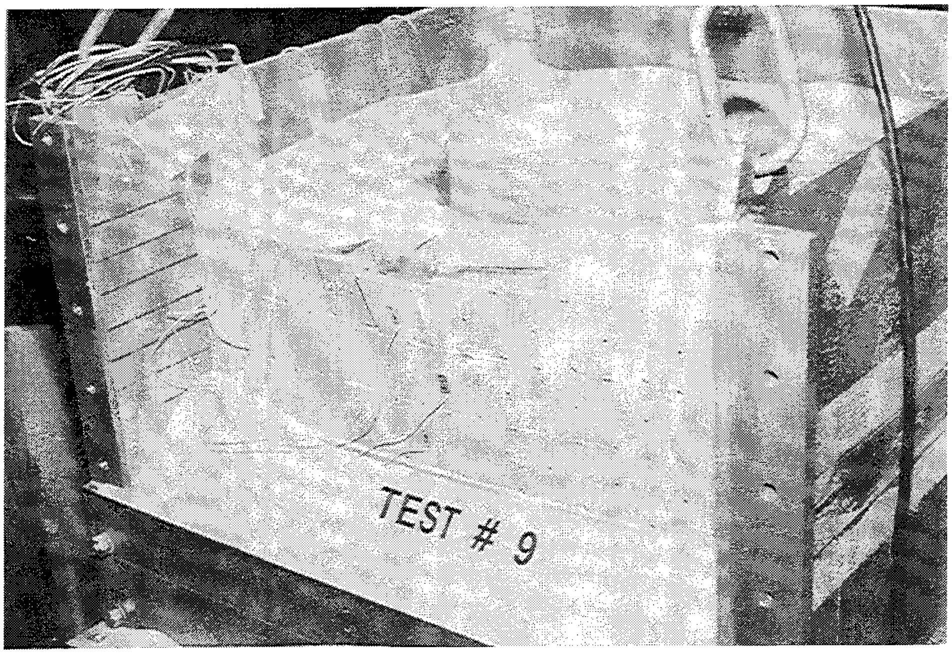


Figure B-1. Post-Shot Wall: Test 9 (Reliability Test)

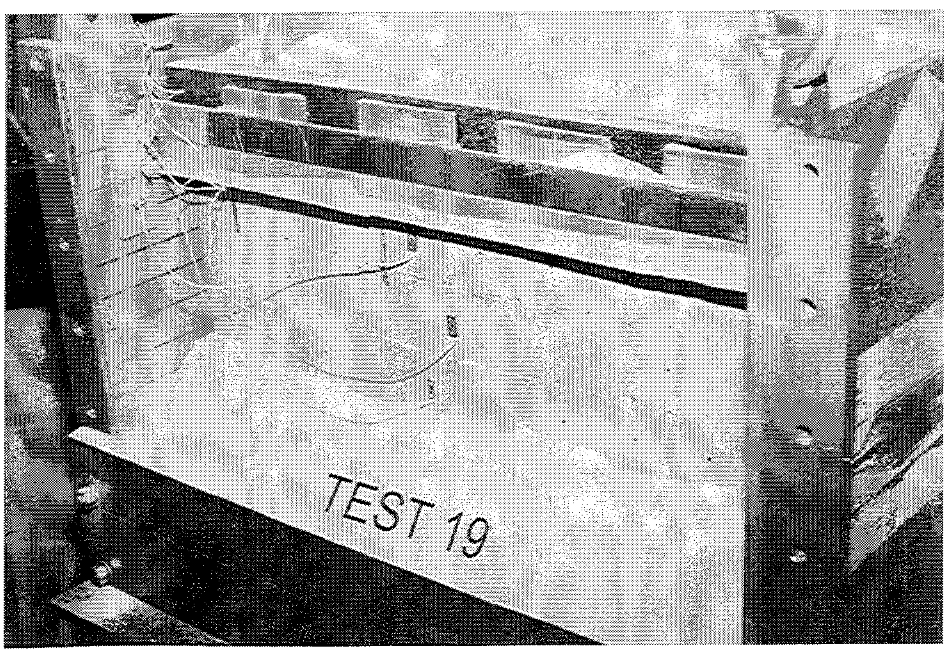


Figure B-2. Post-Shot Wall: Test 19 (With Roof Structure)

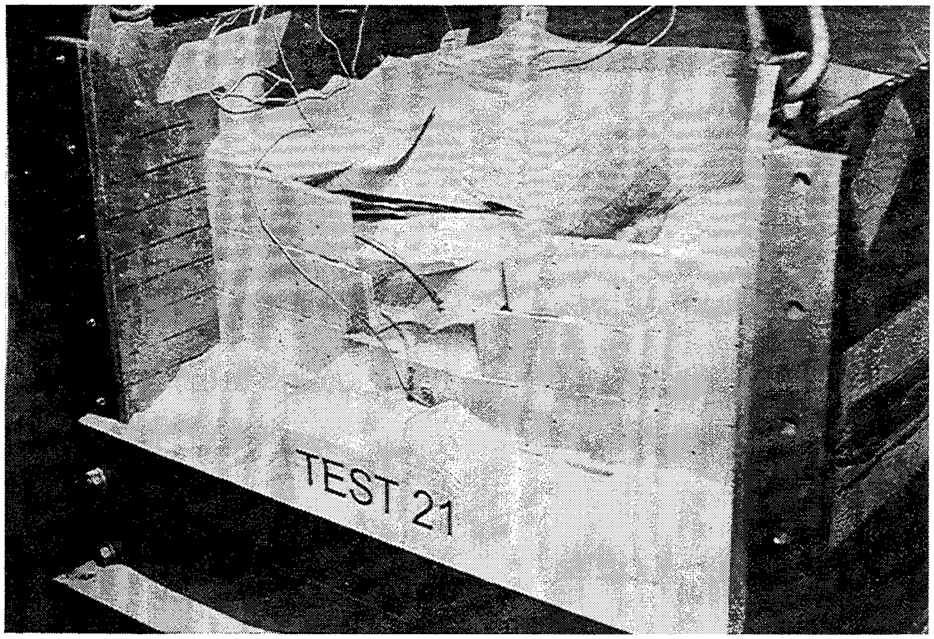


Figure B-3. Post-Shot Wall: Test 21 (6.5-inch Standoff)

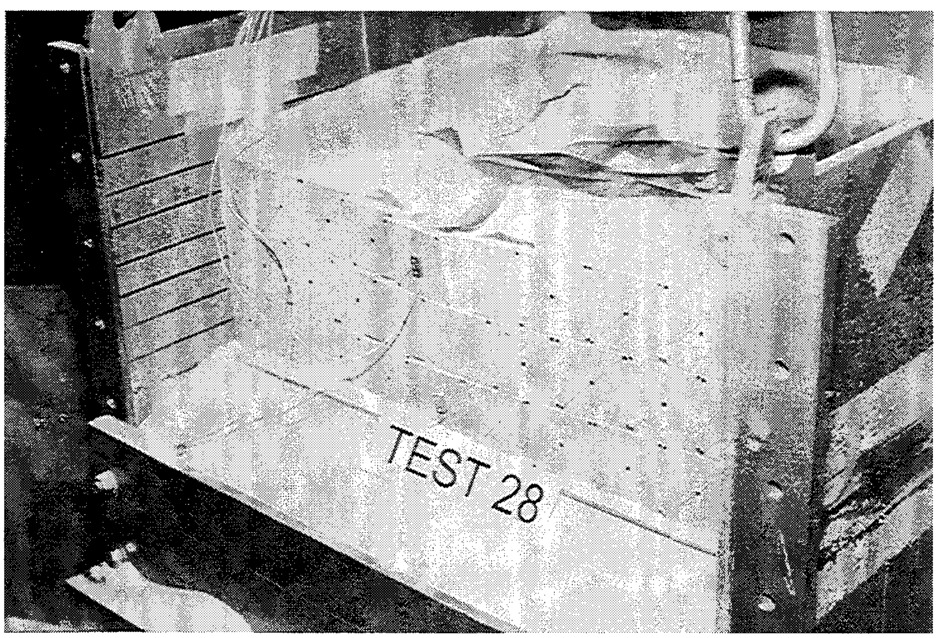


Figure B-4. Post-Shot Wall: Test 28 (15-inch Long Reinforcing Strips)

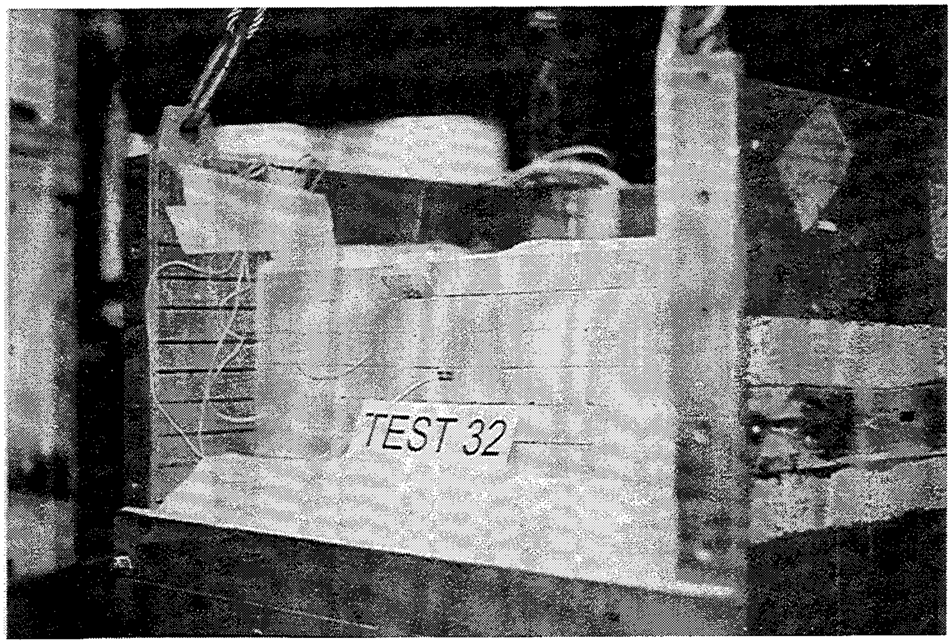
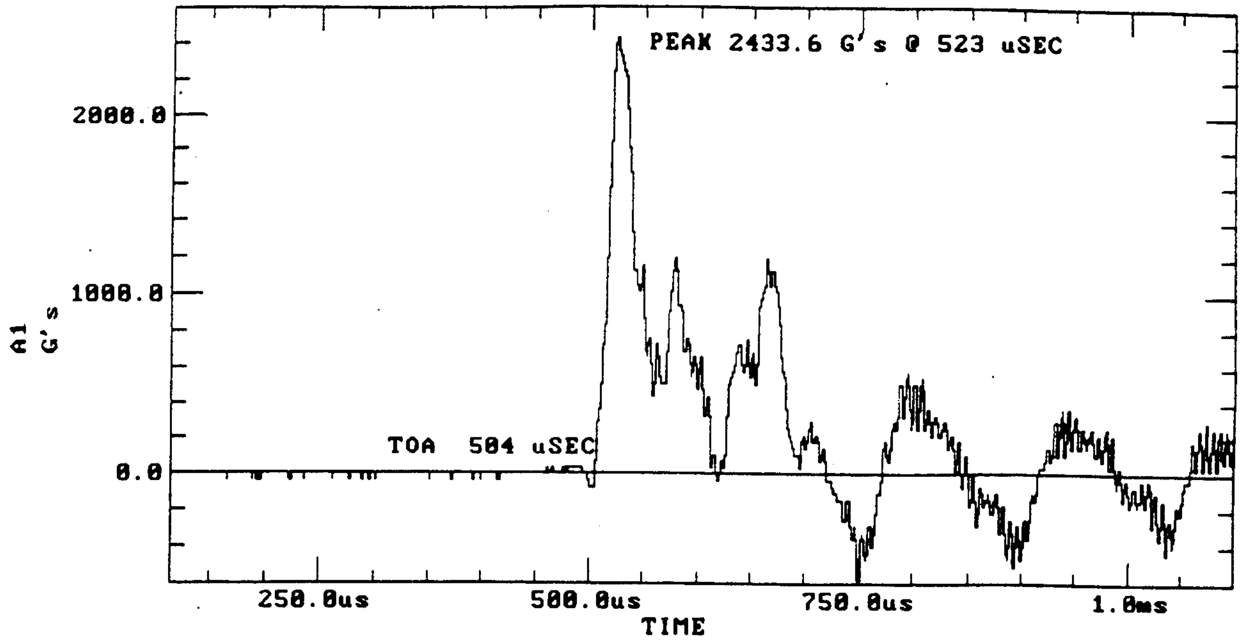
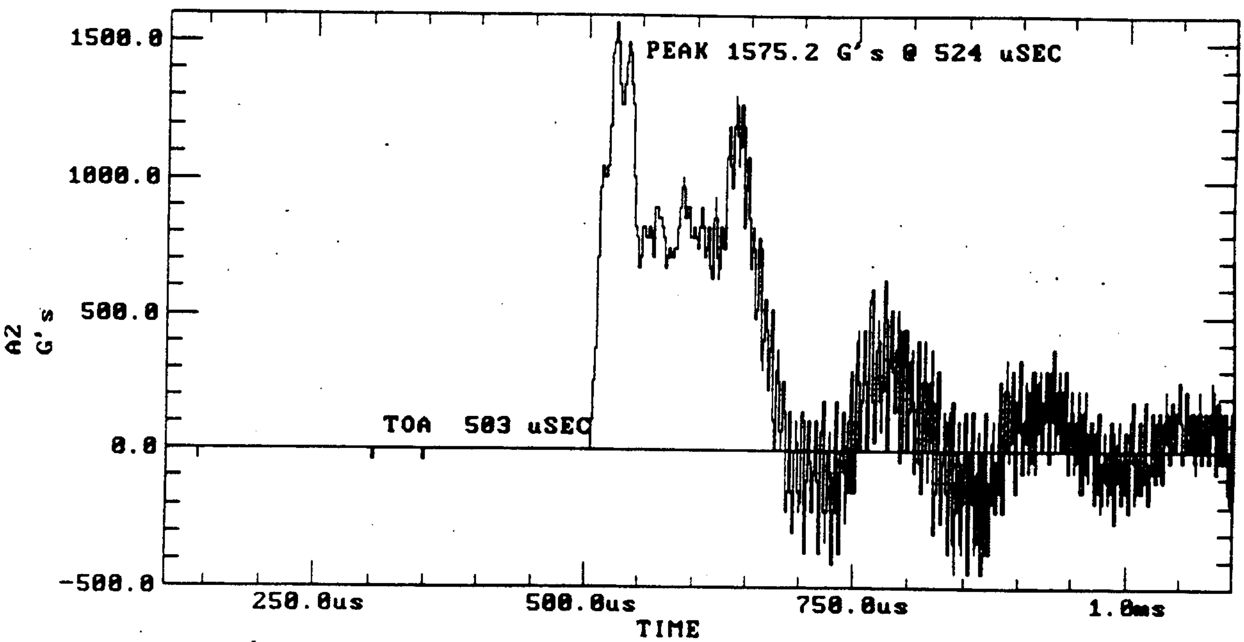


Figure B-5. Post-Shot Wall: Test 31 (2-inch Long by 1-inch High Facing Panels)

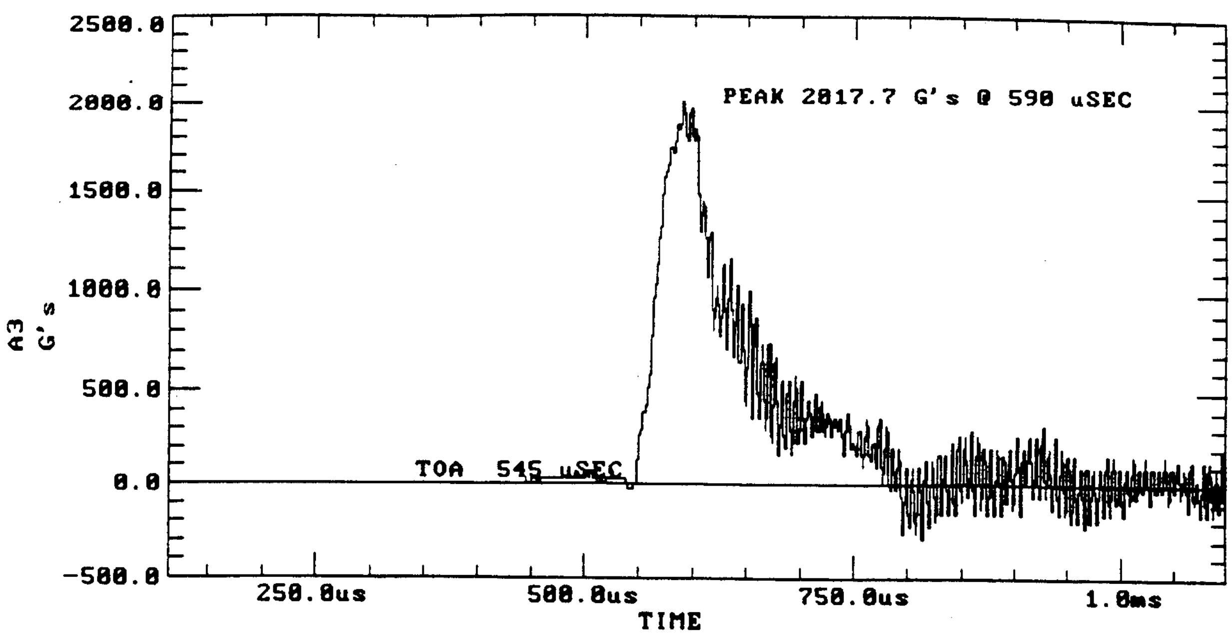
Appendix C

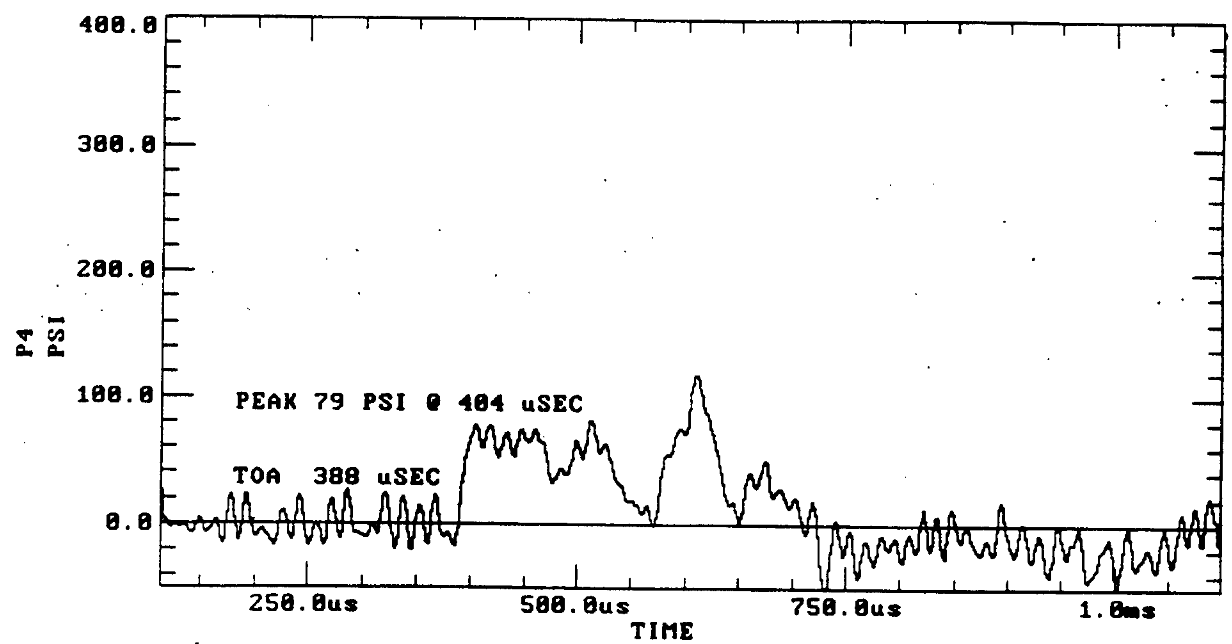
Gage Data



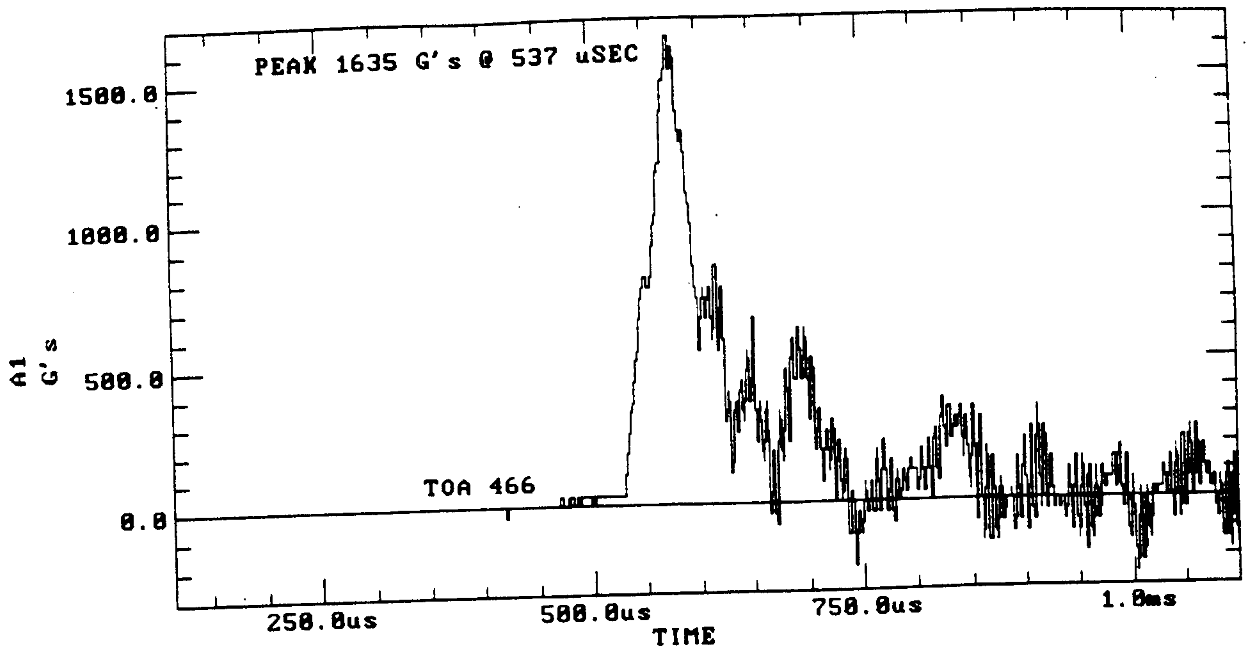


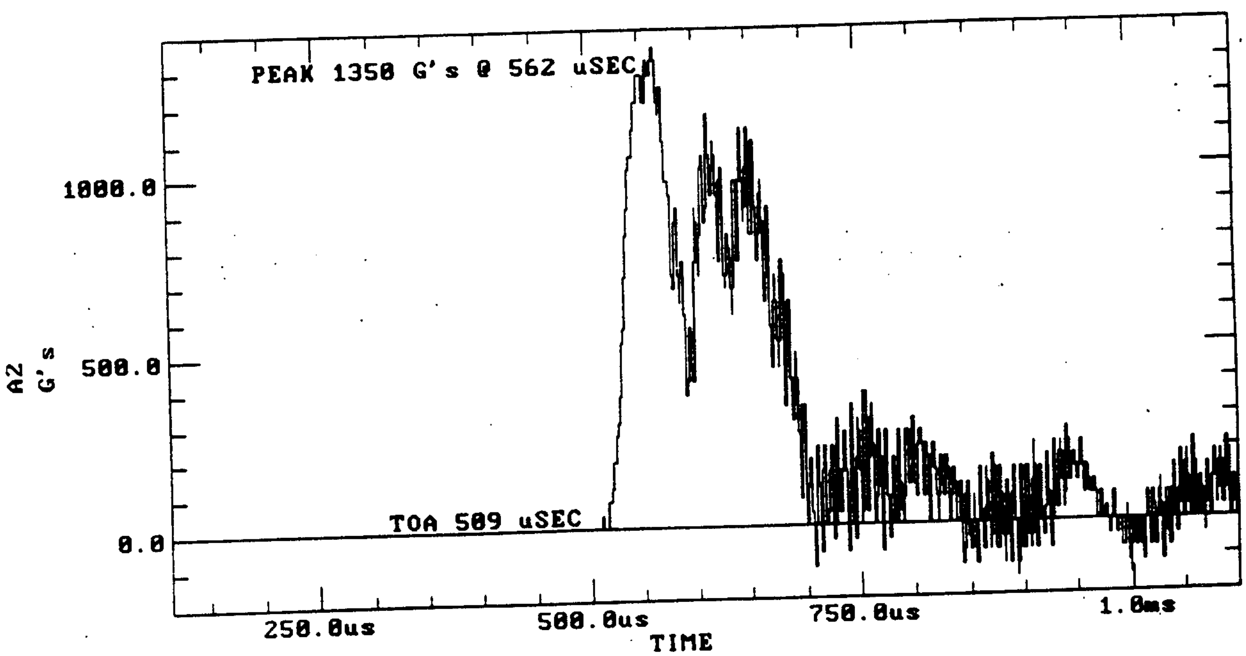
Test 1. Gage Data





Test 1. Gage Data (continued)





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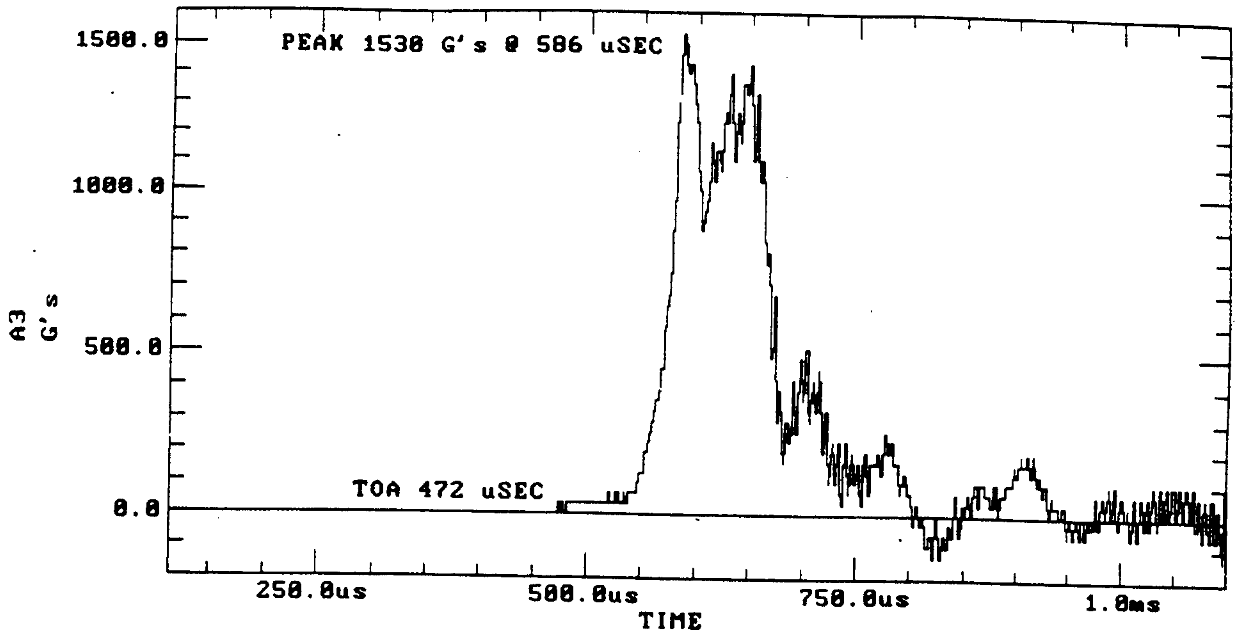

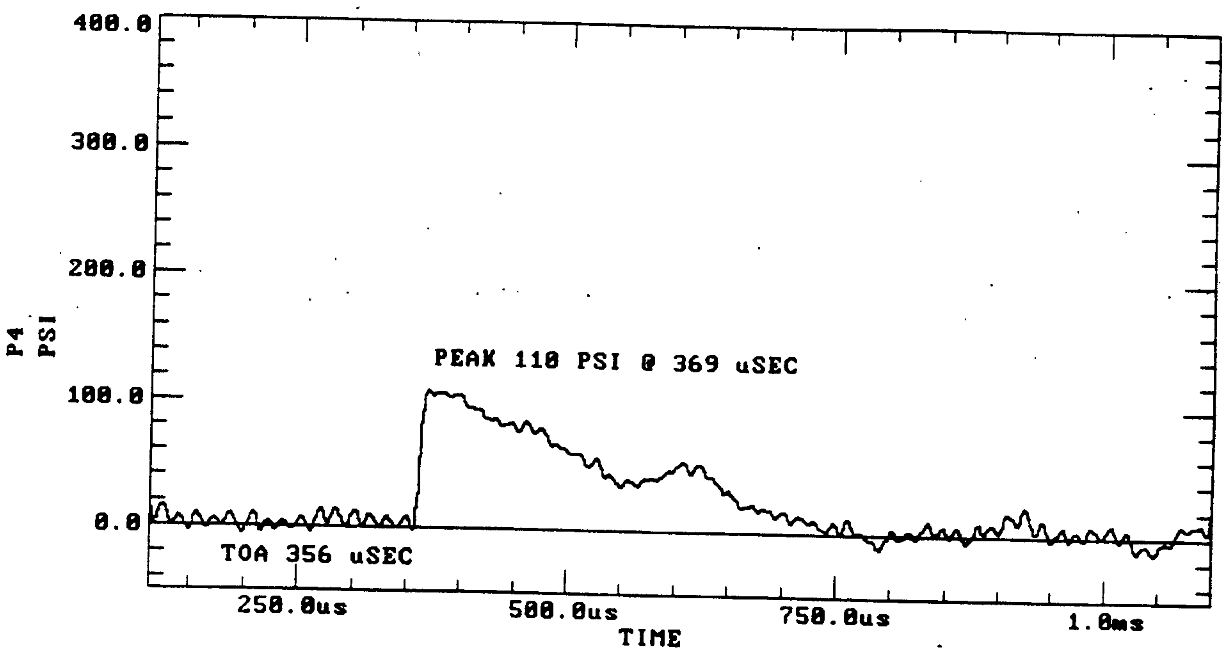

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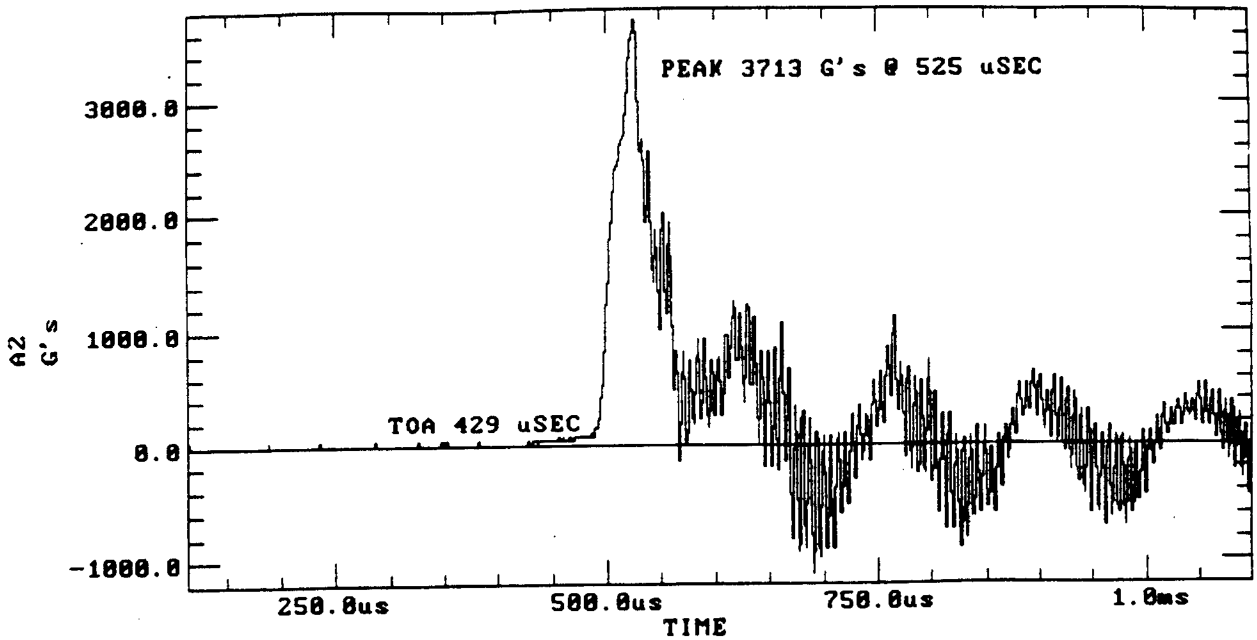

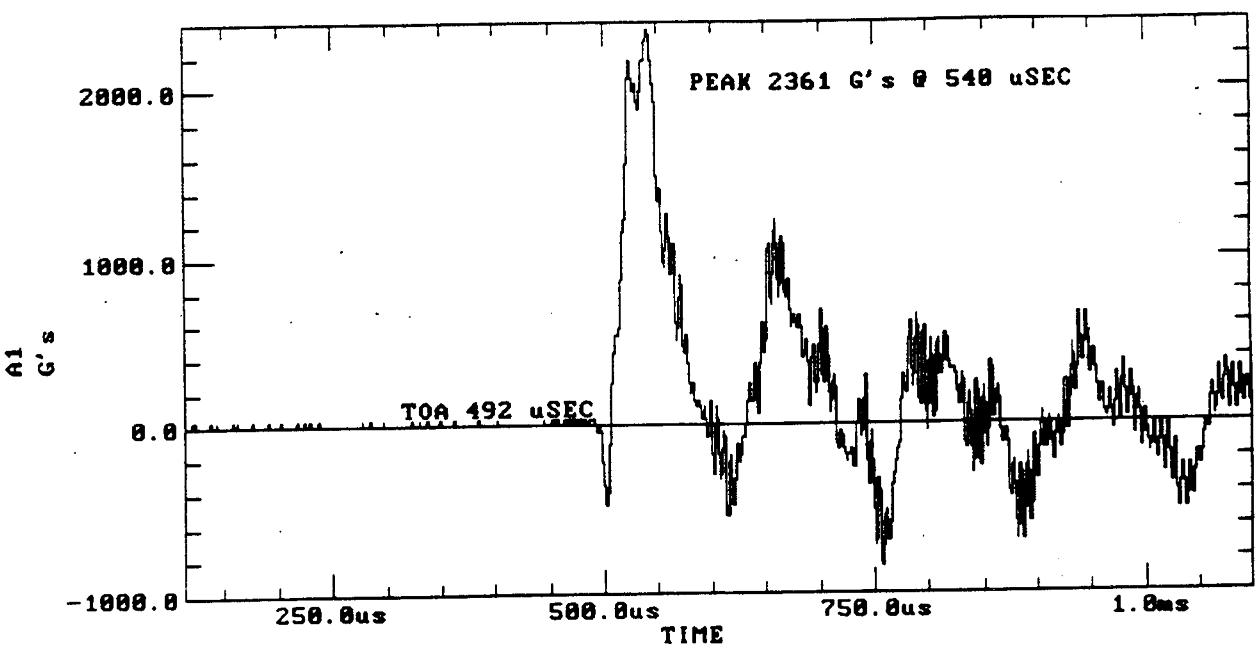

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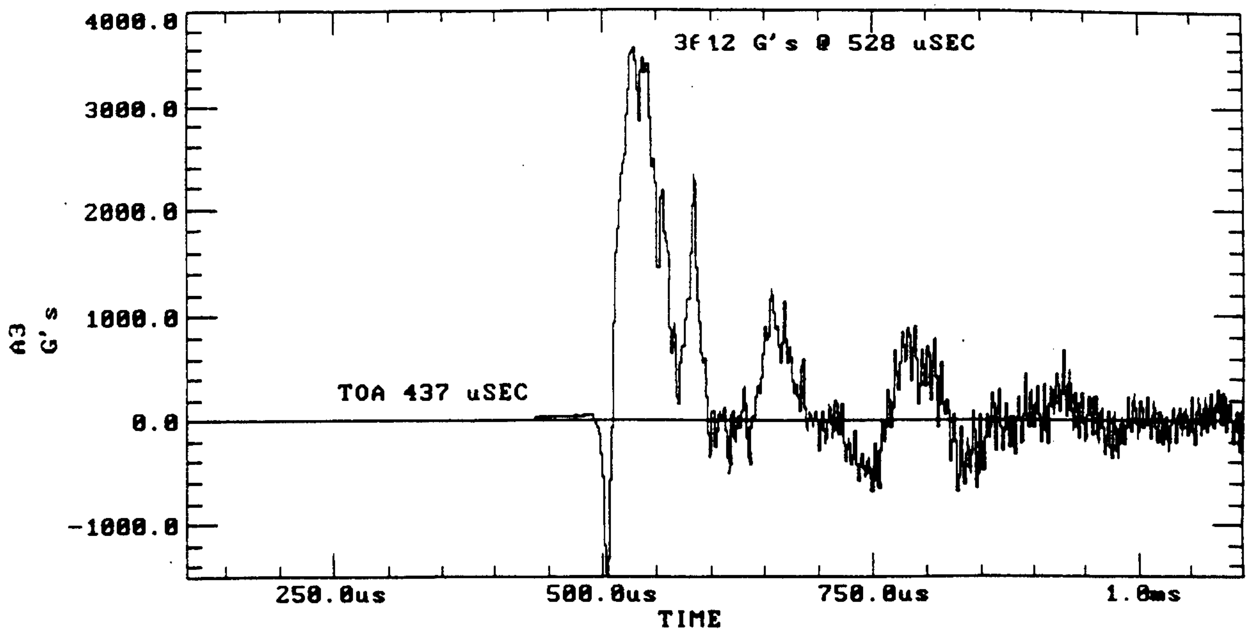

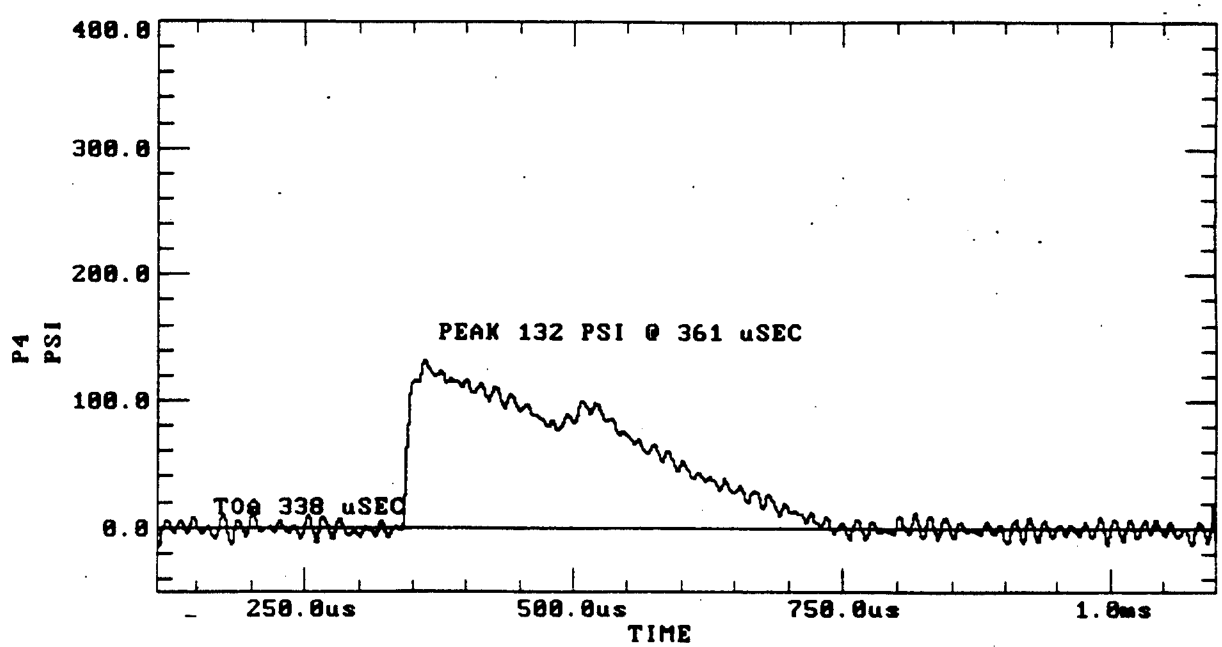

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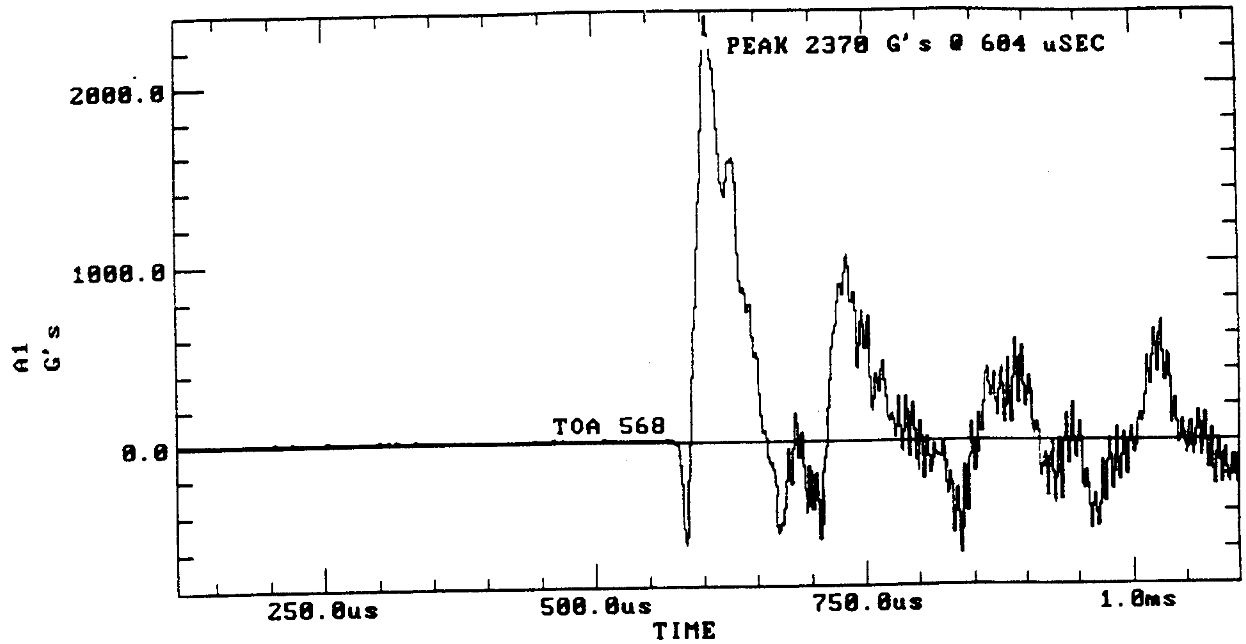

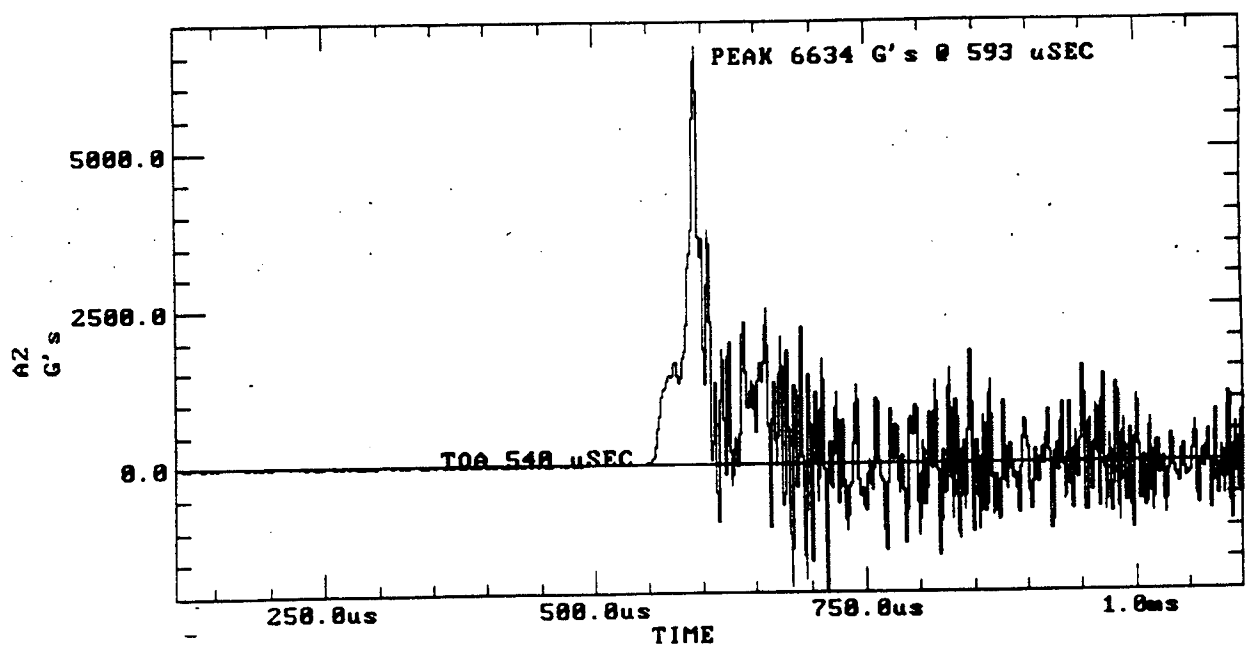

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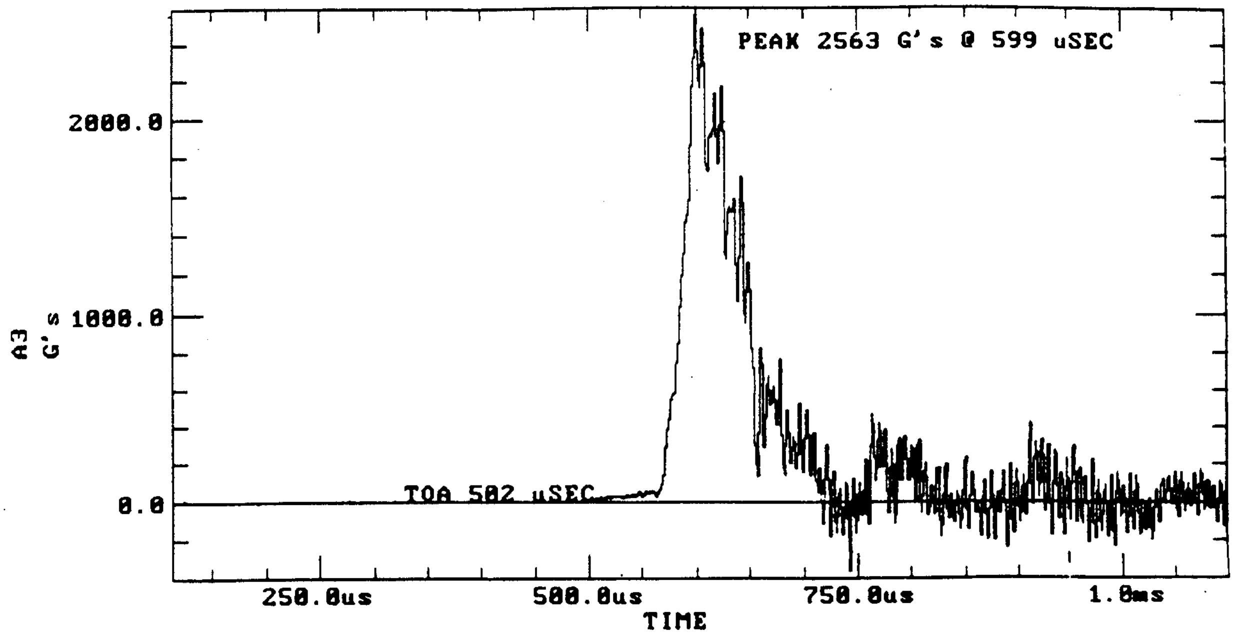

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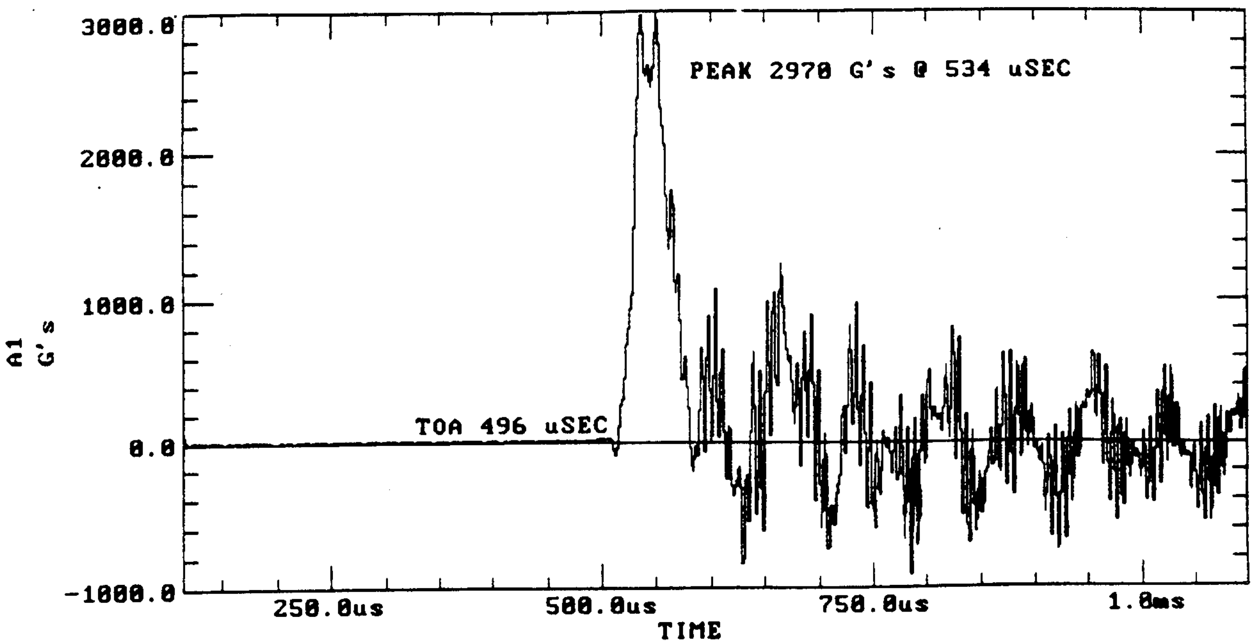

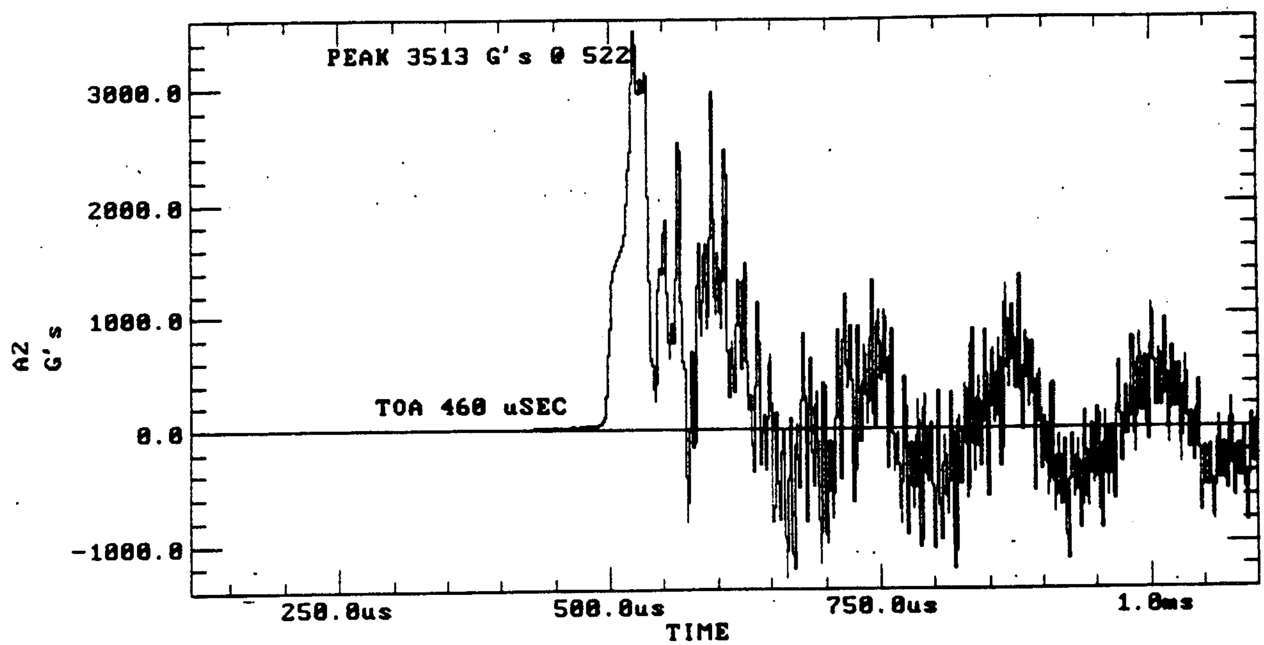

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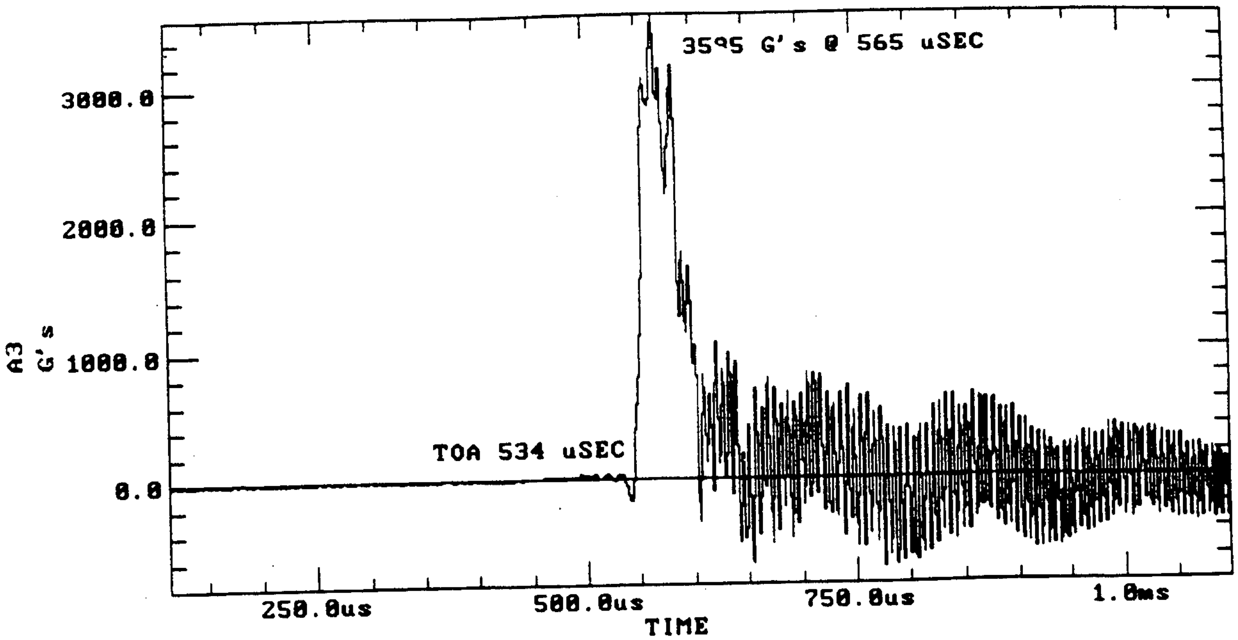

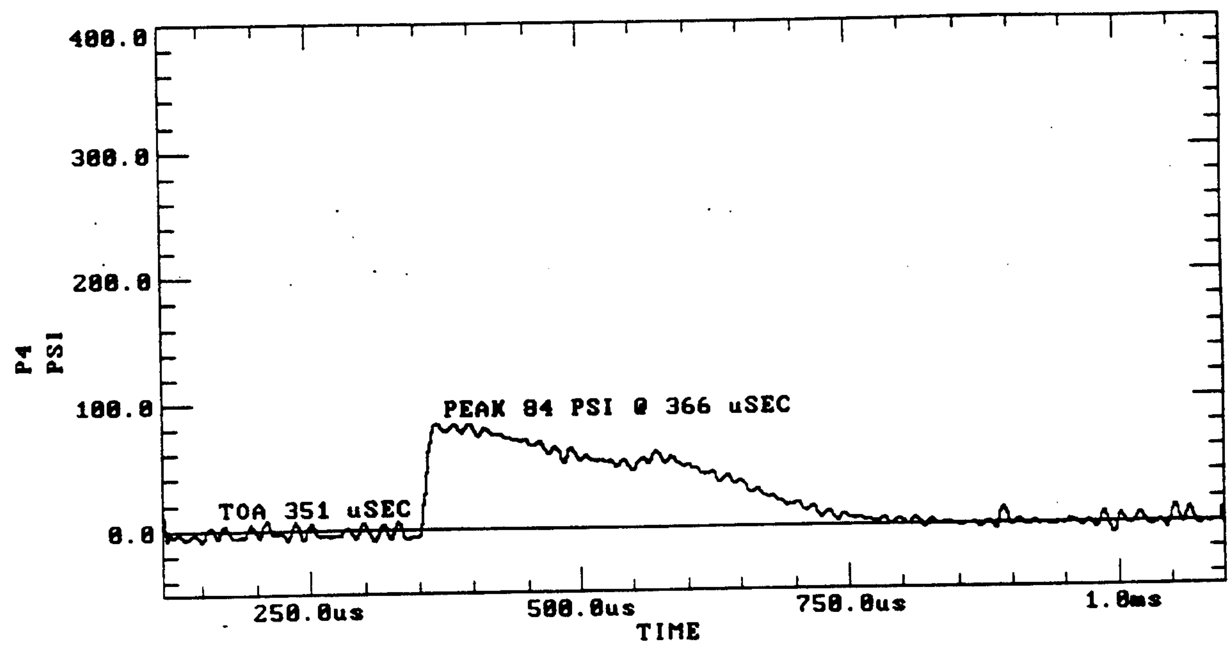

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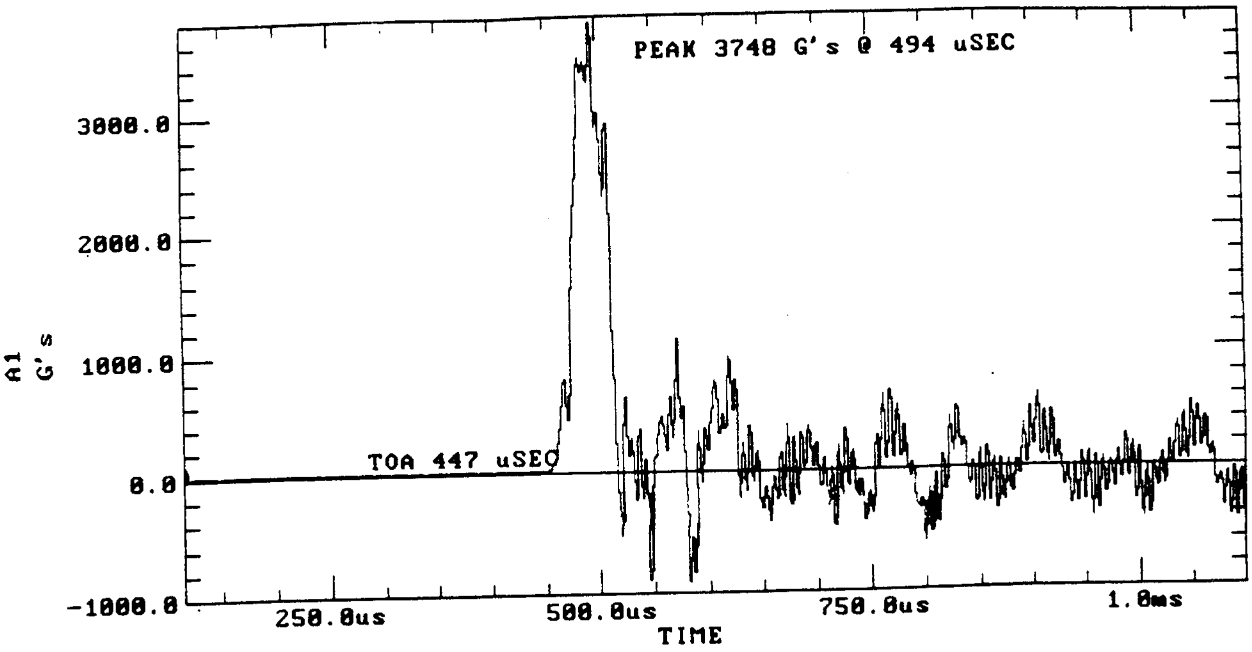

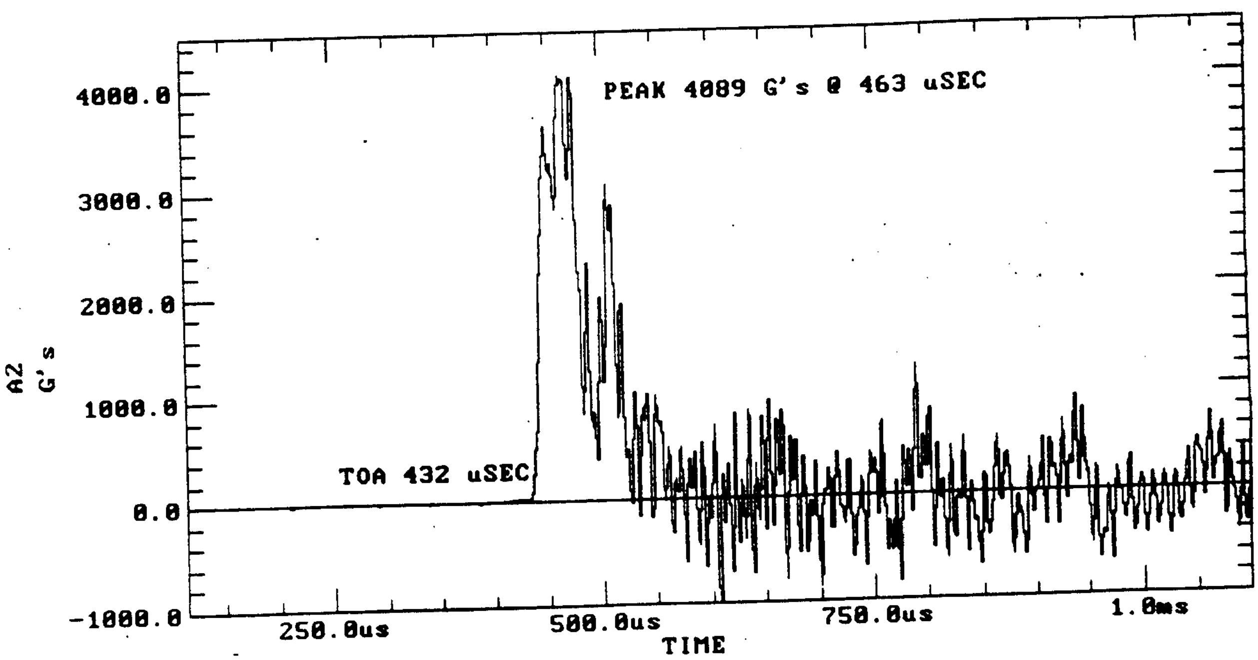

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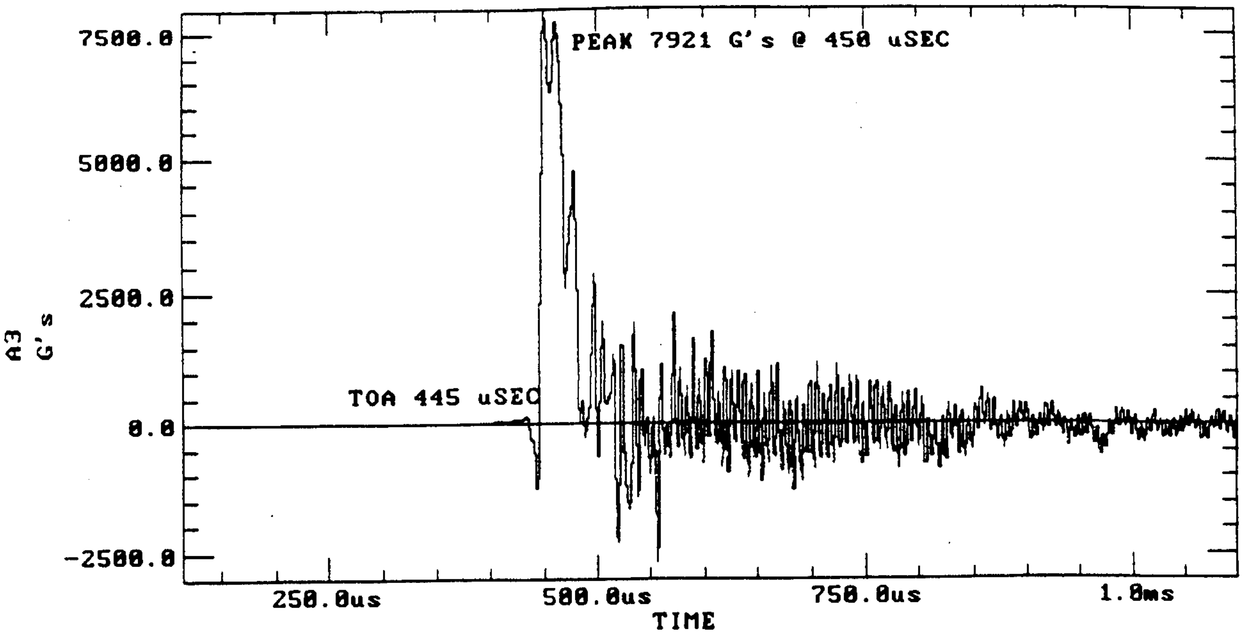

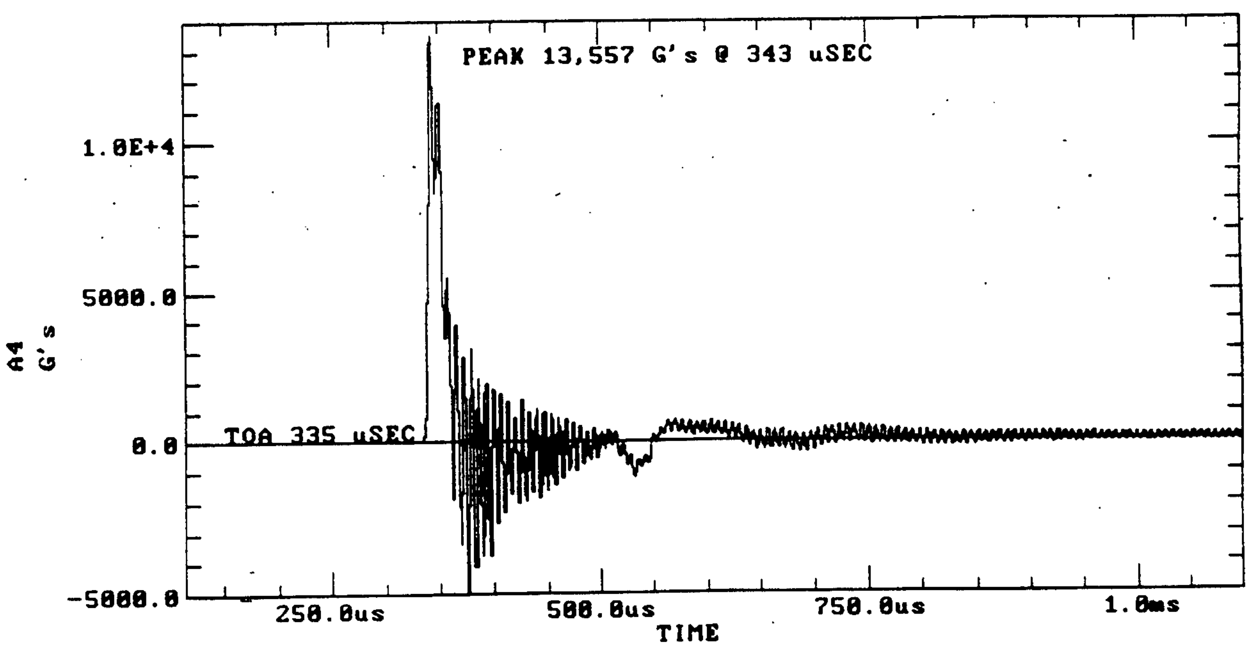

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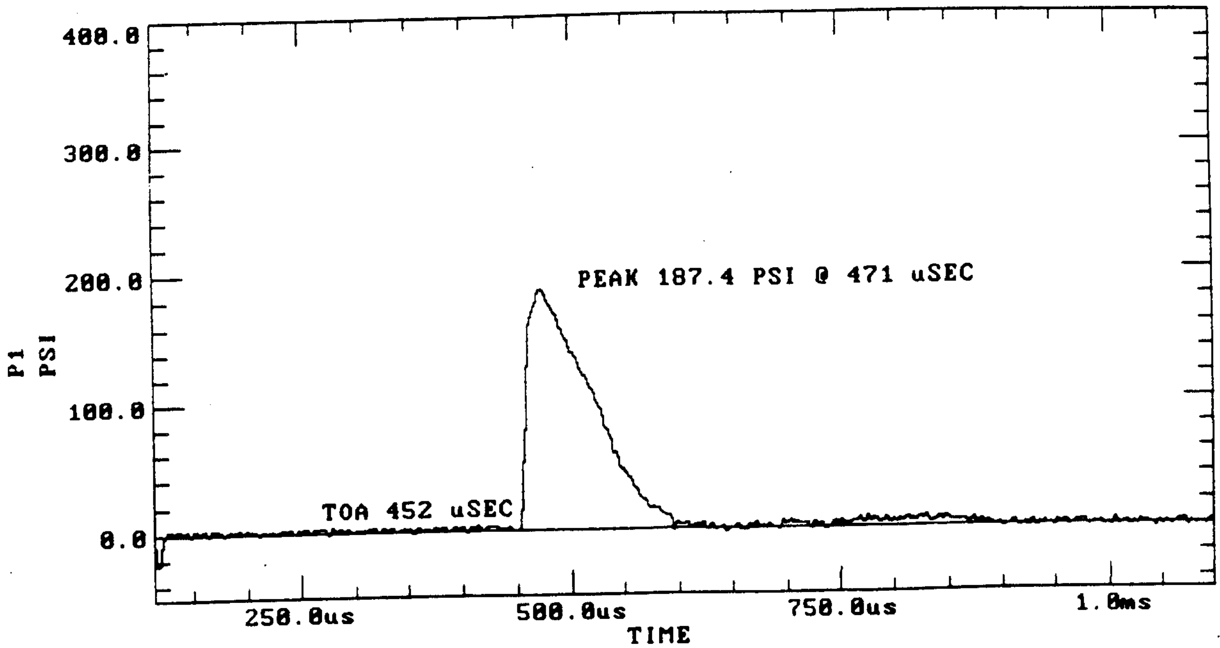

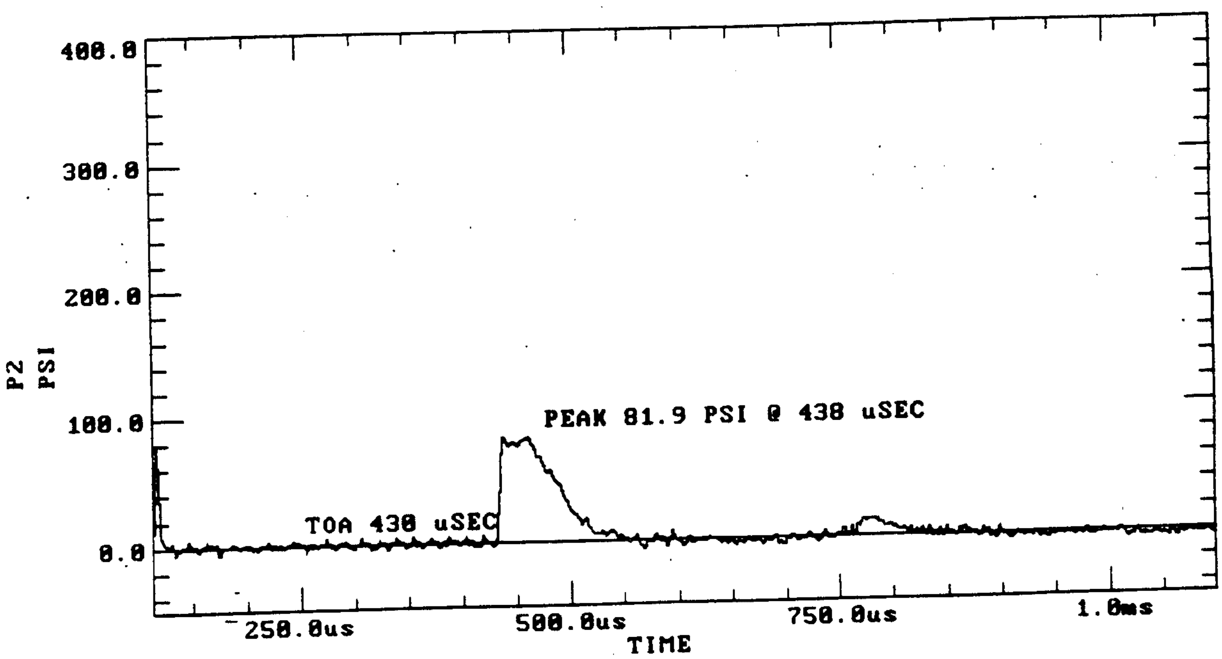

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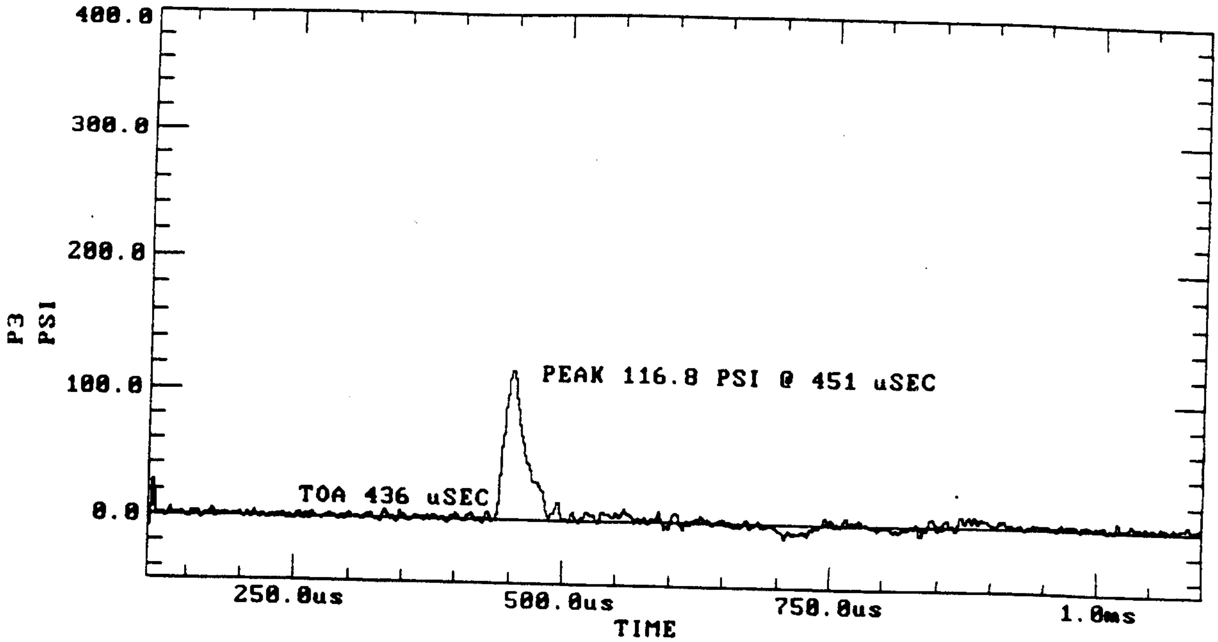

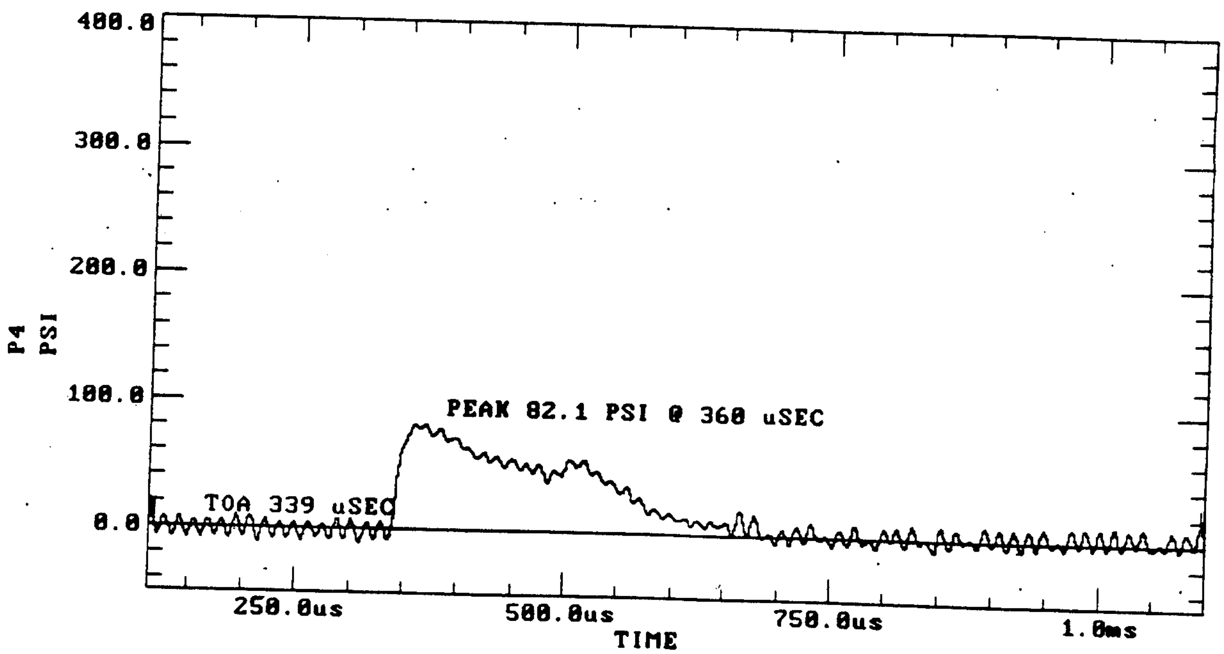

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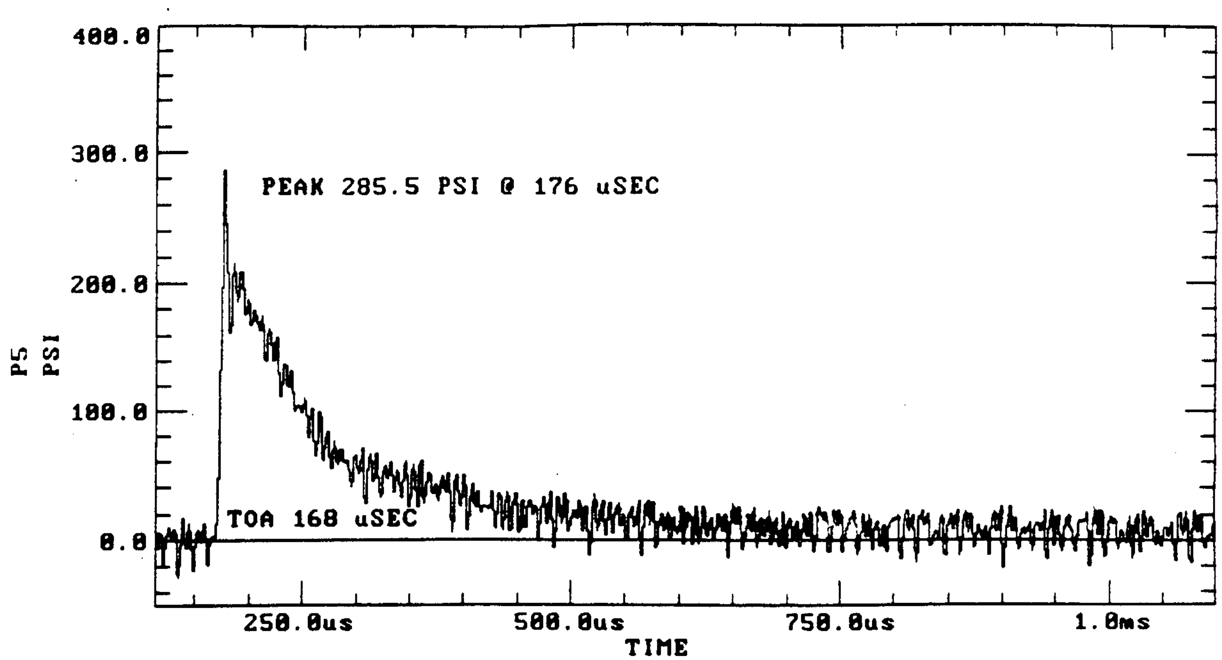

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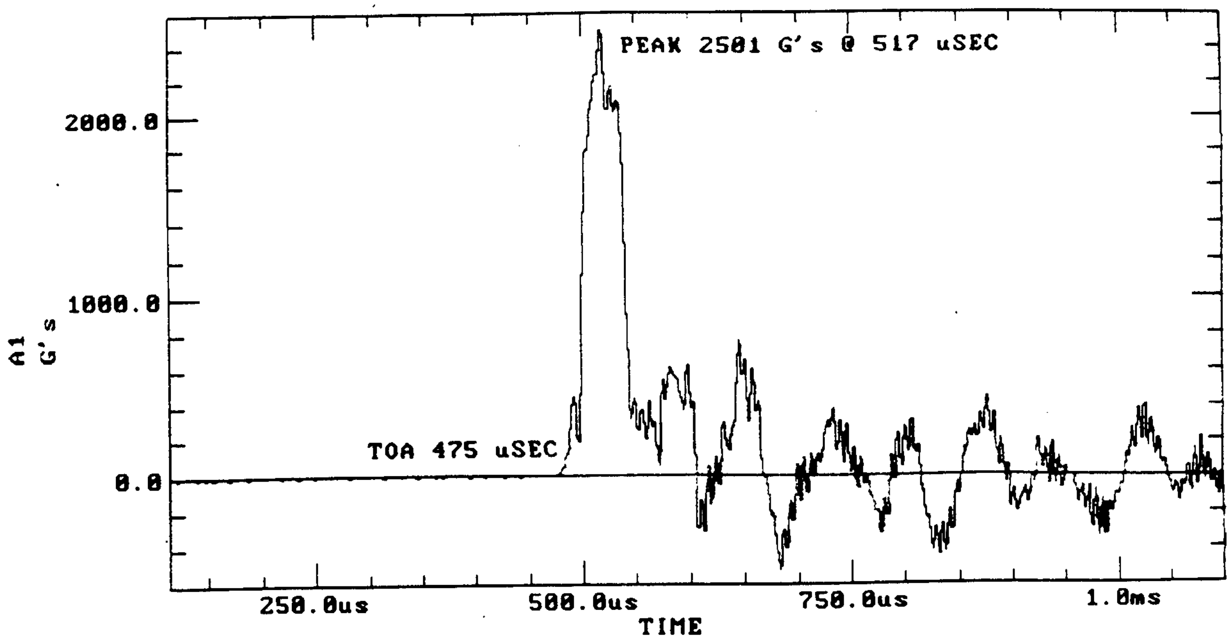

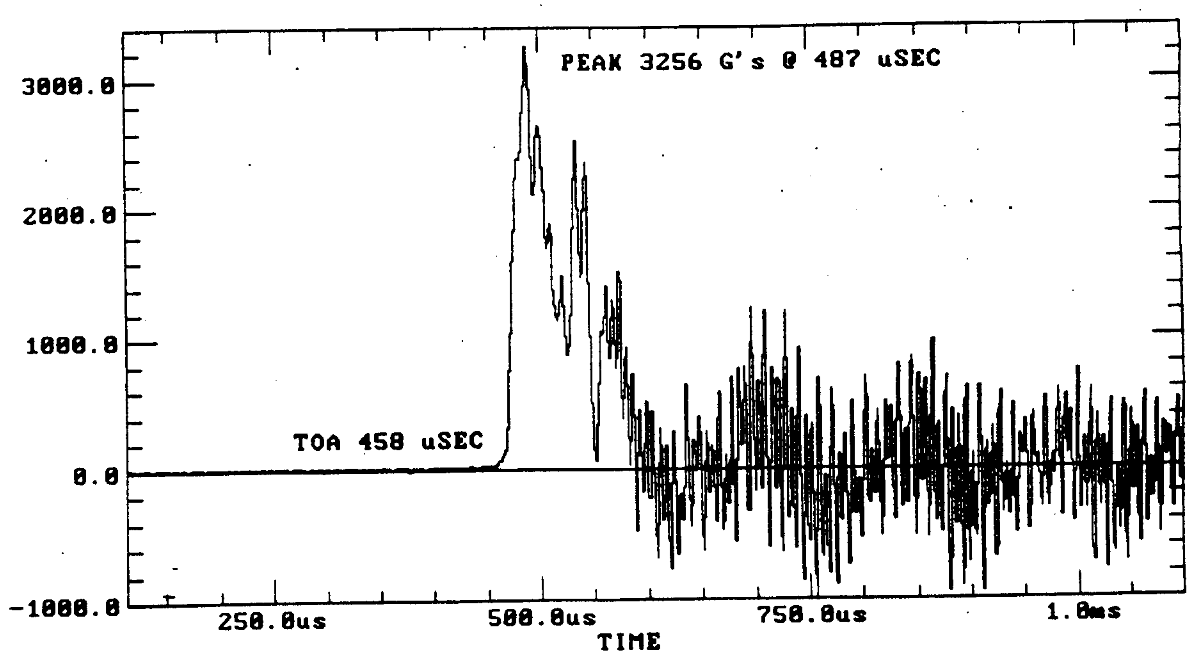

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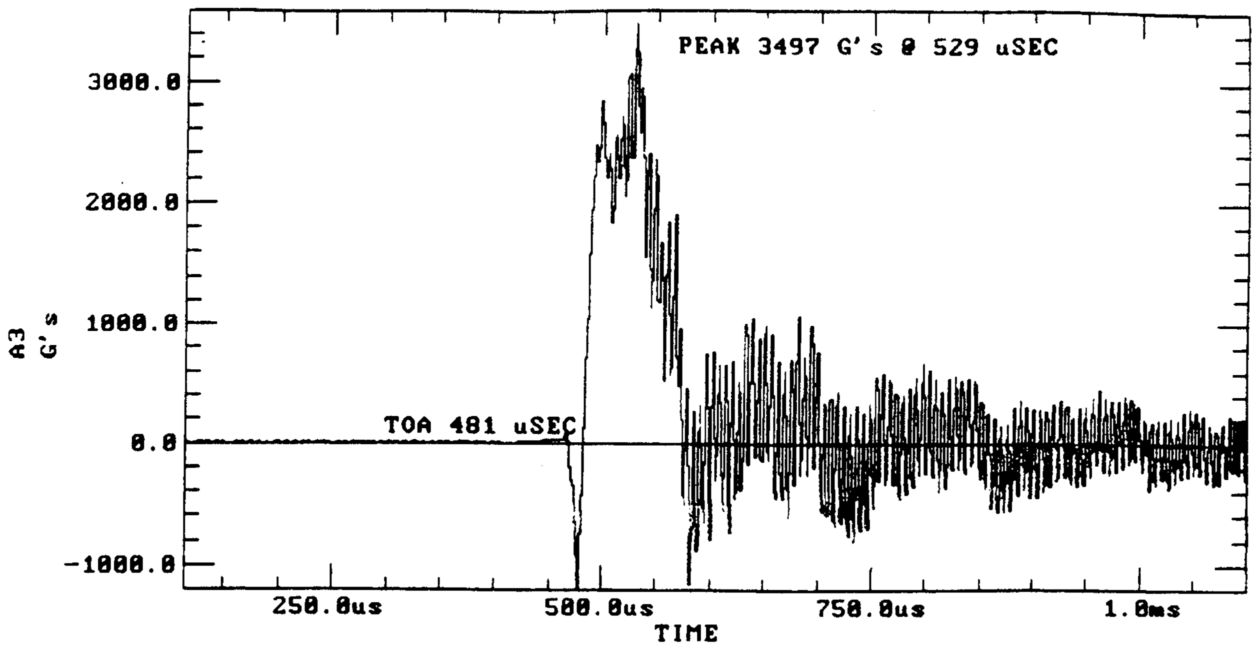

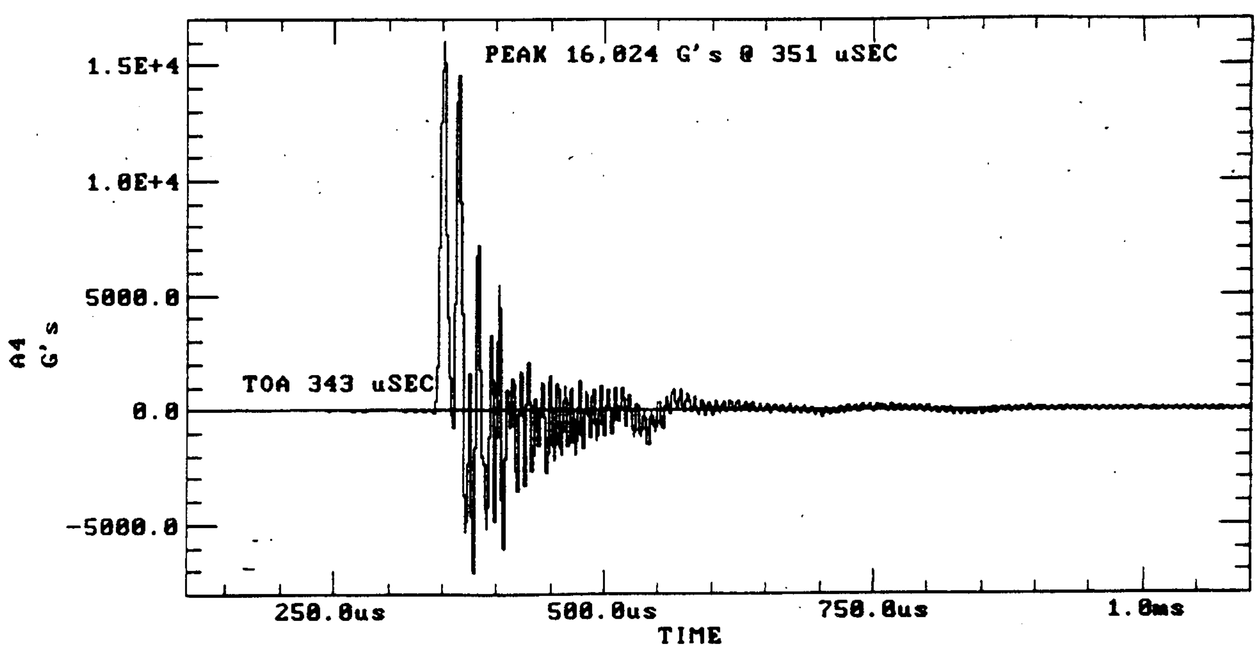

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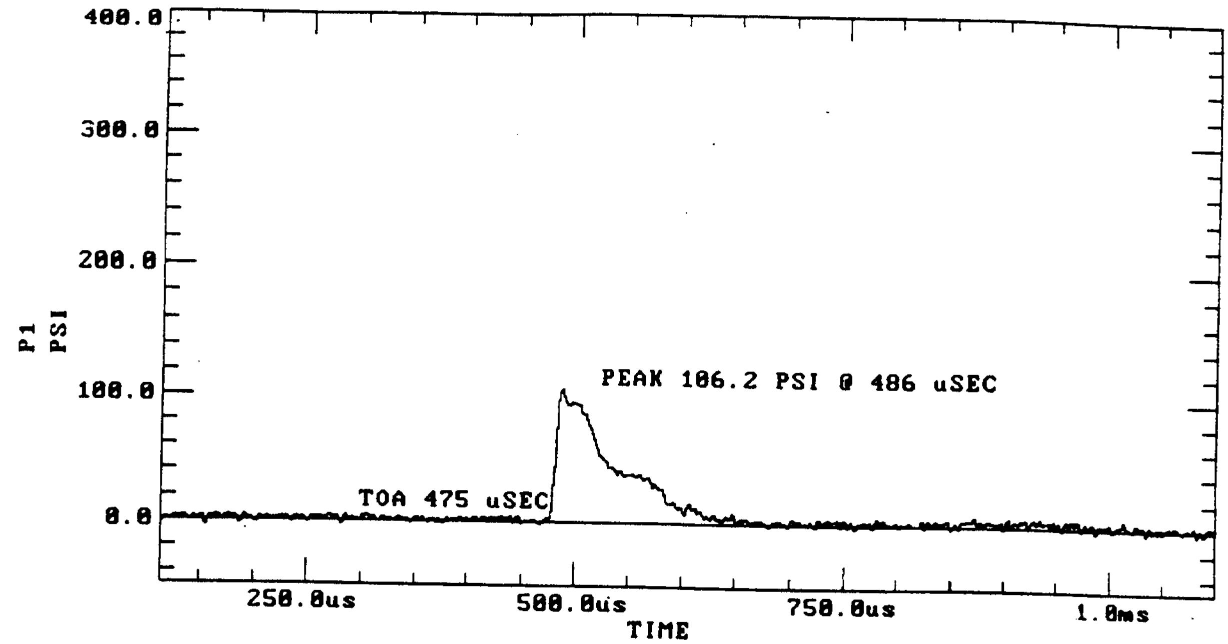

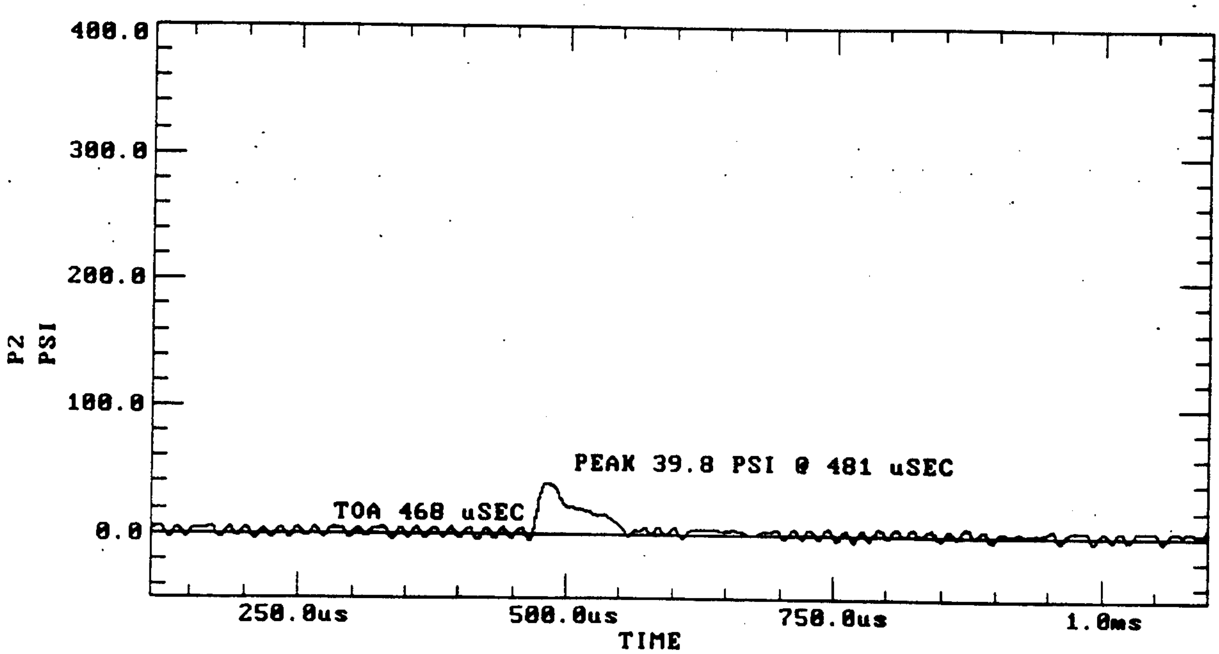

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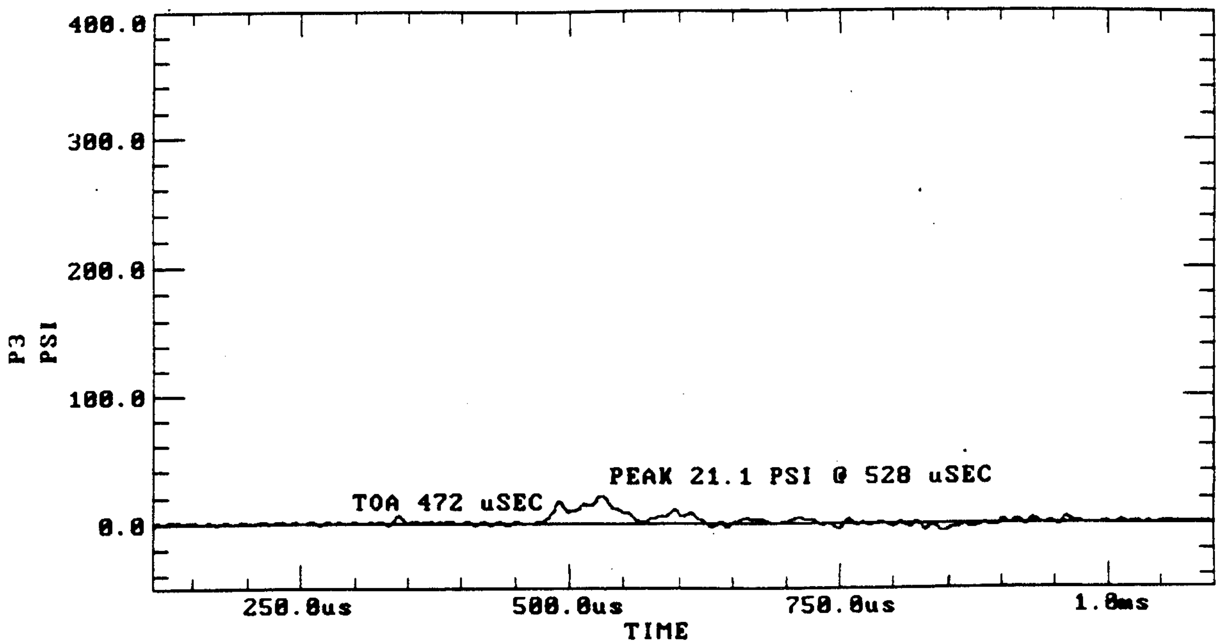

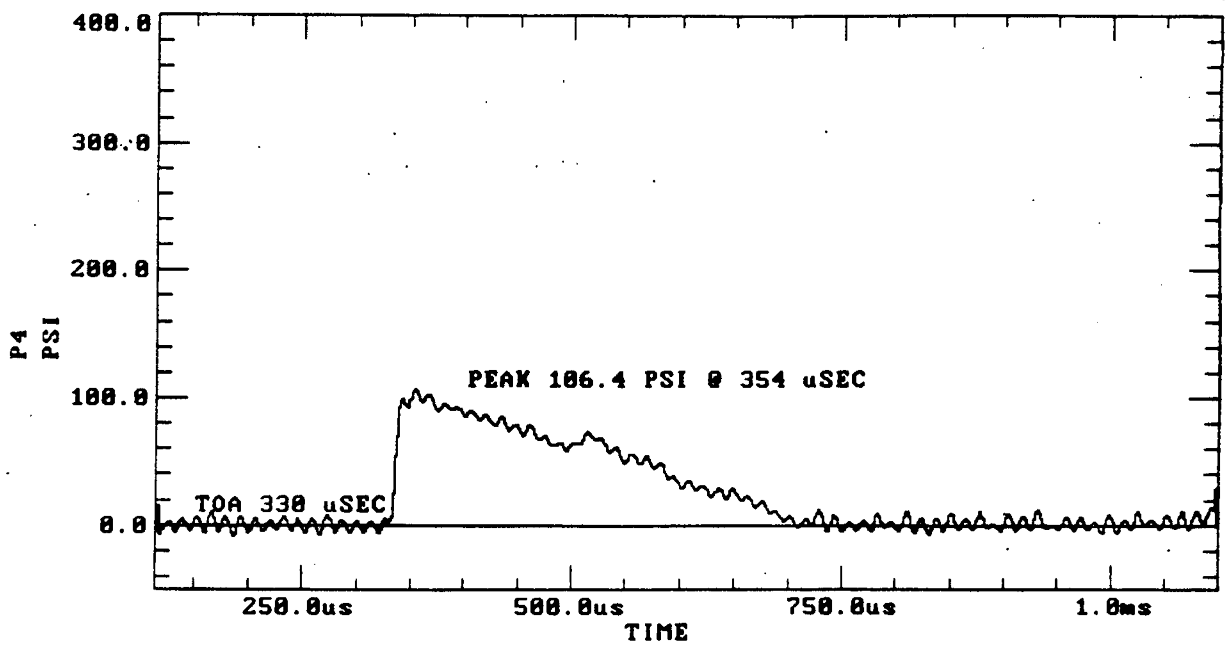

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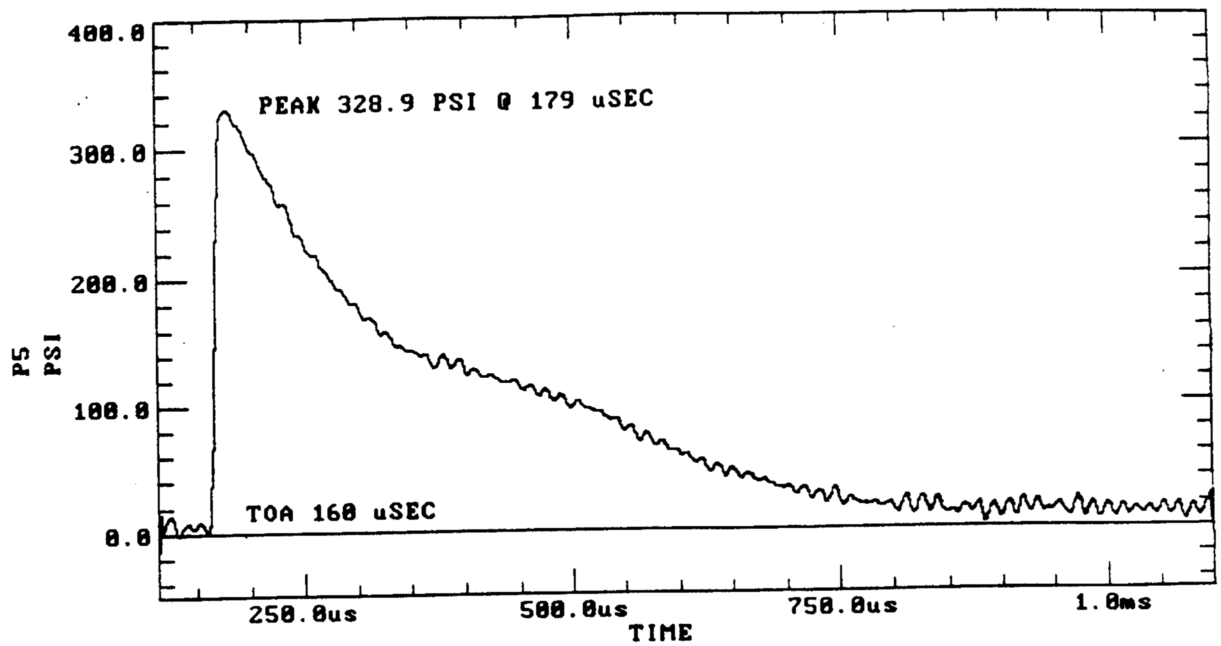

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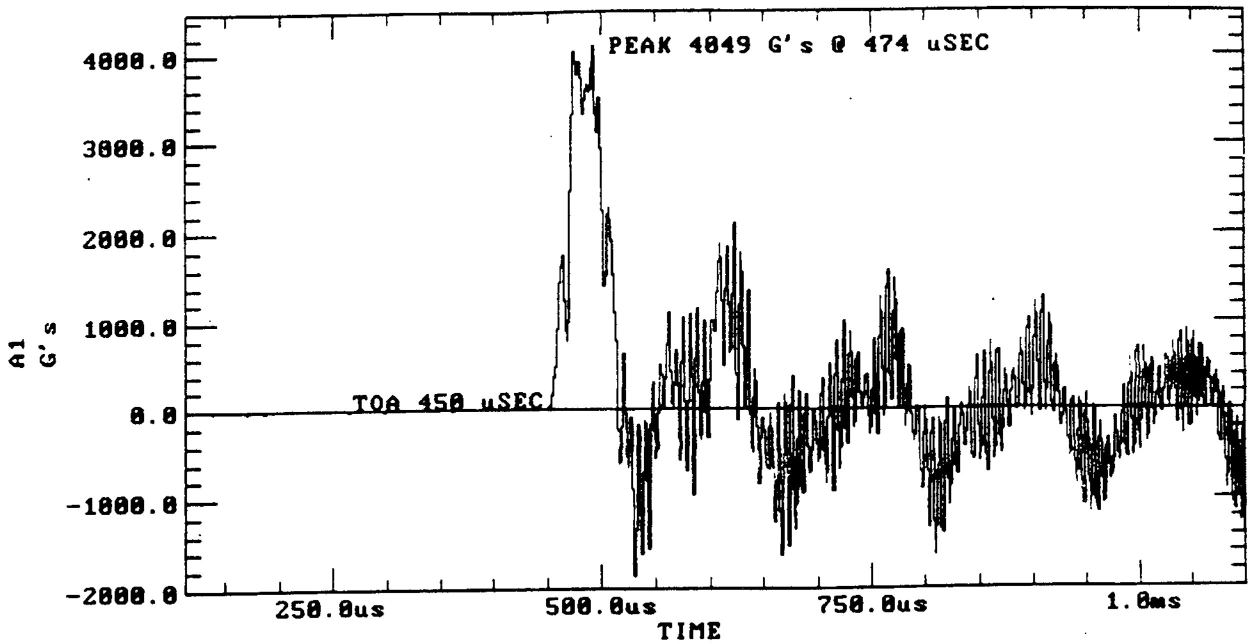

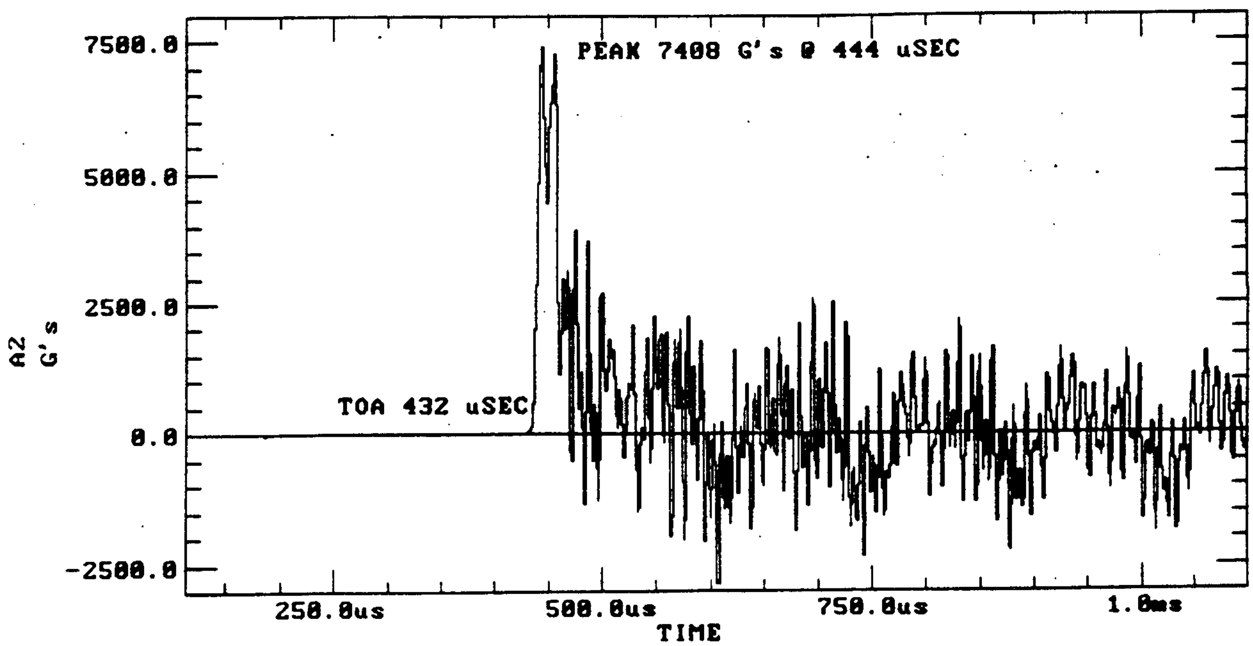

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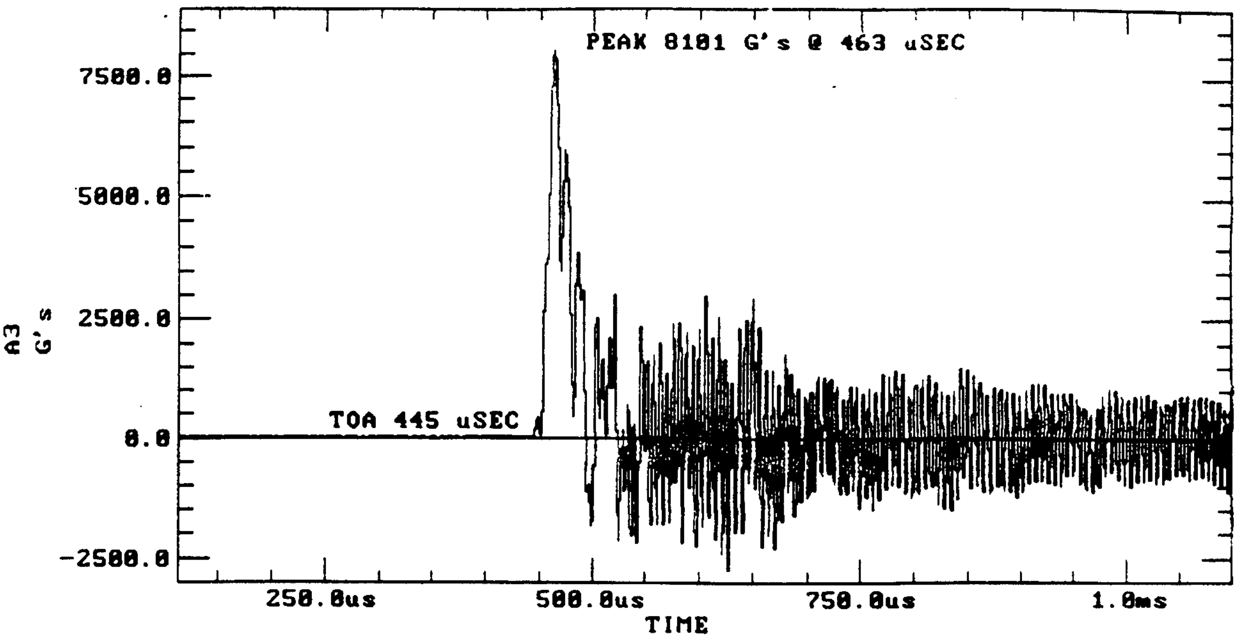

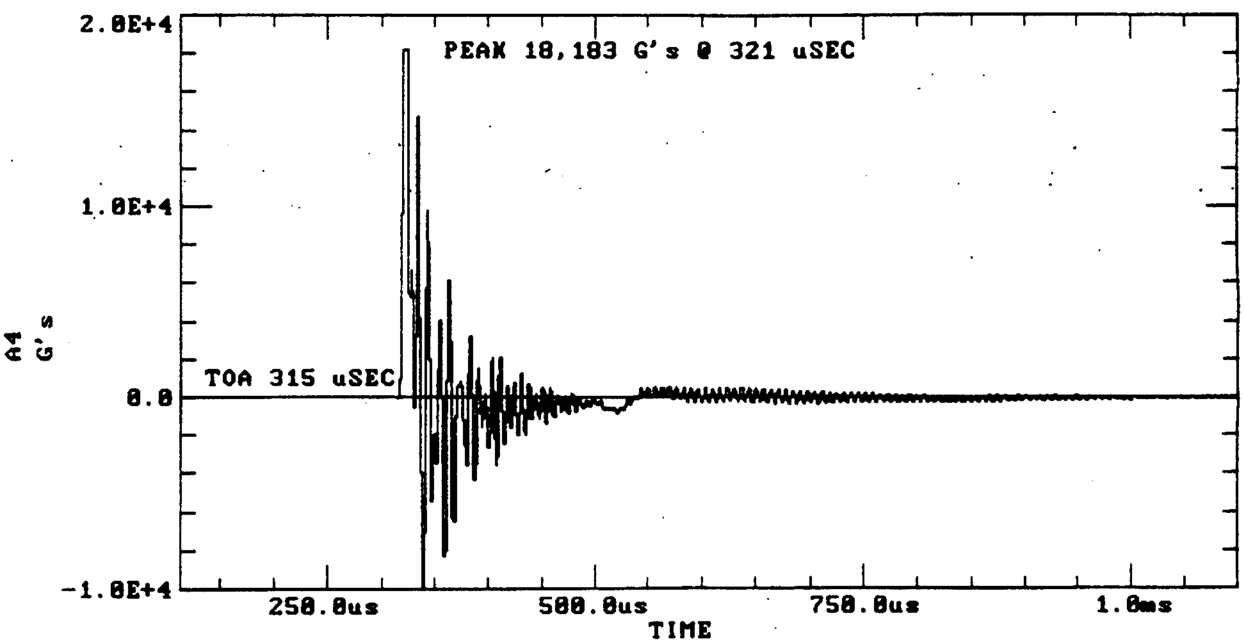

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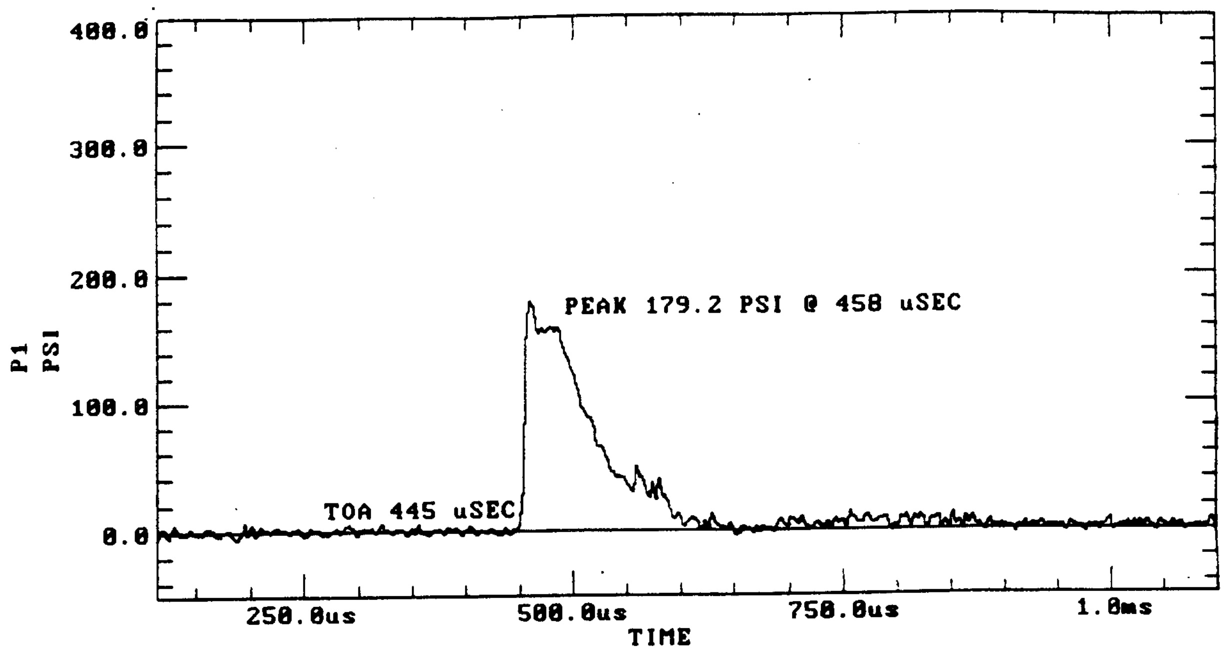

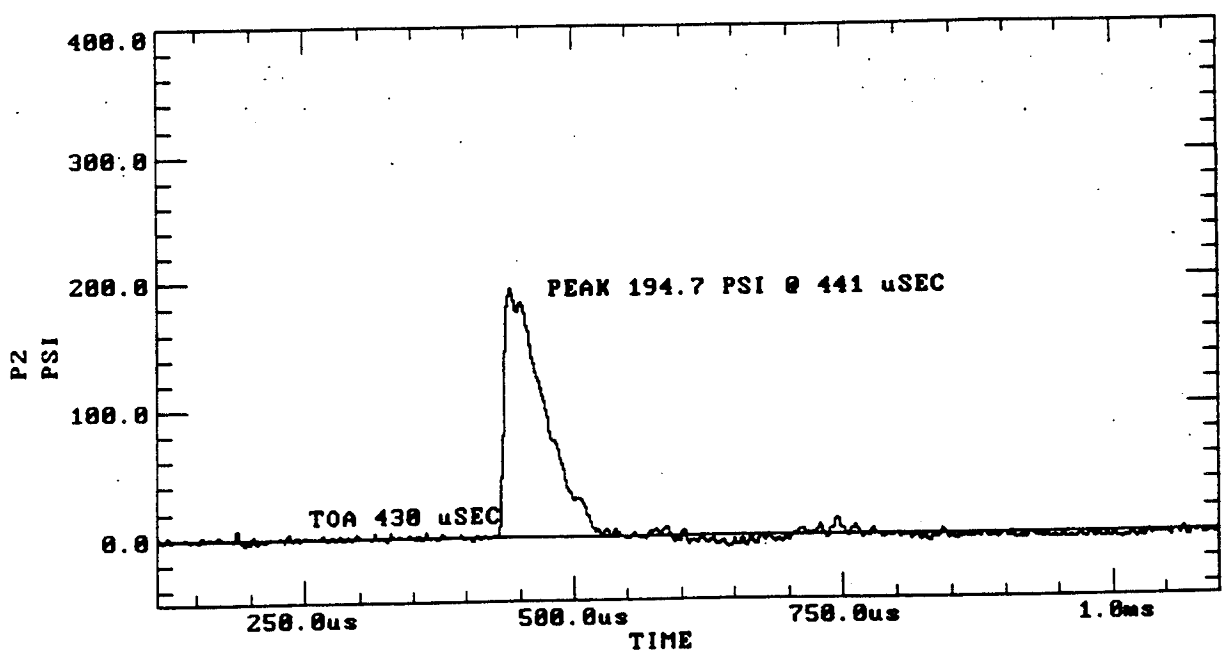

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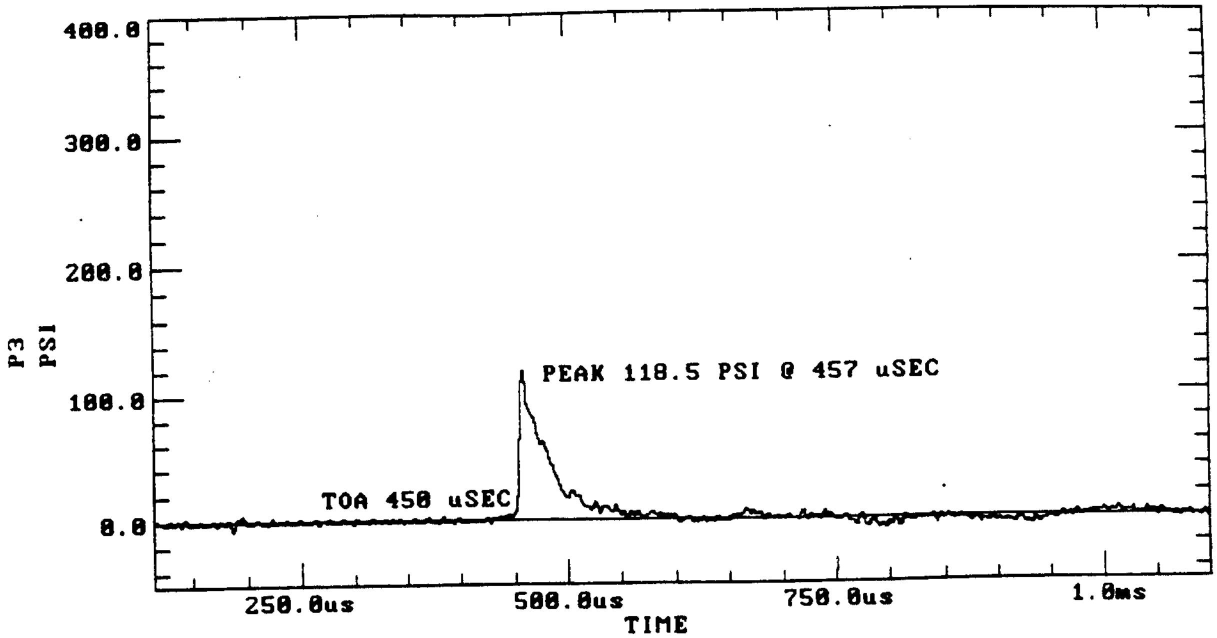

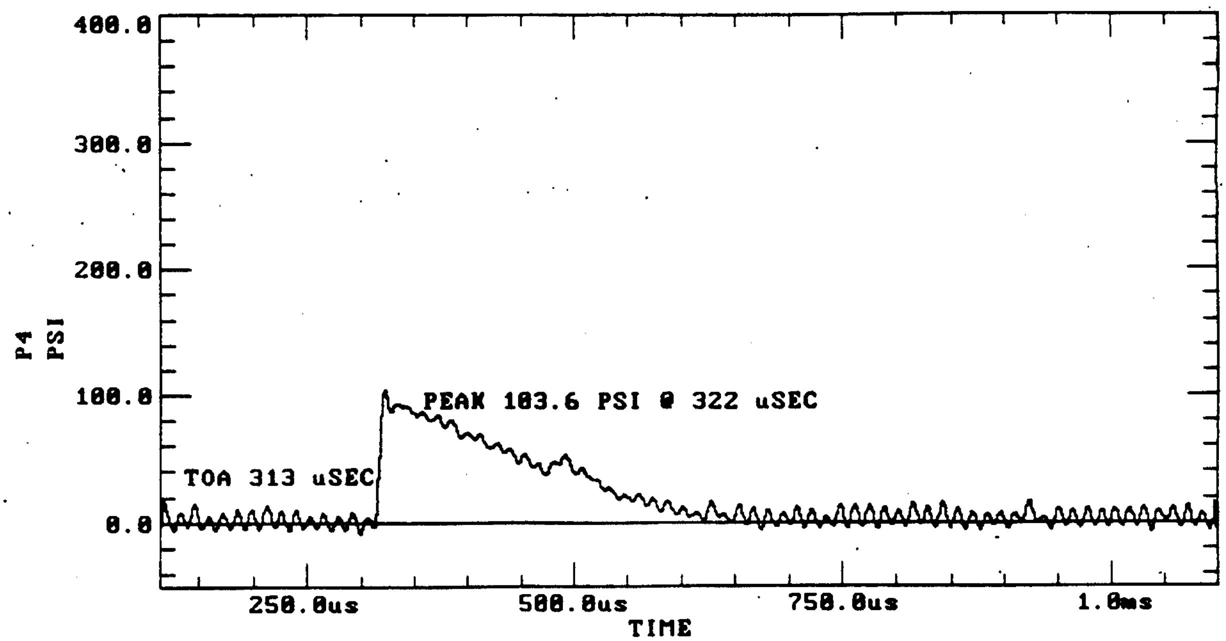

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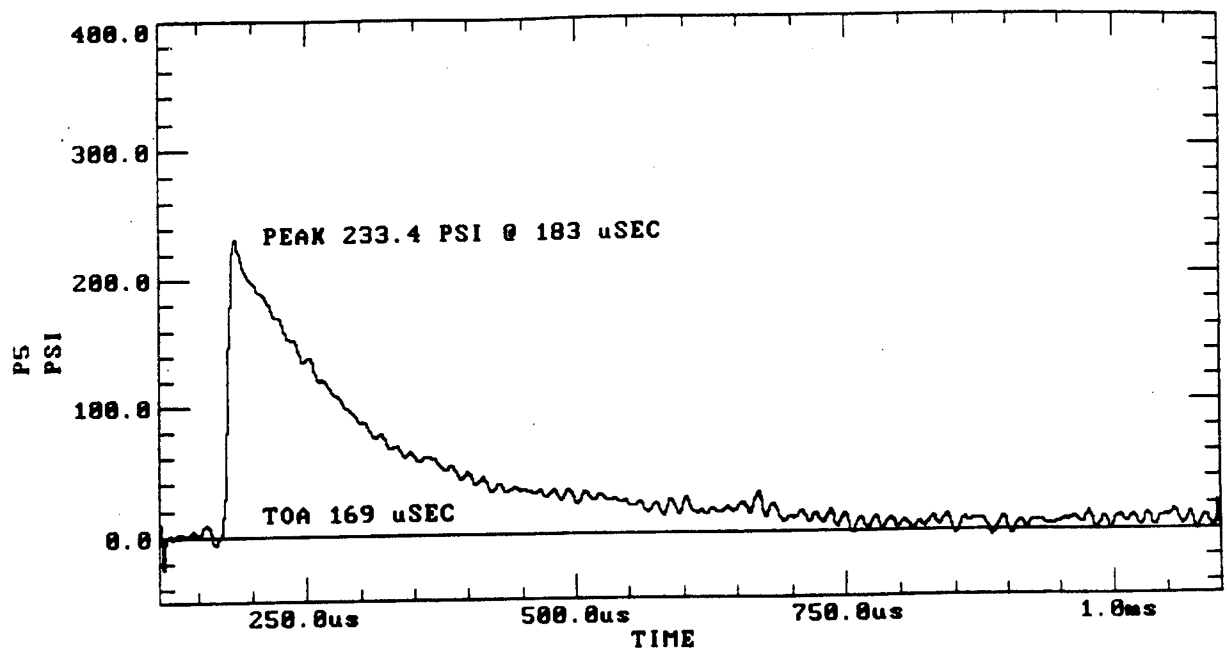

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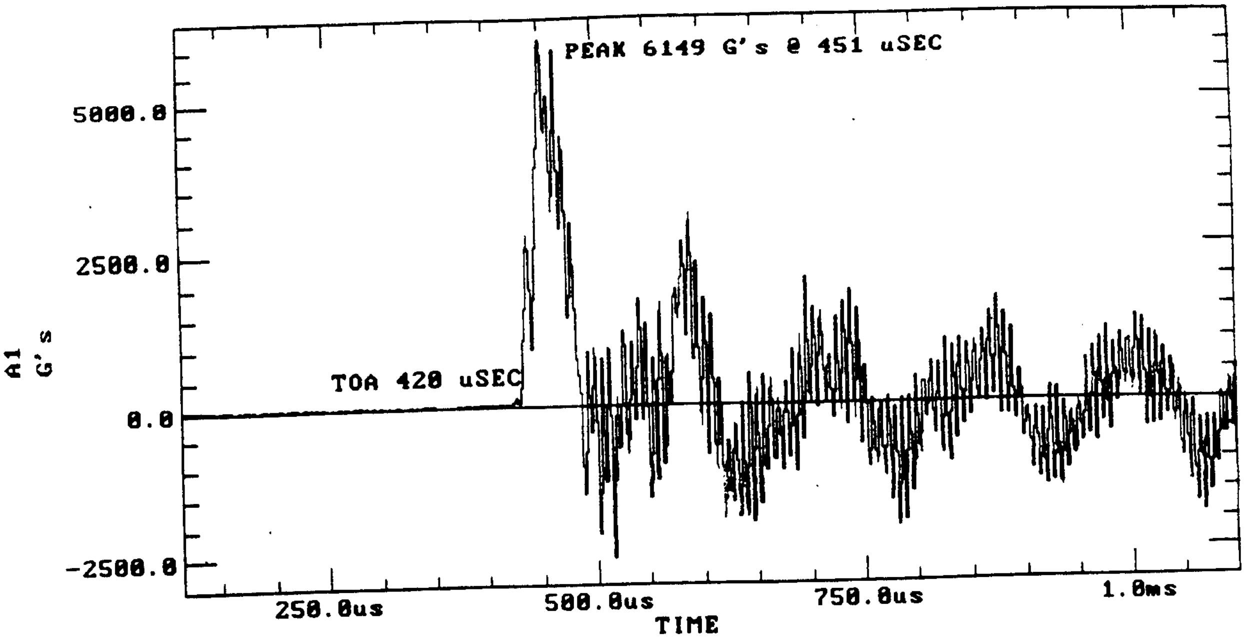

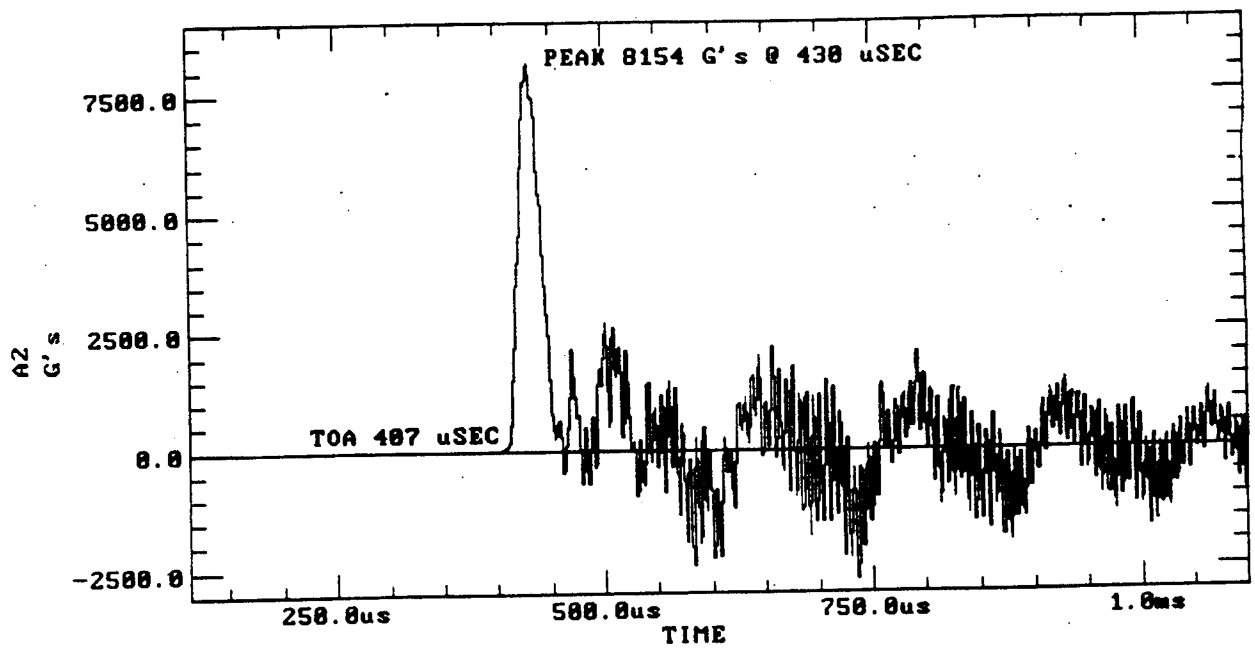

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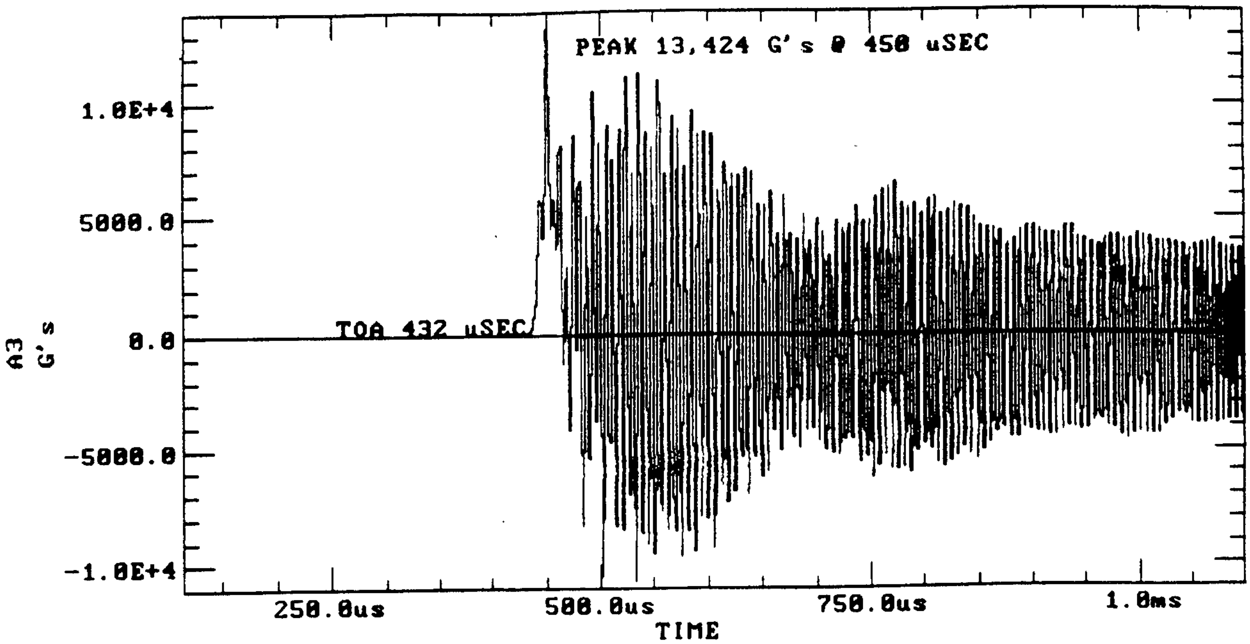

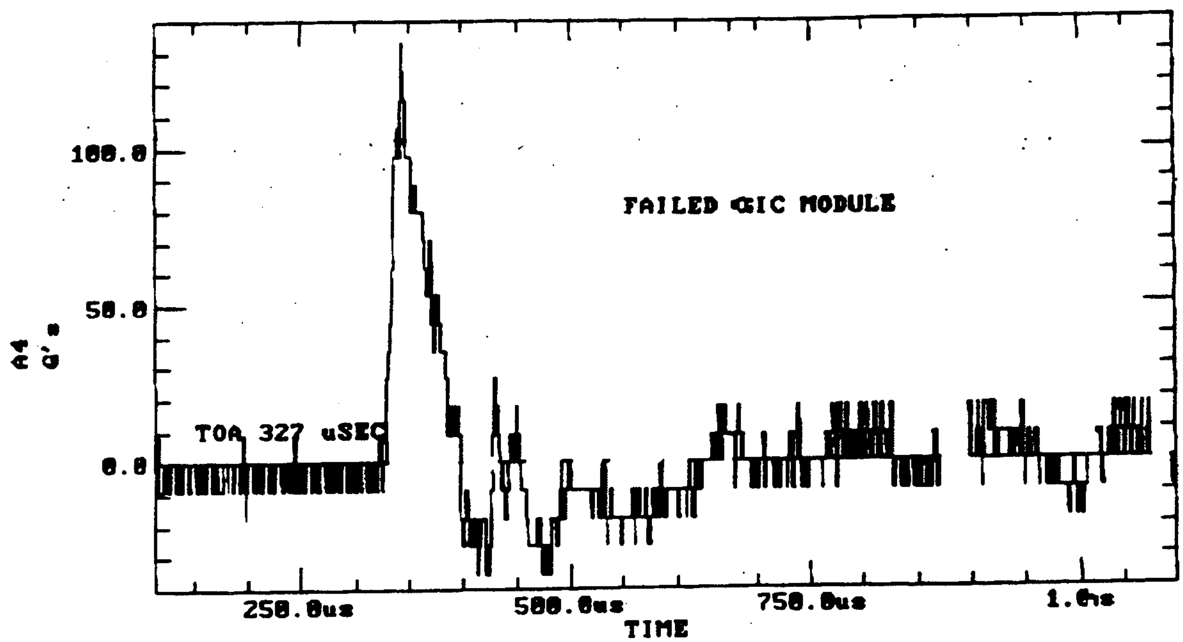

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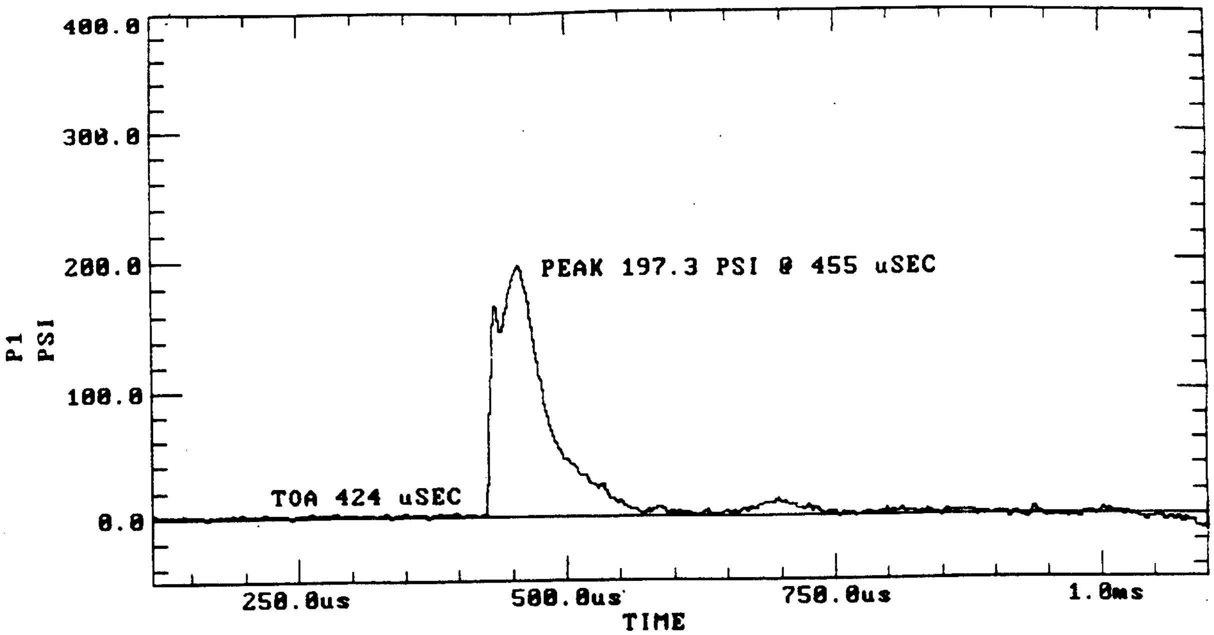

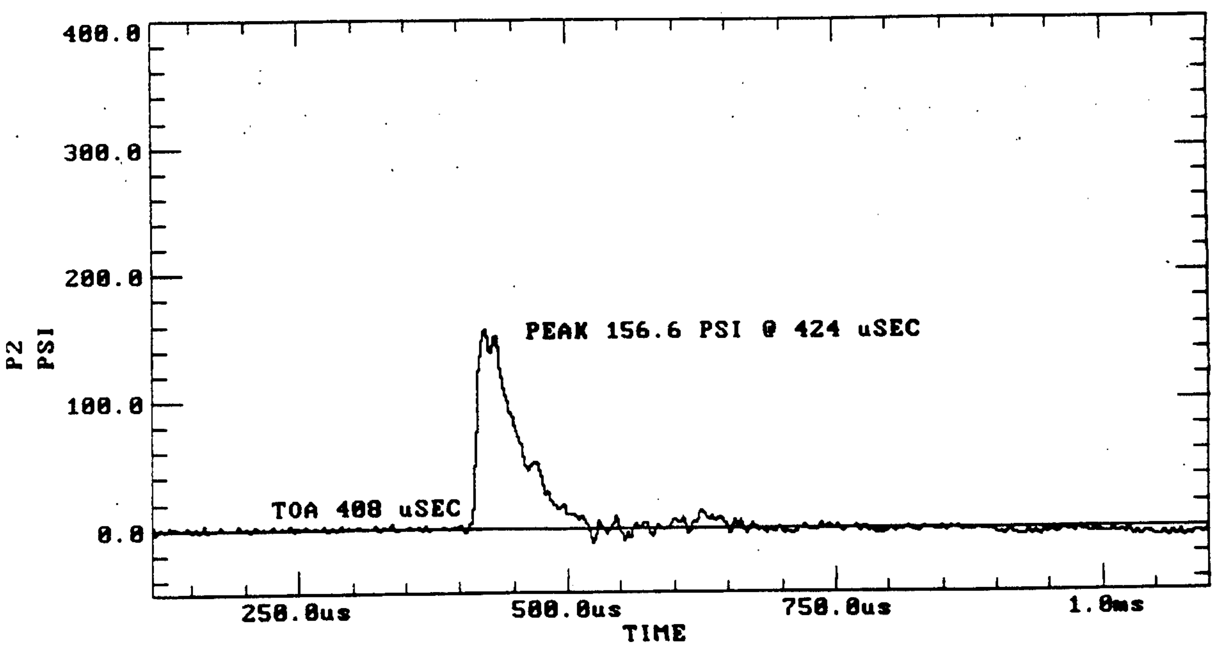

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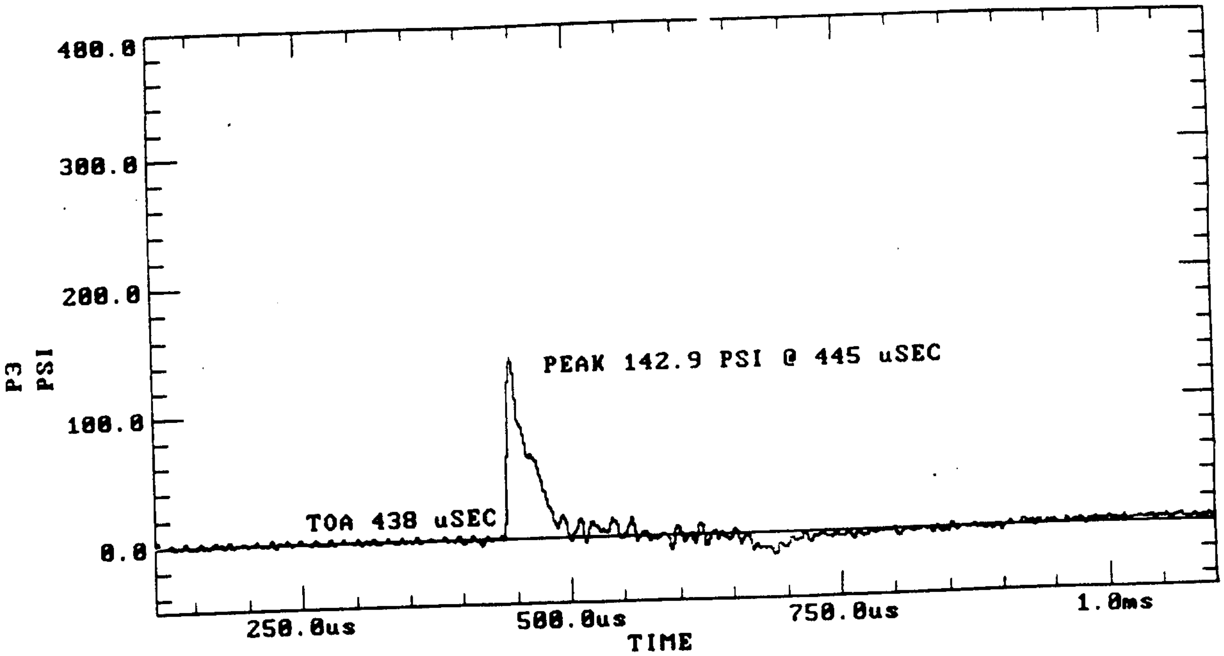

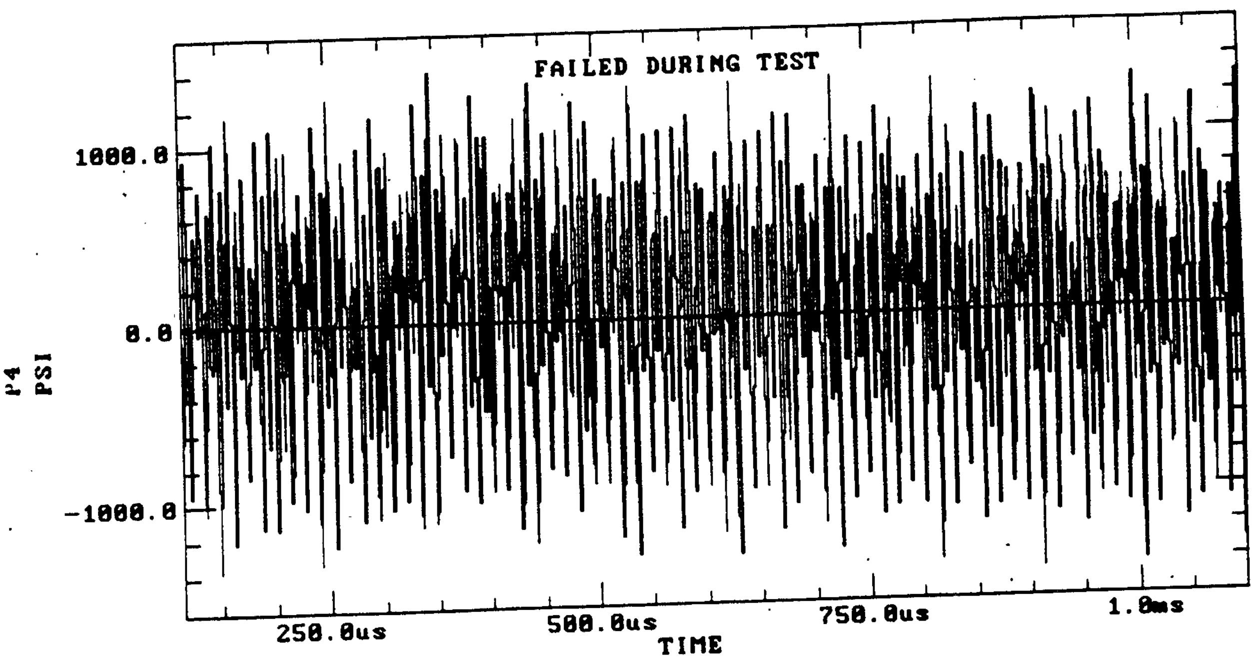

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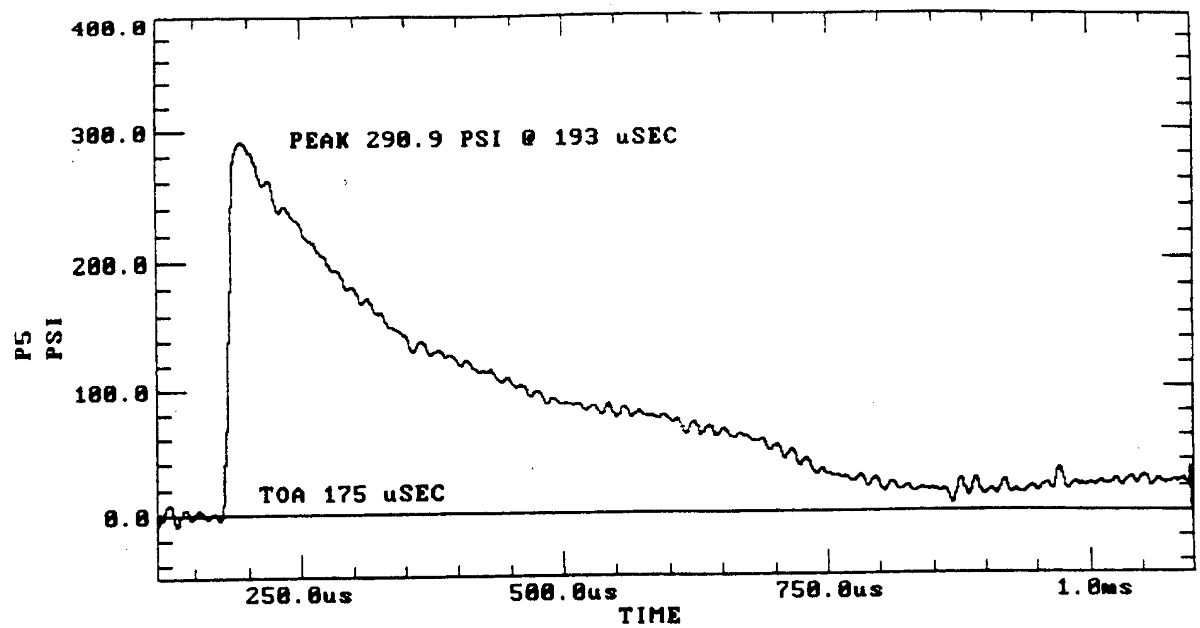

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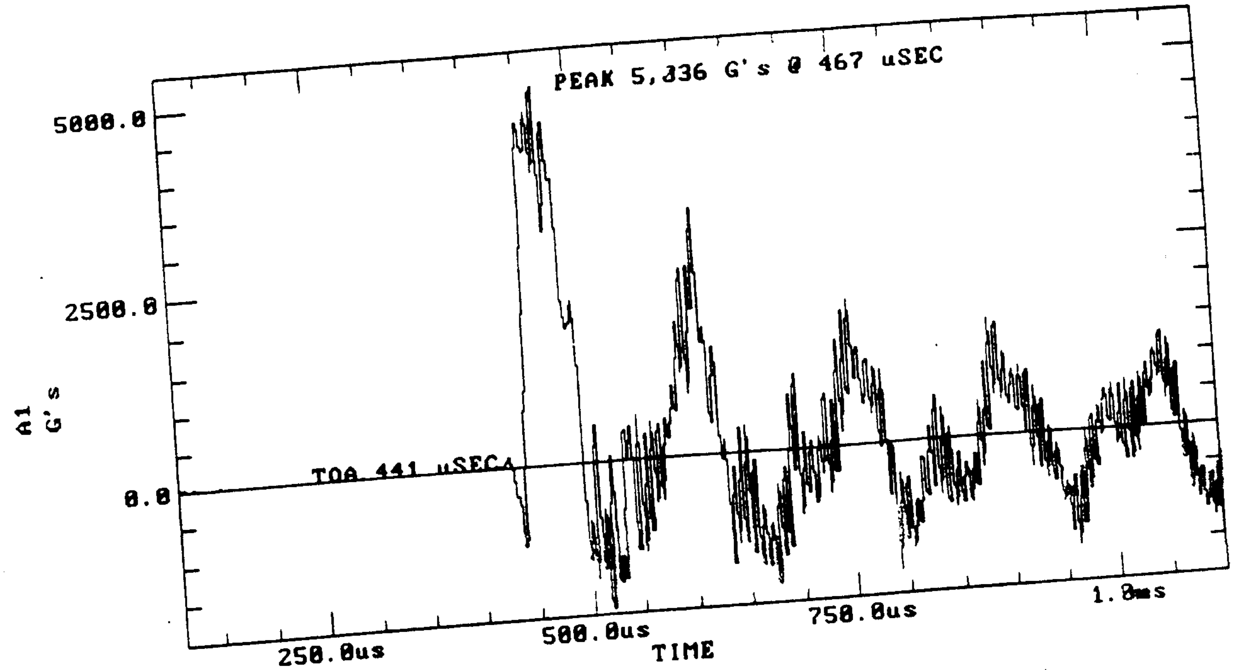

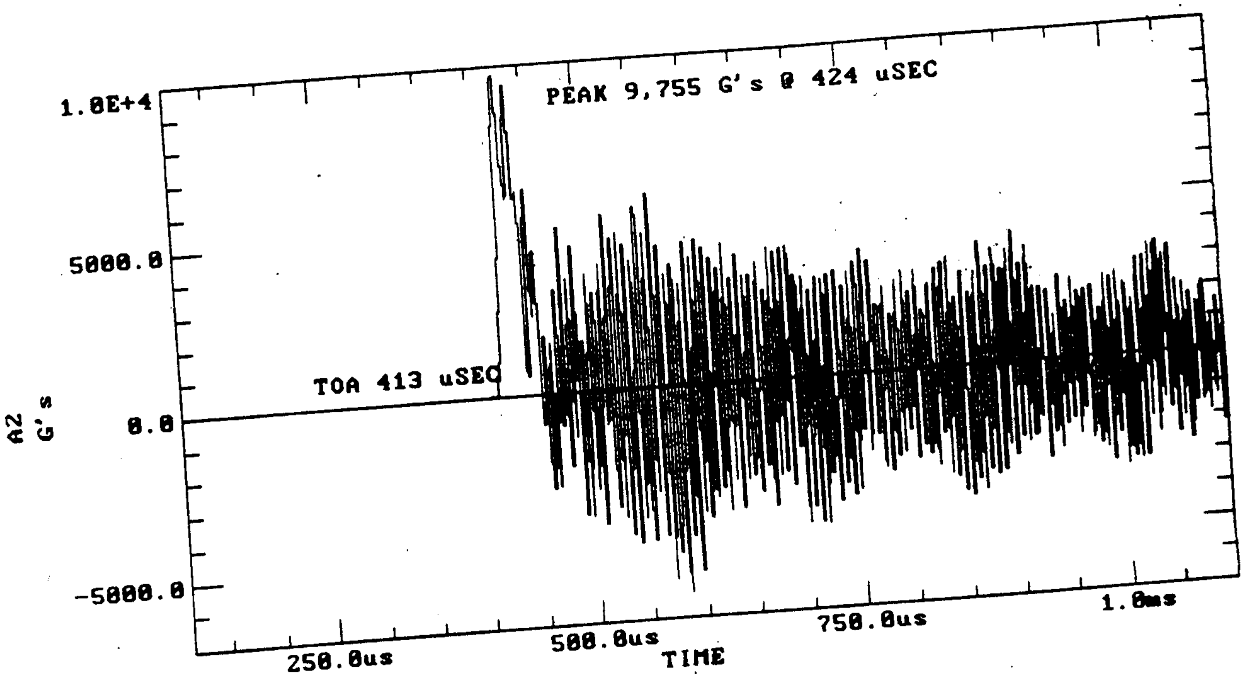

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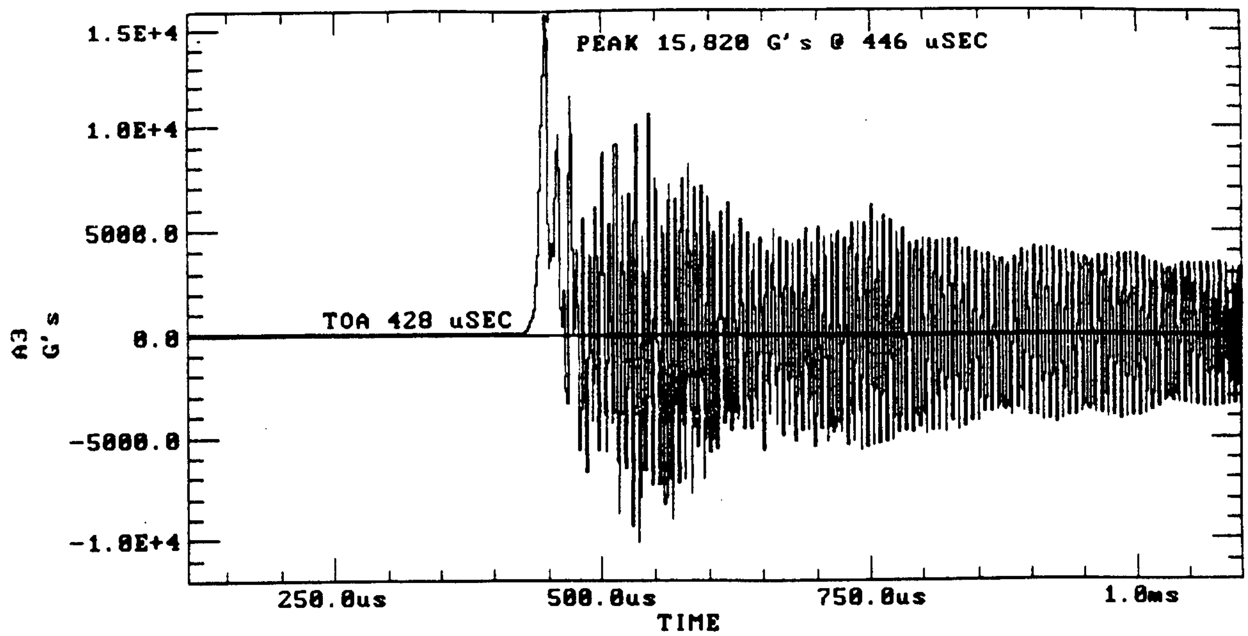

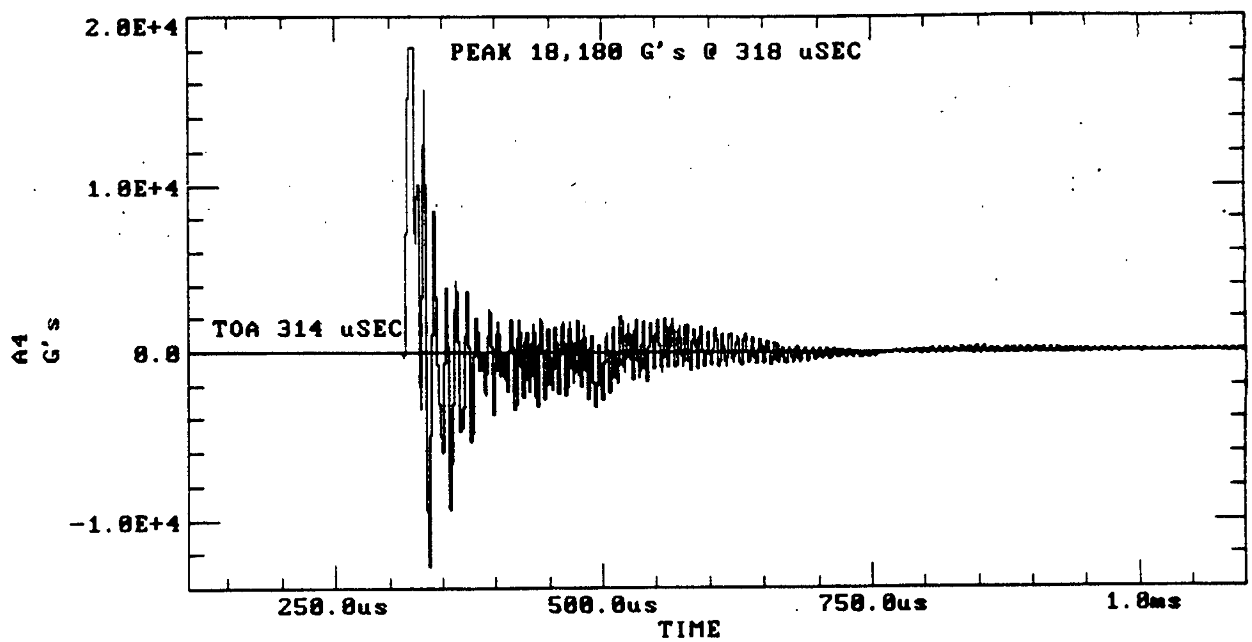

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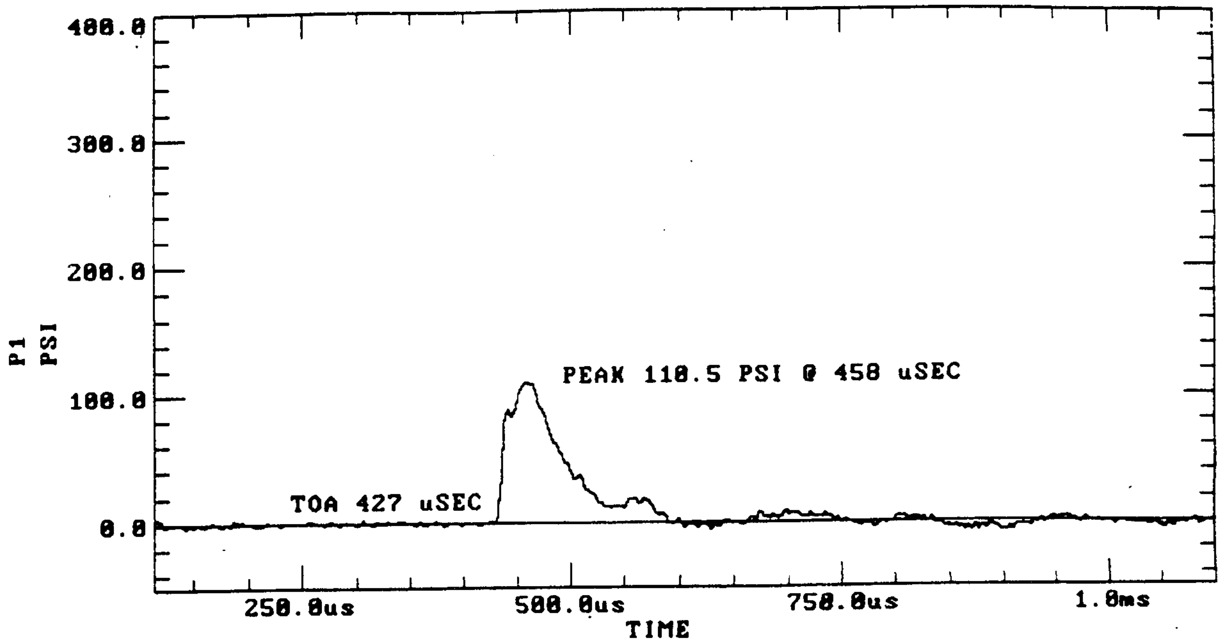

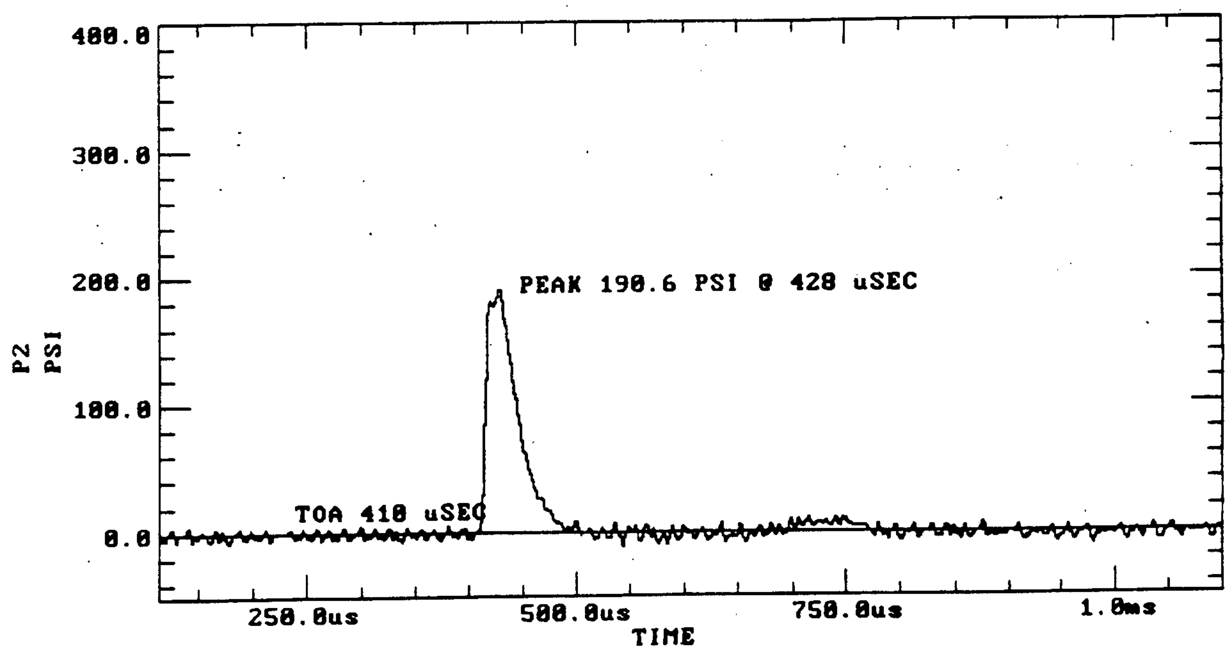

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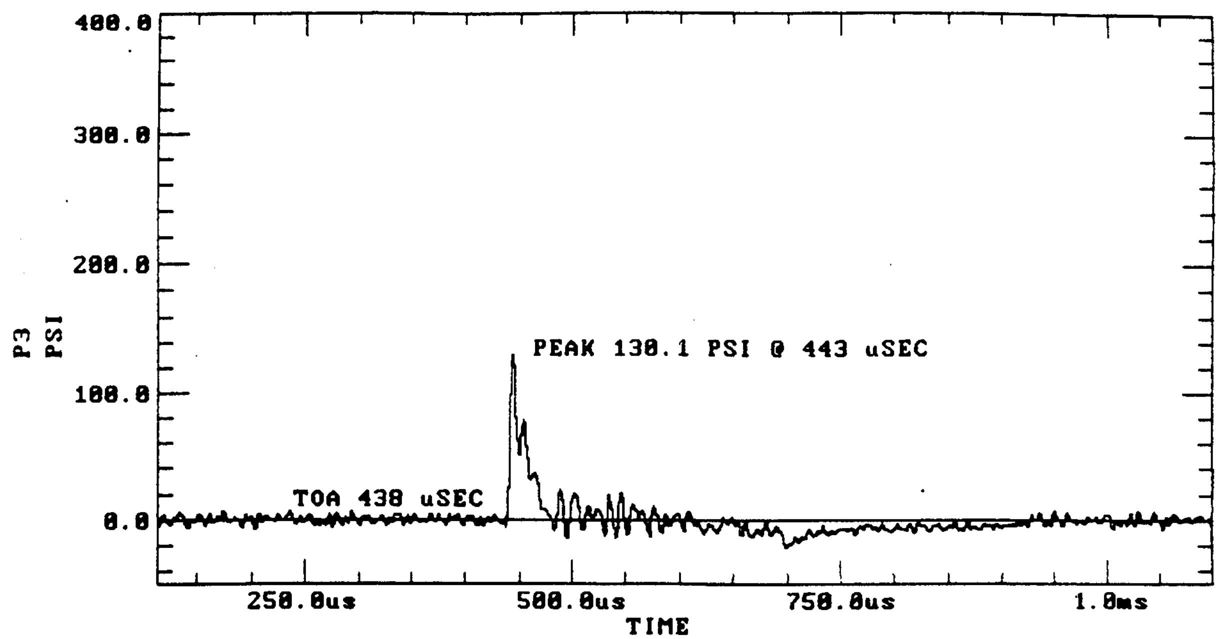

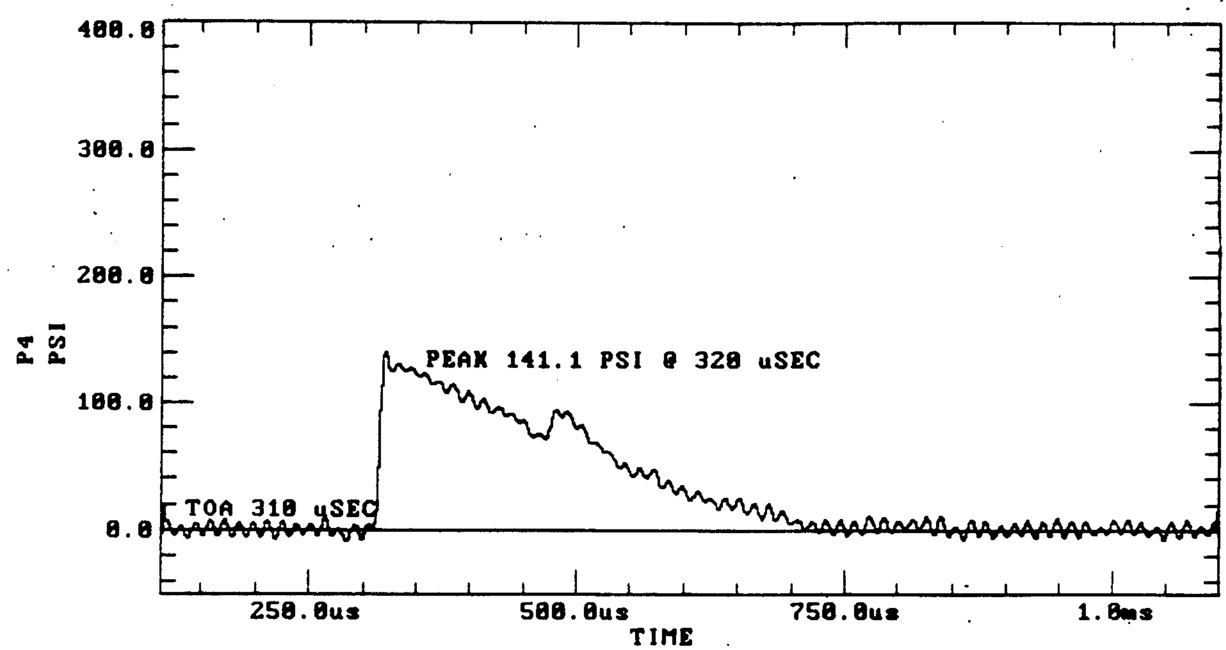

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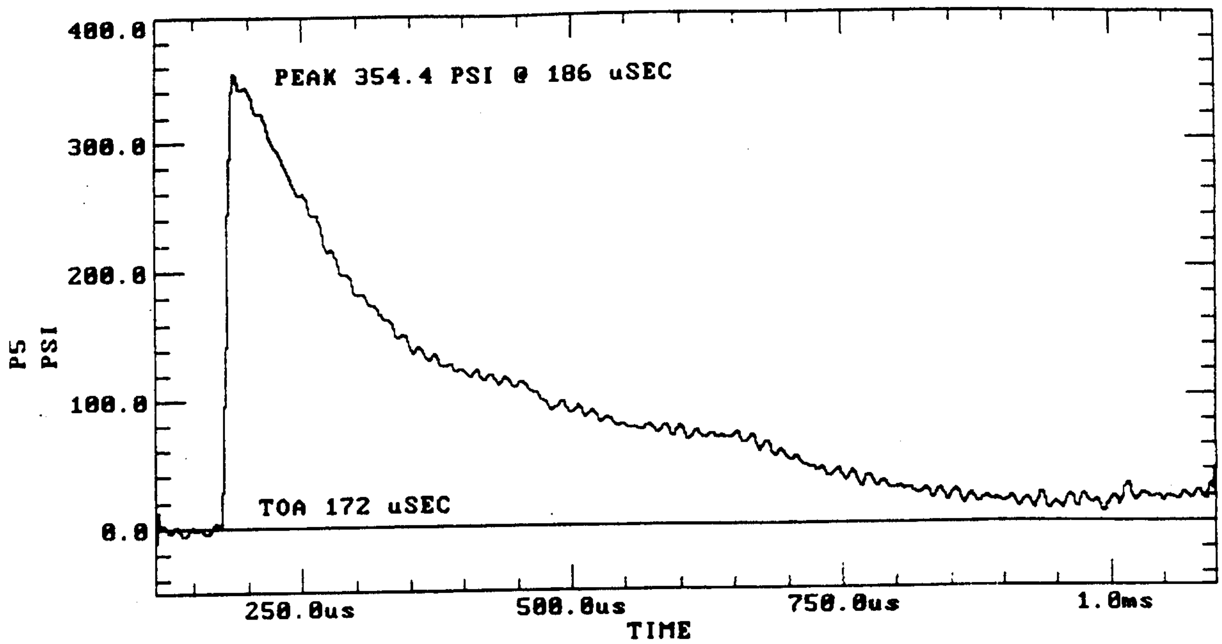

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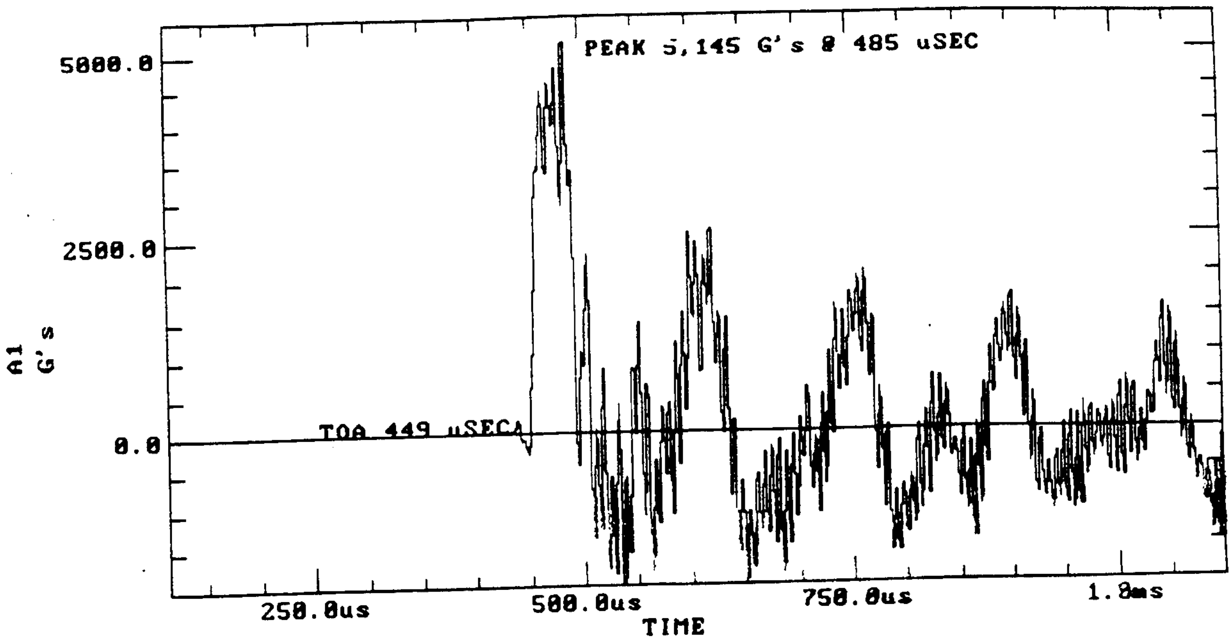

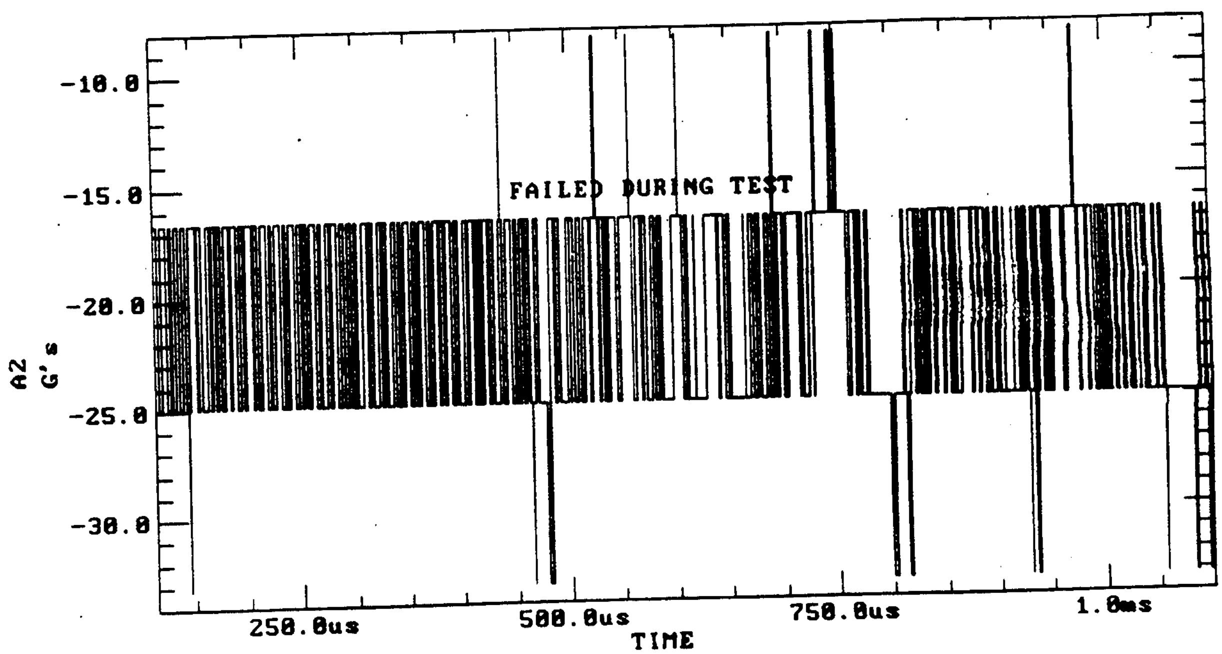

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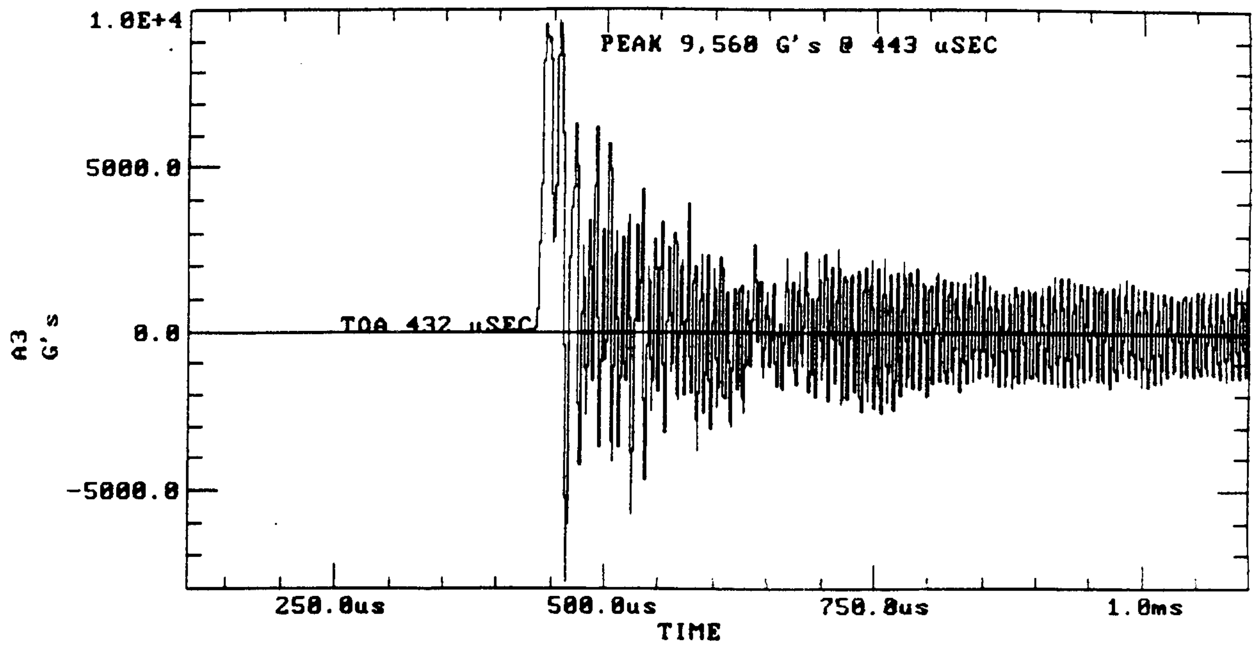
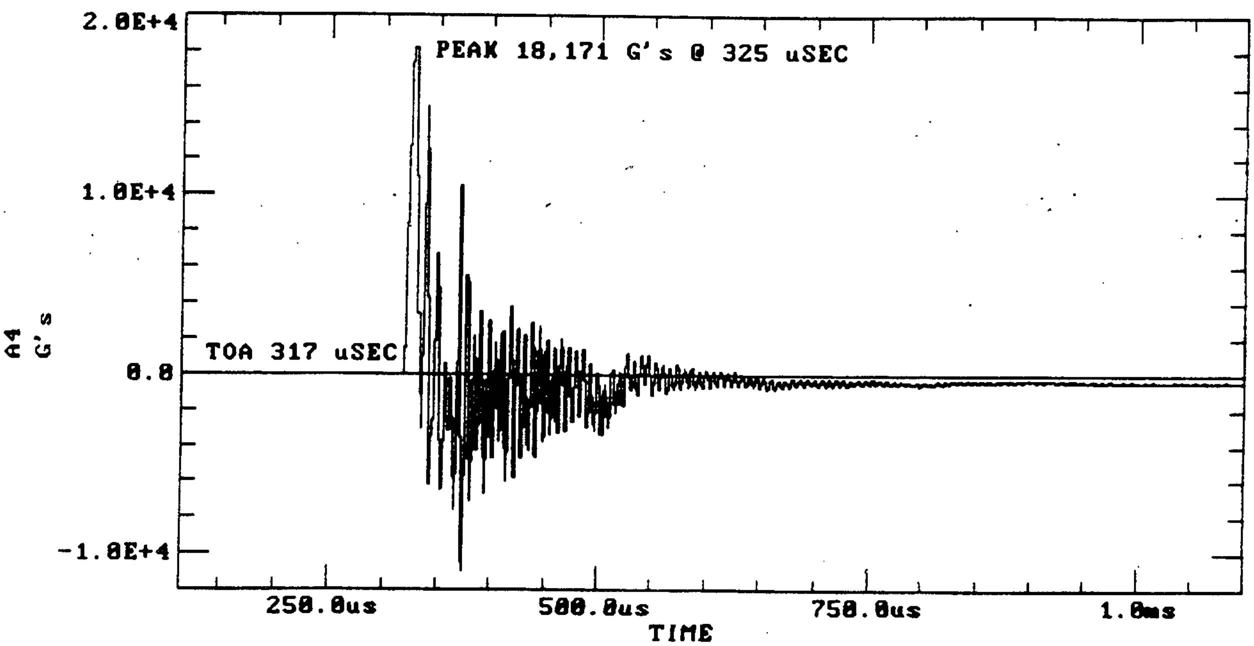
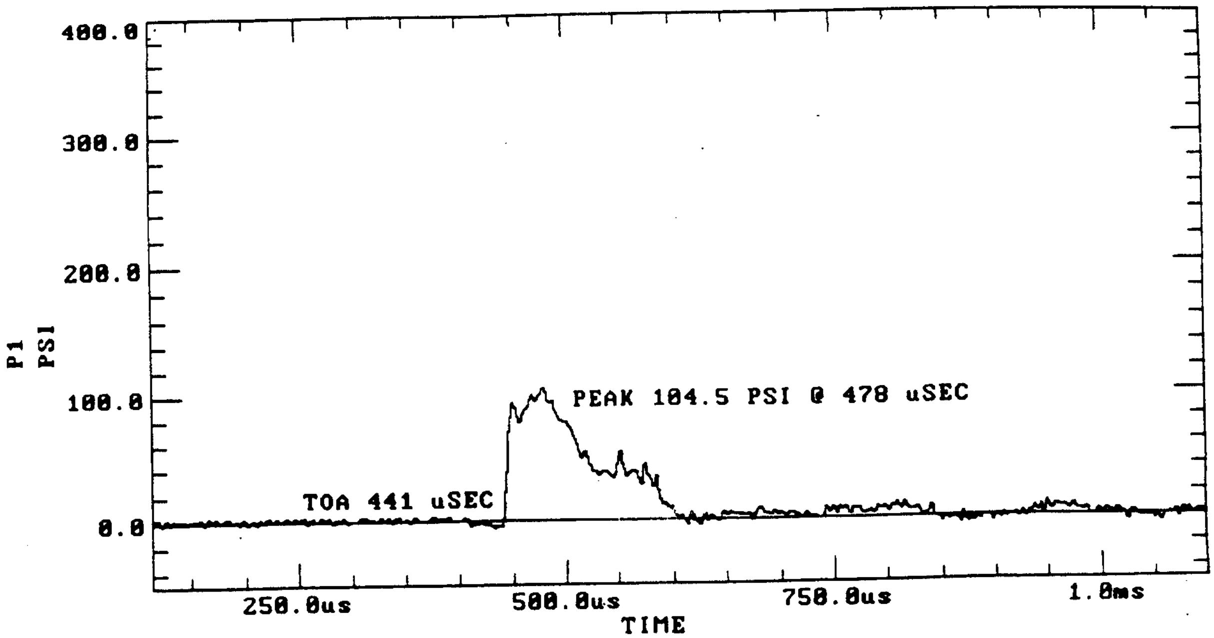
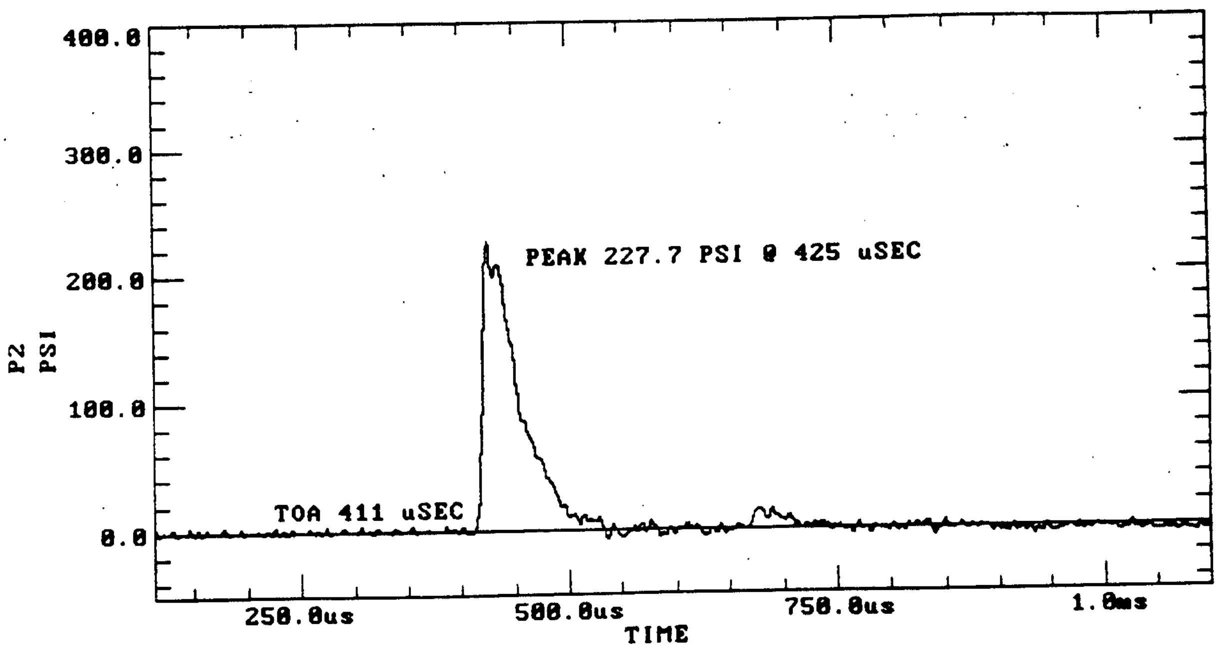




Test 12. Gage Data (continued)





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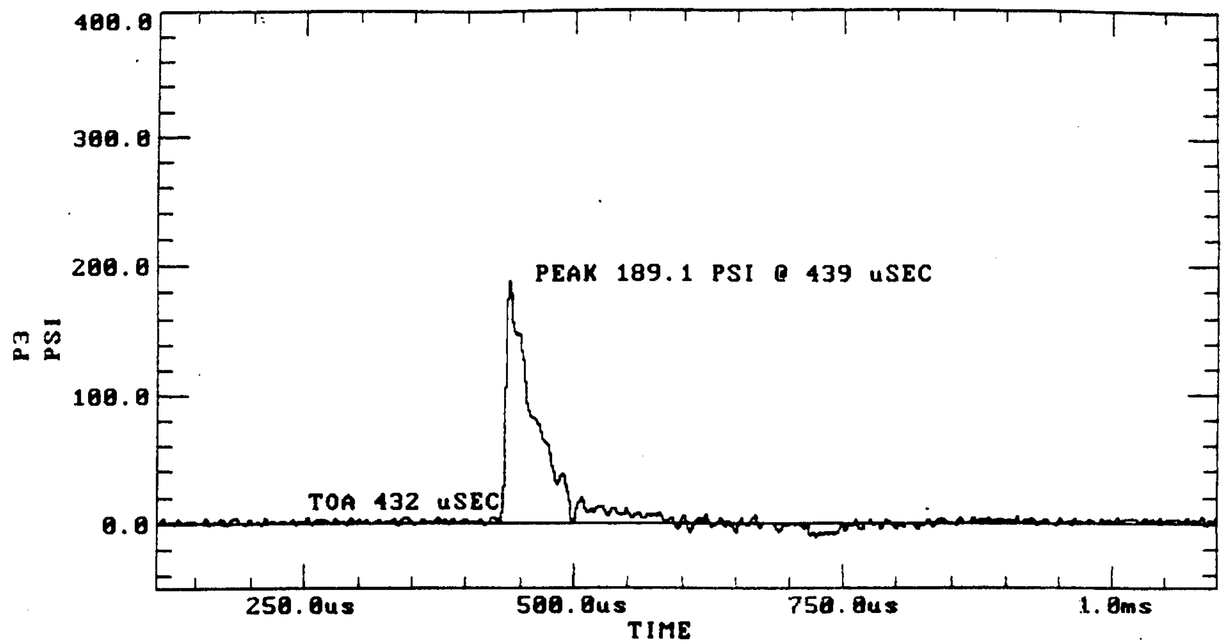

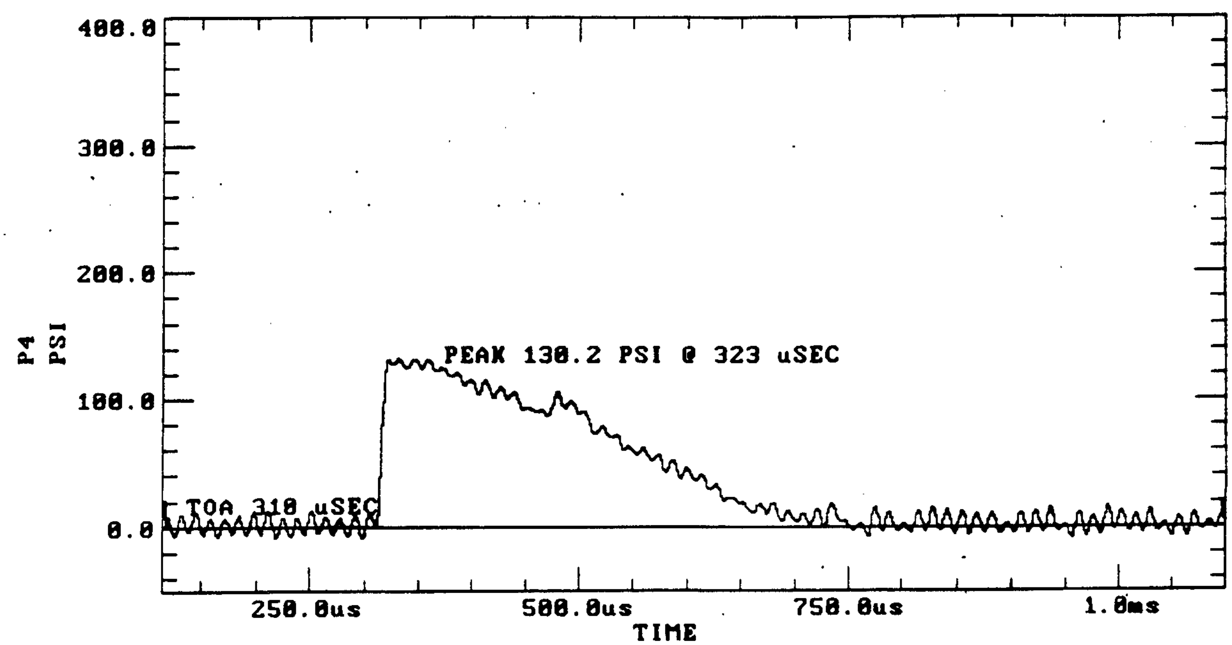

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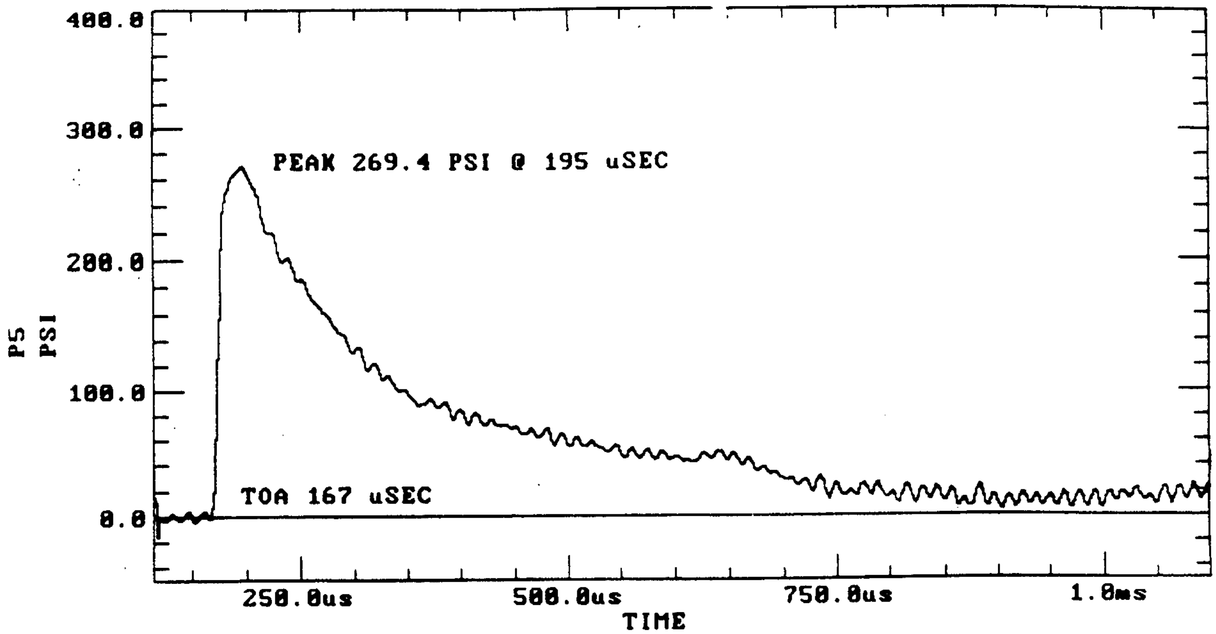

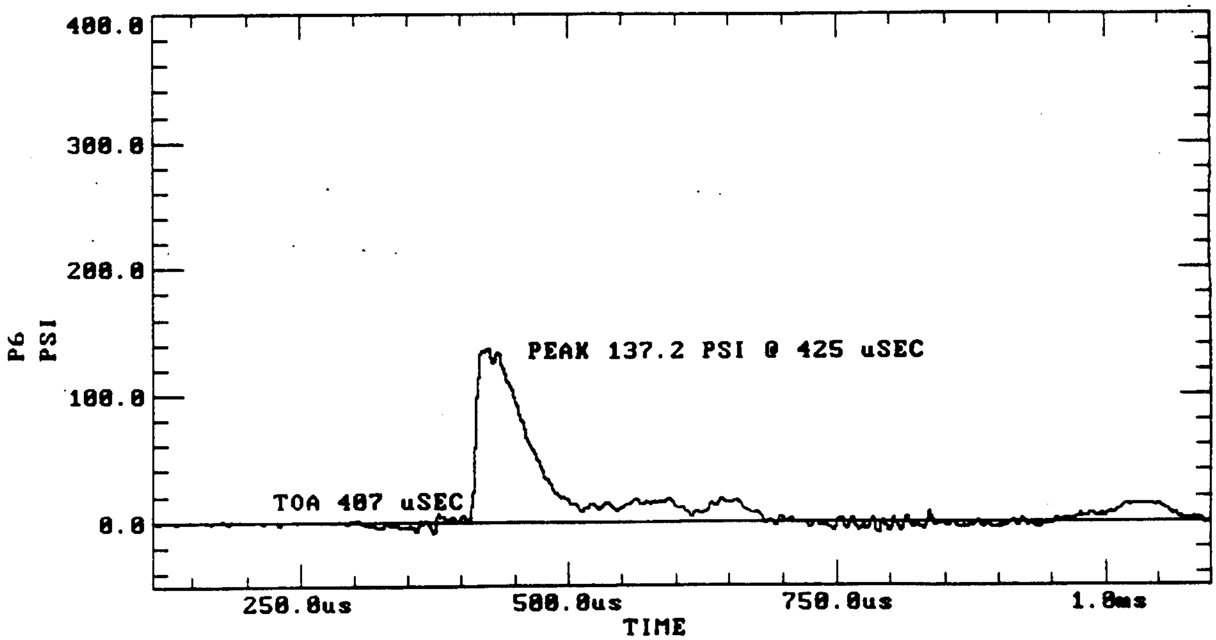

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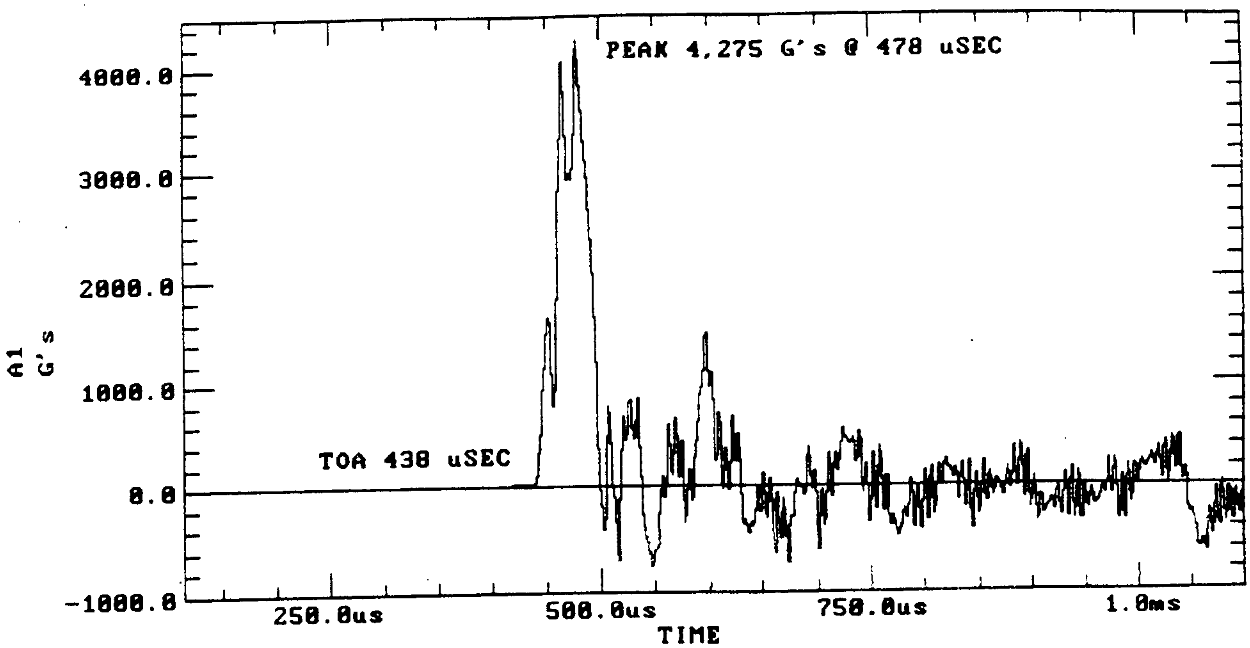

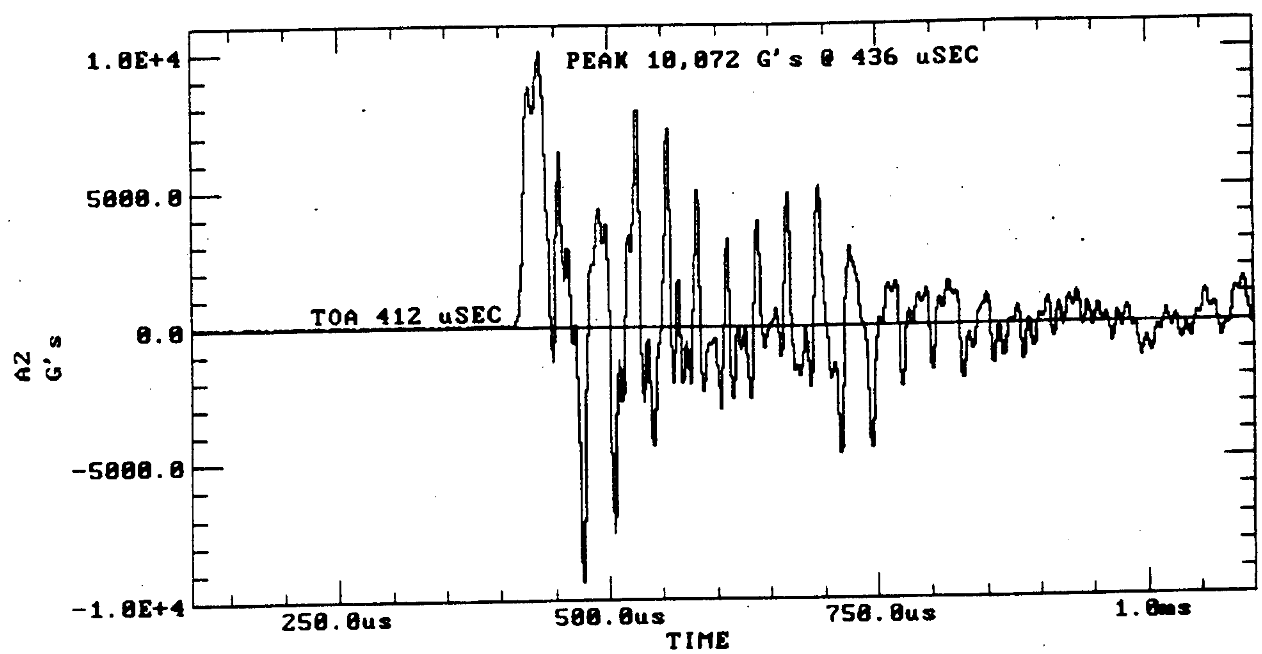

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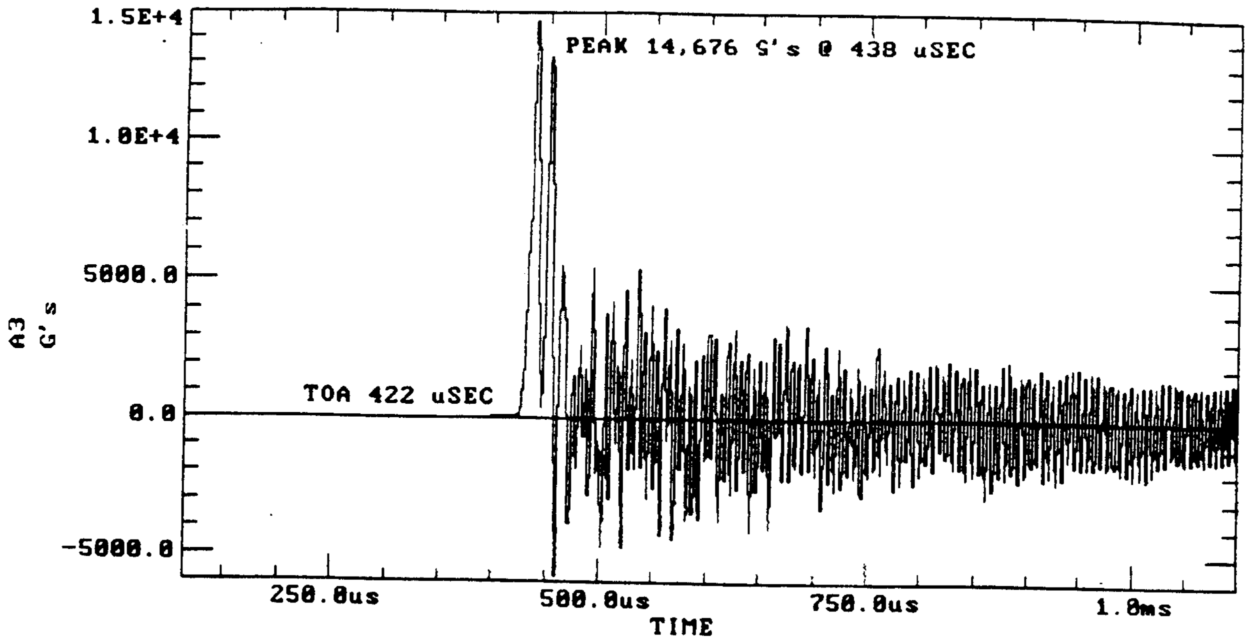

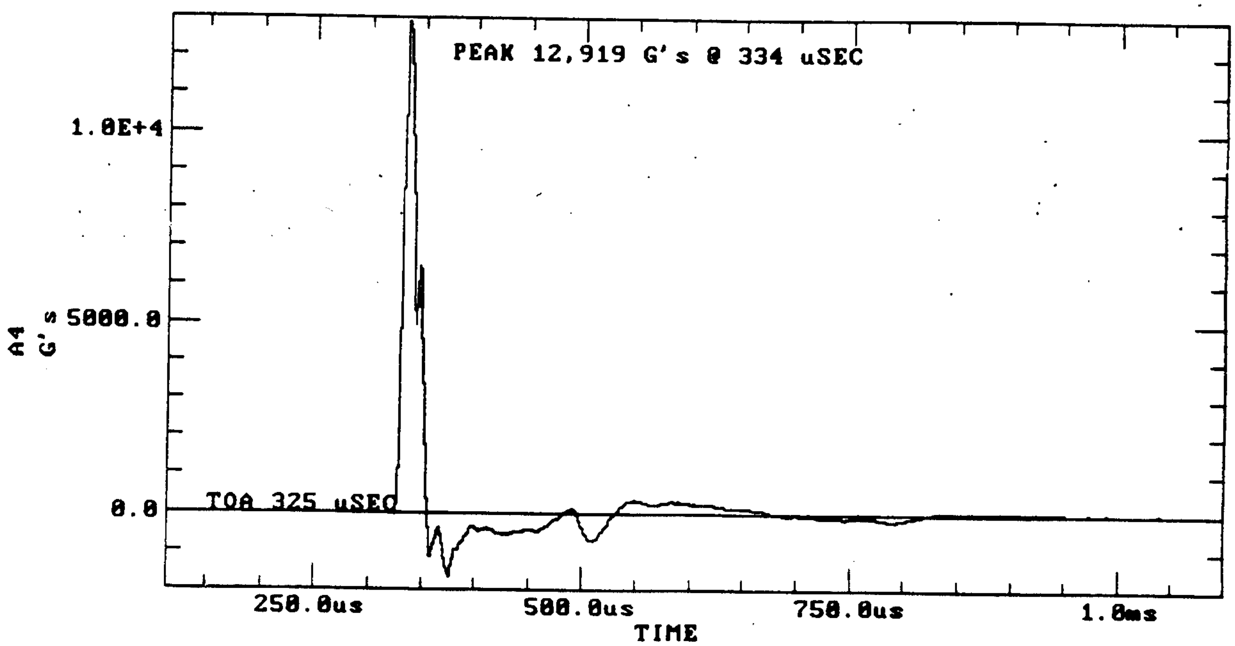

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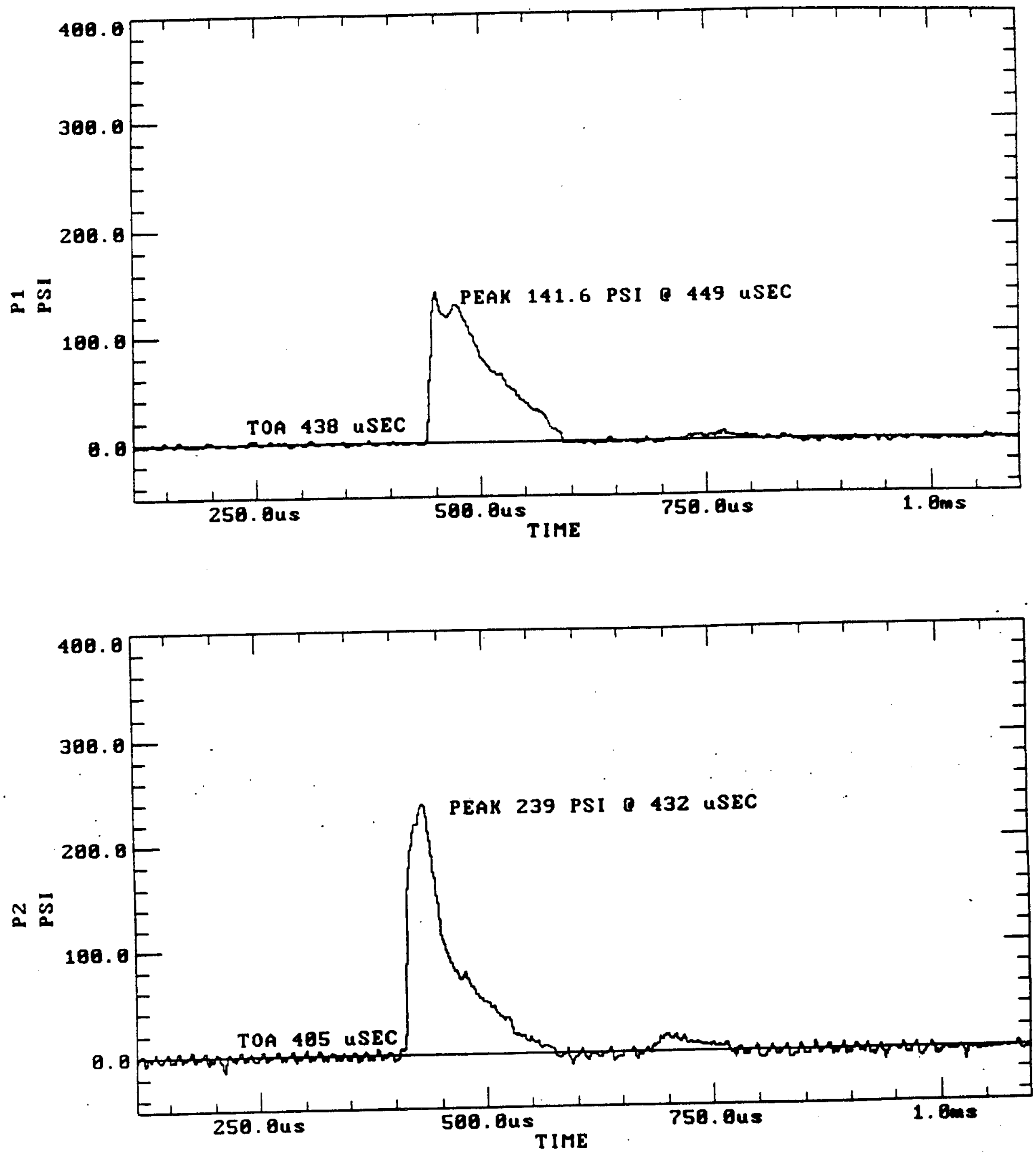

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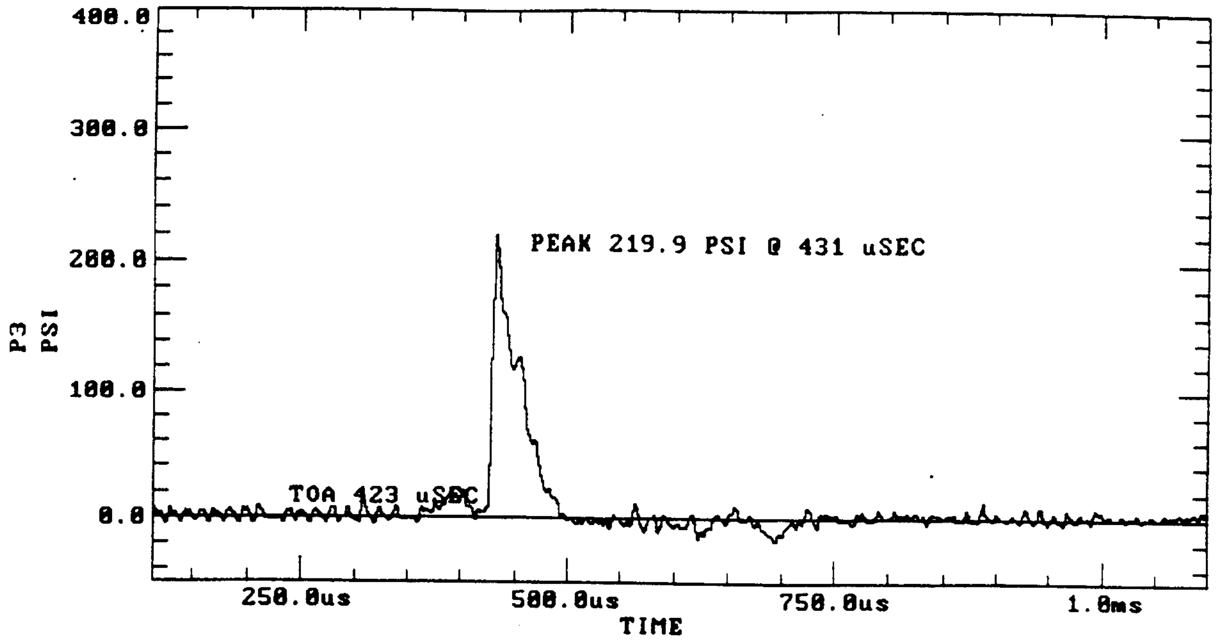

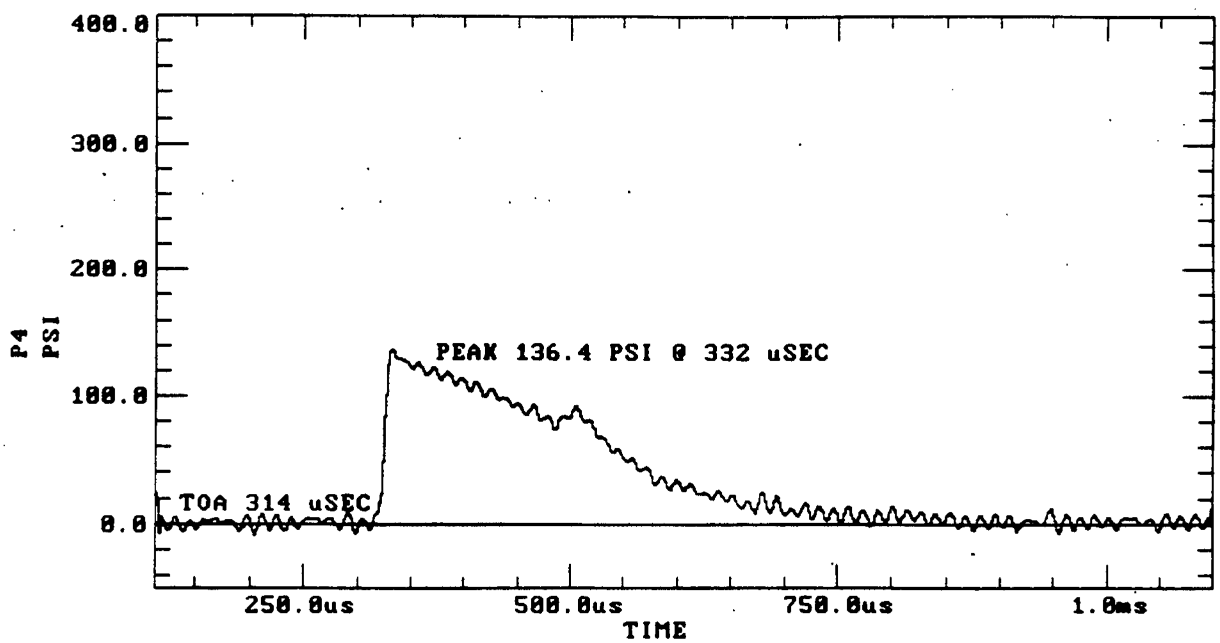

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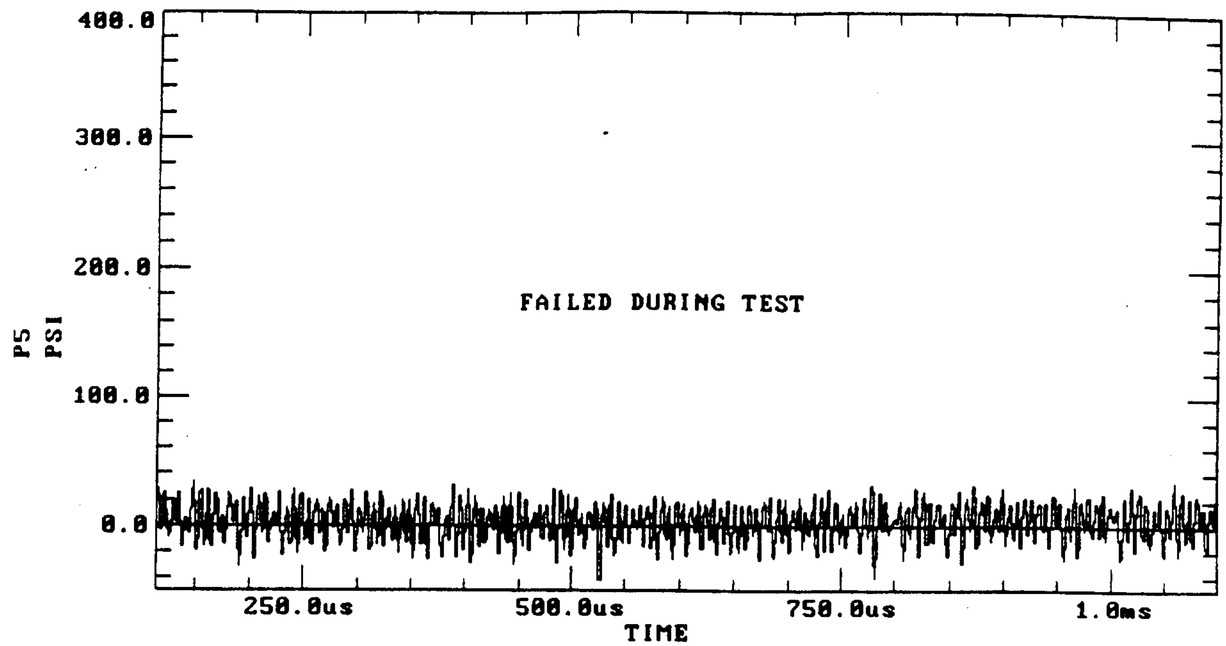

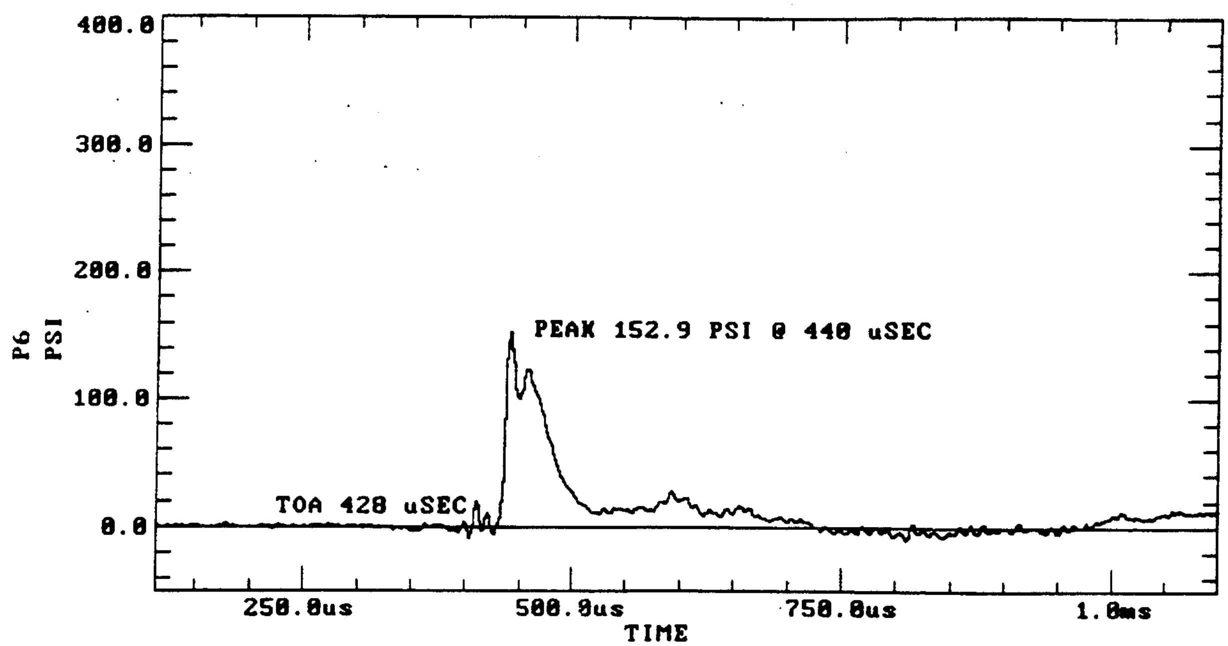

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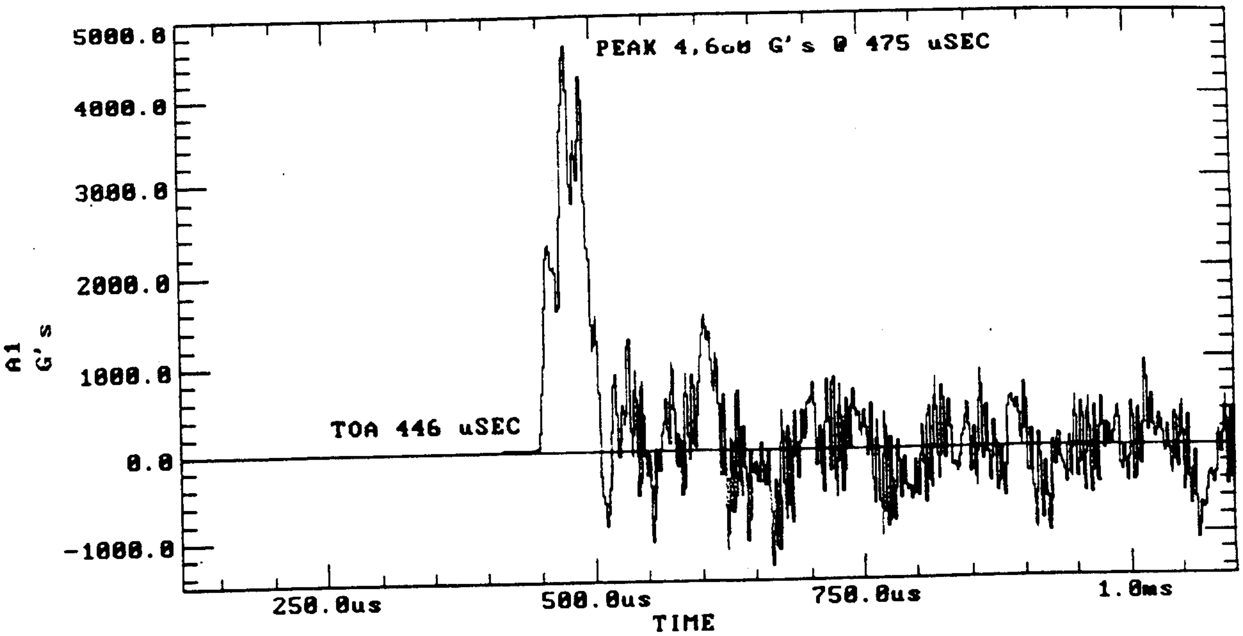

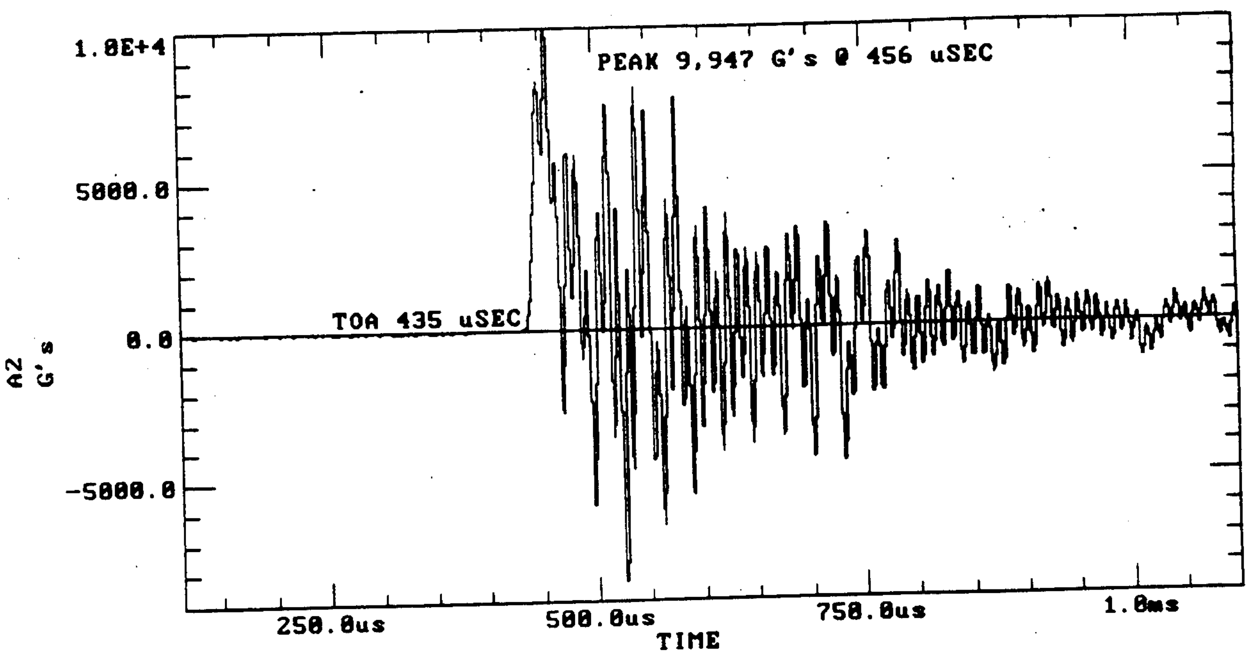

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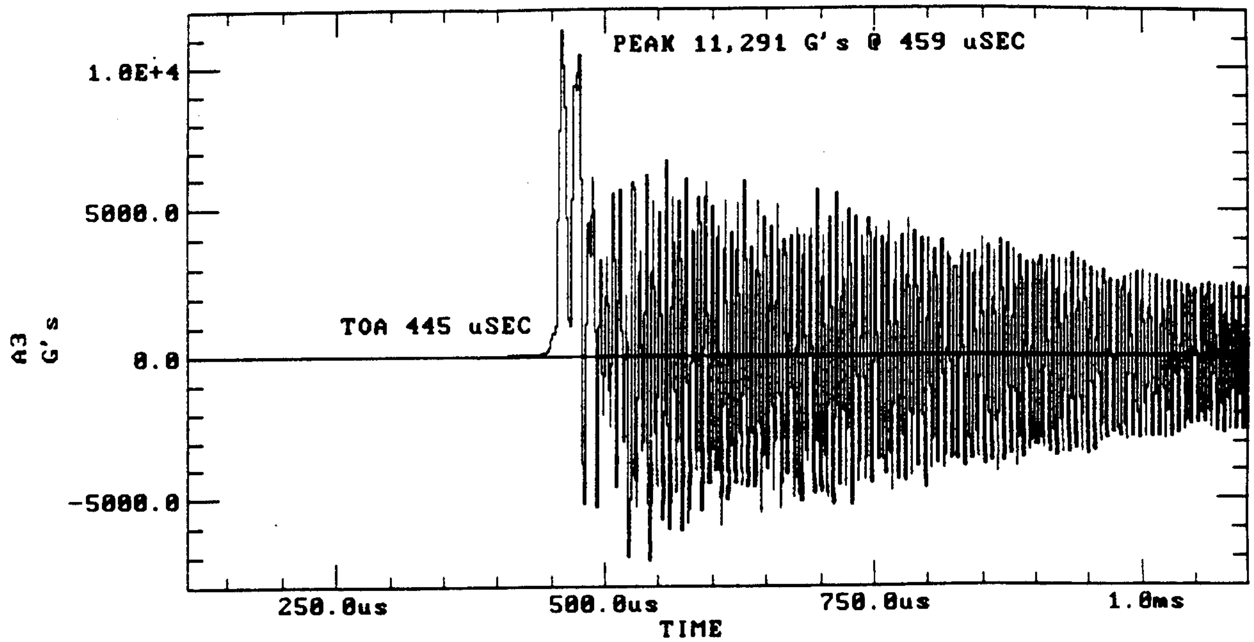

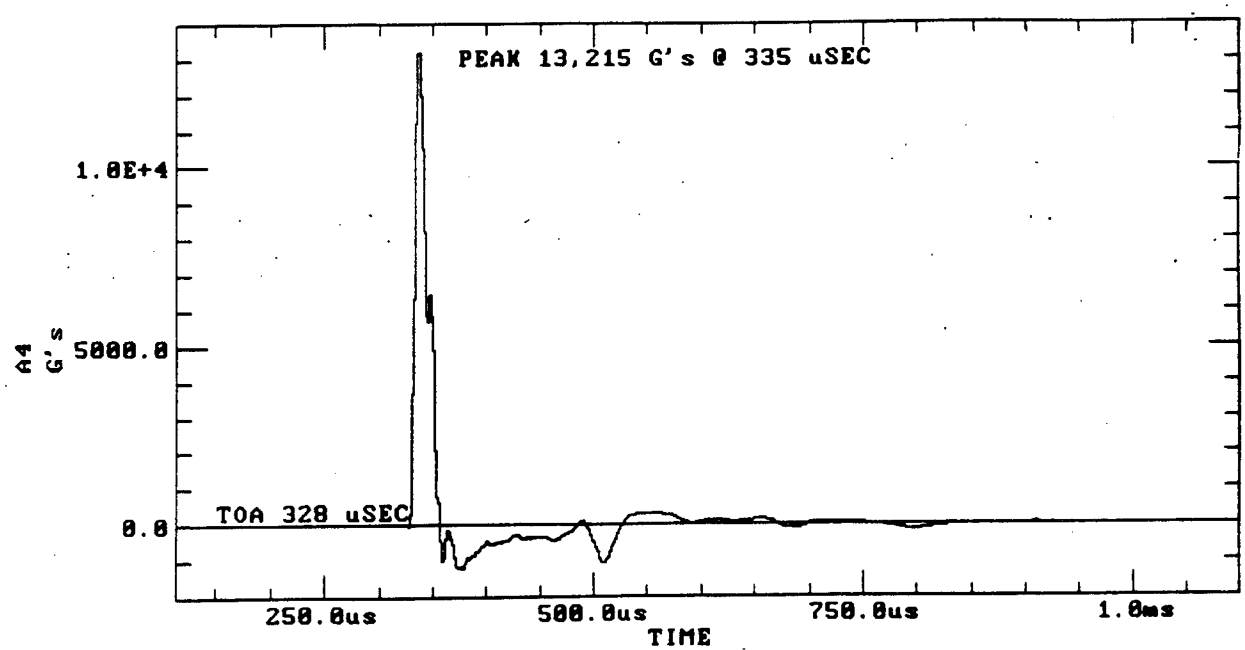

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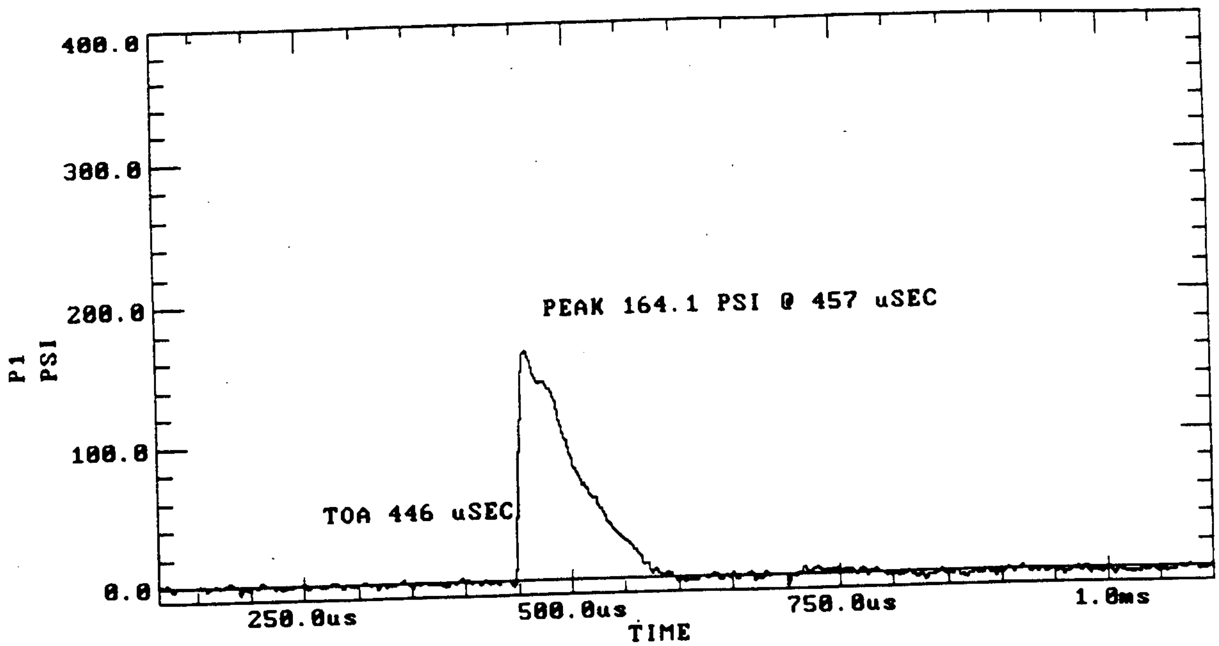

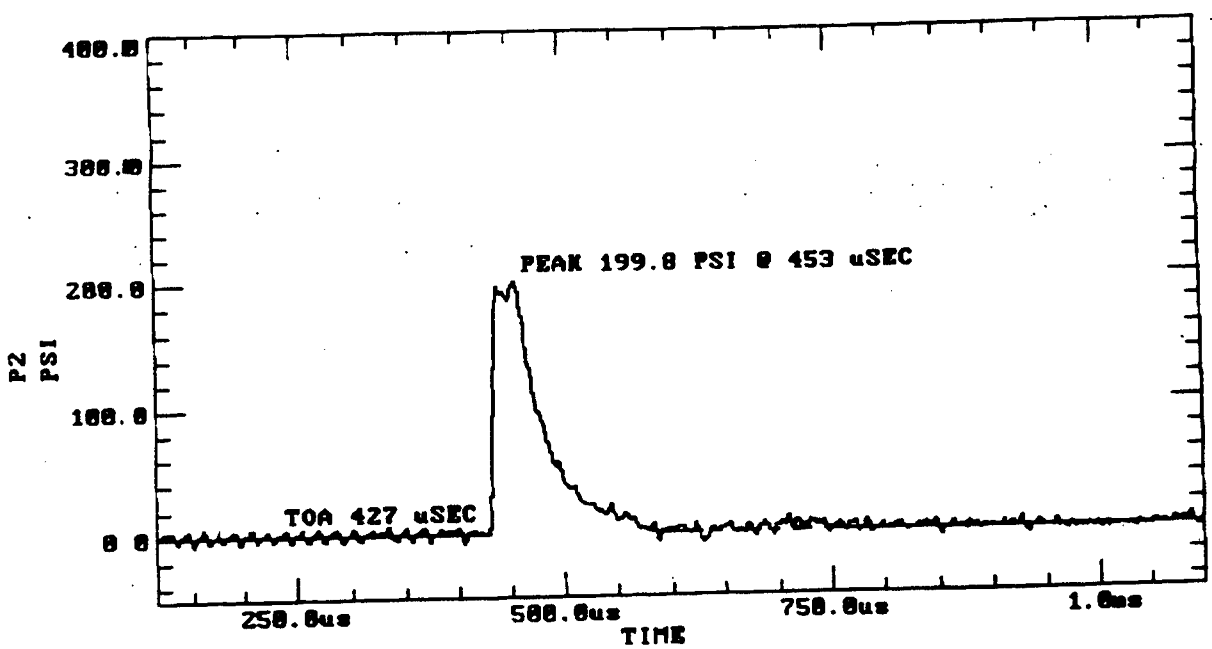

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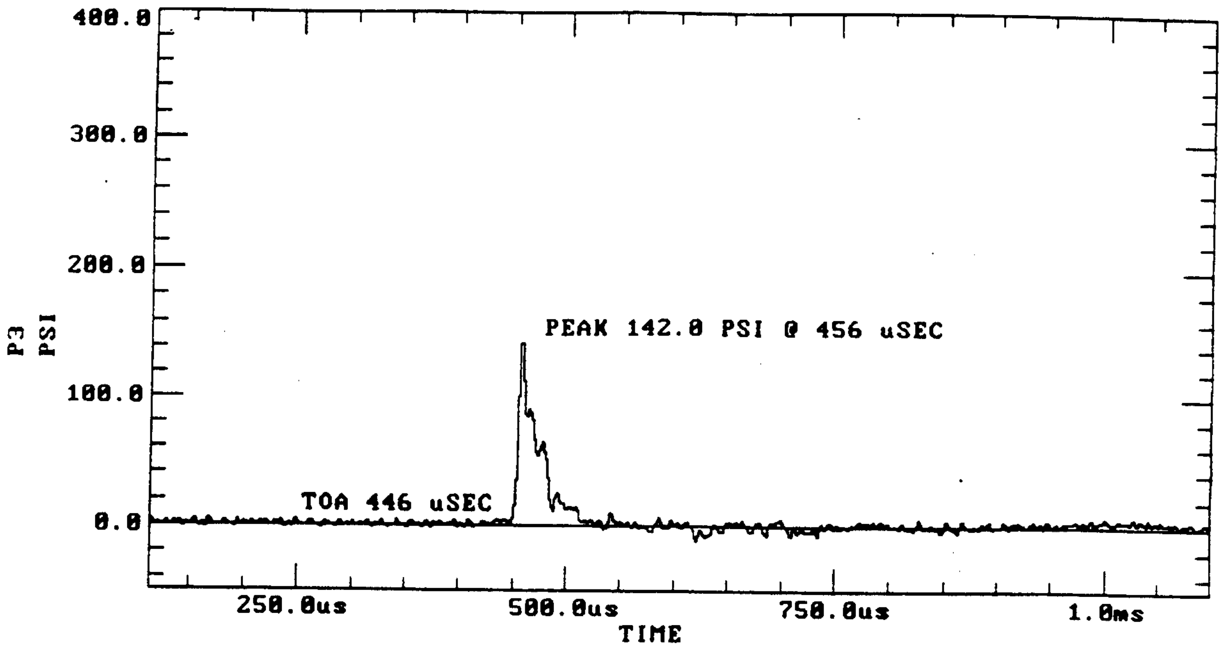

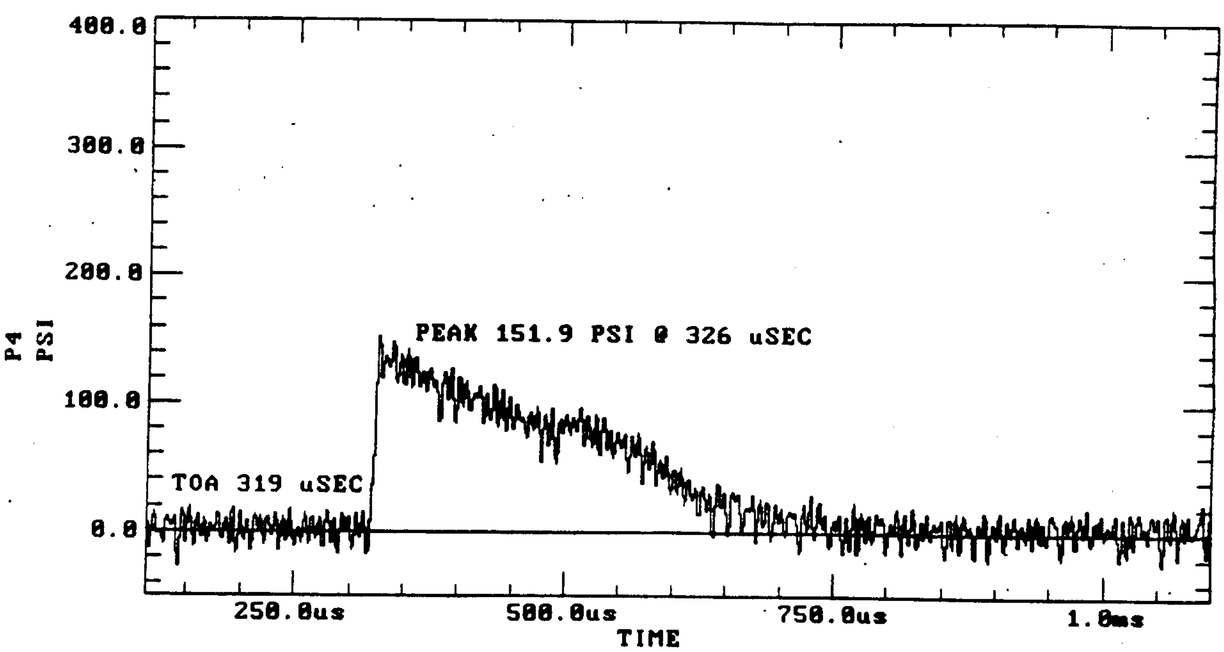

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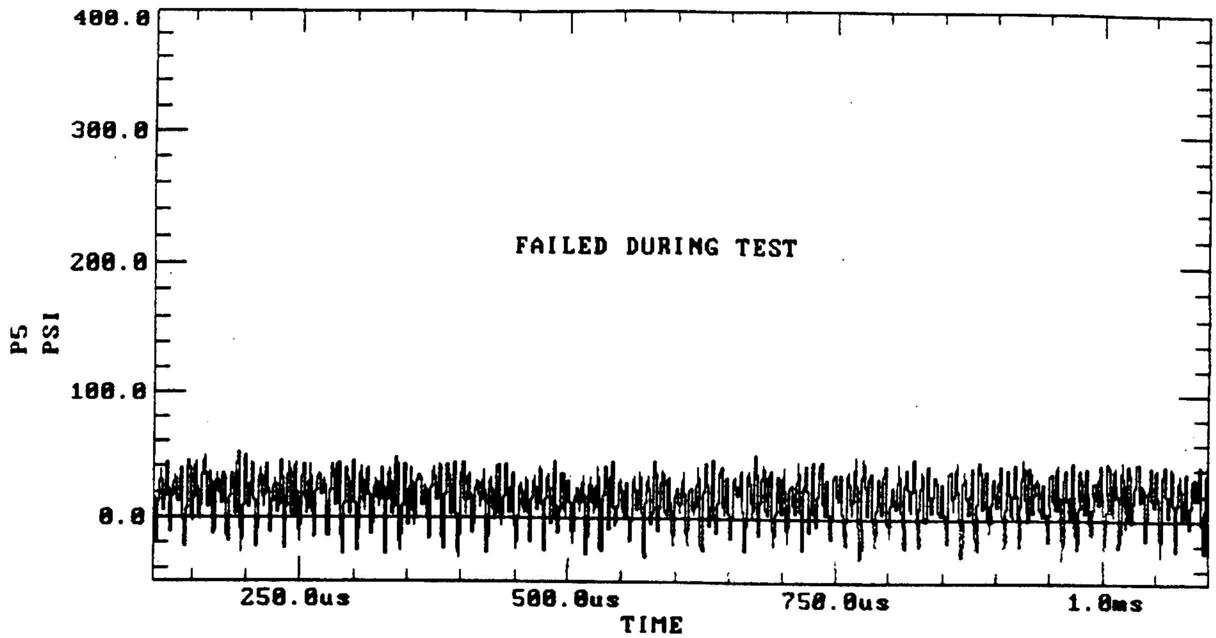

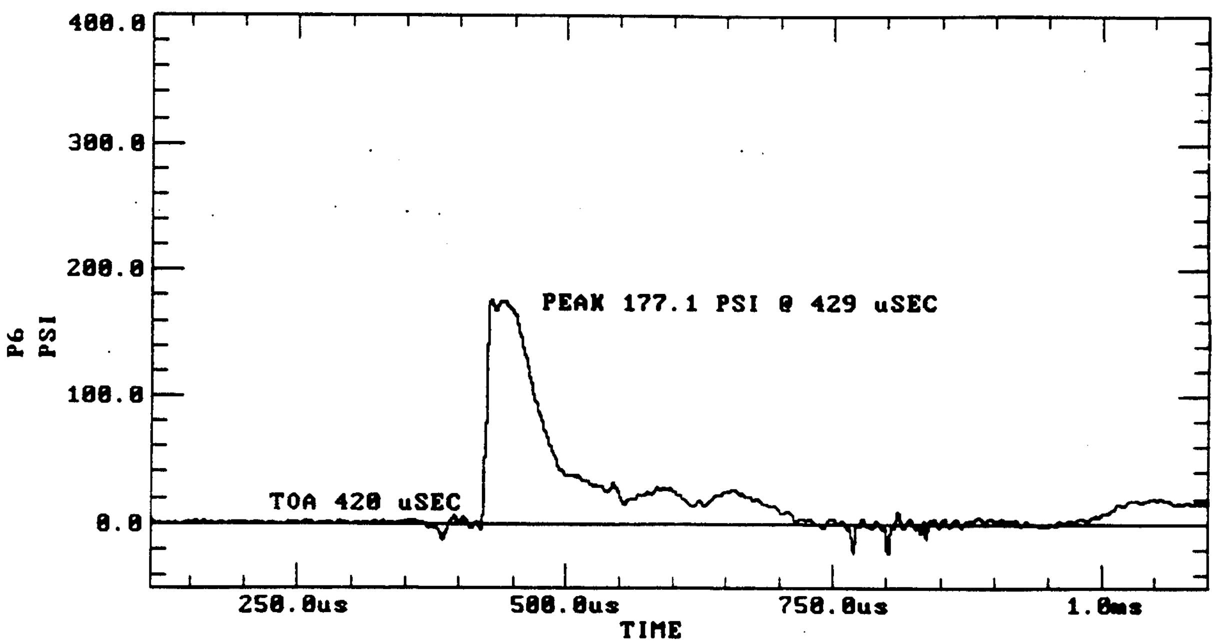

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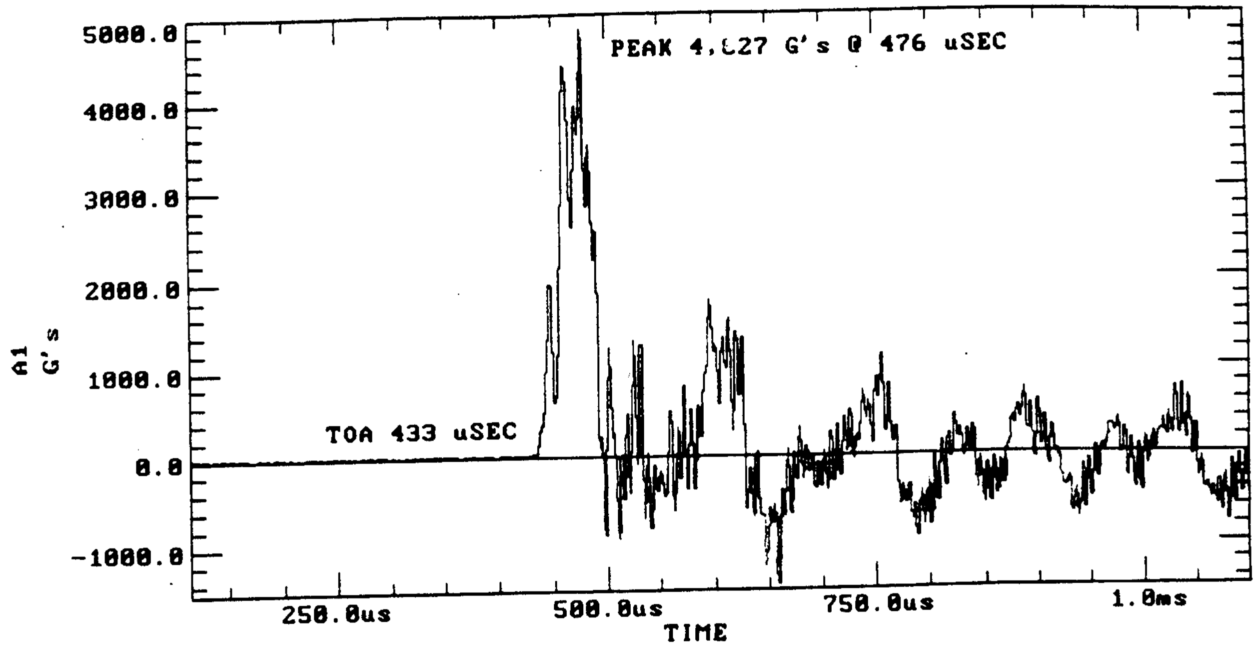

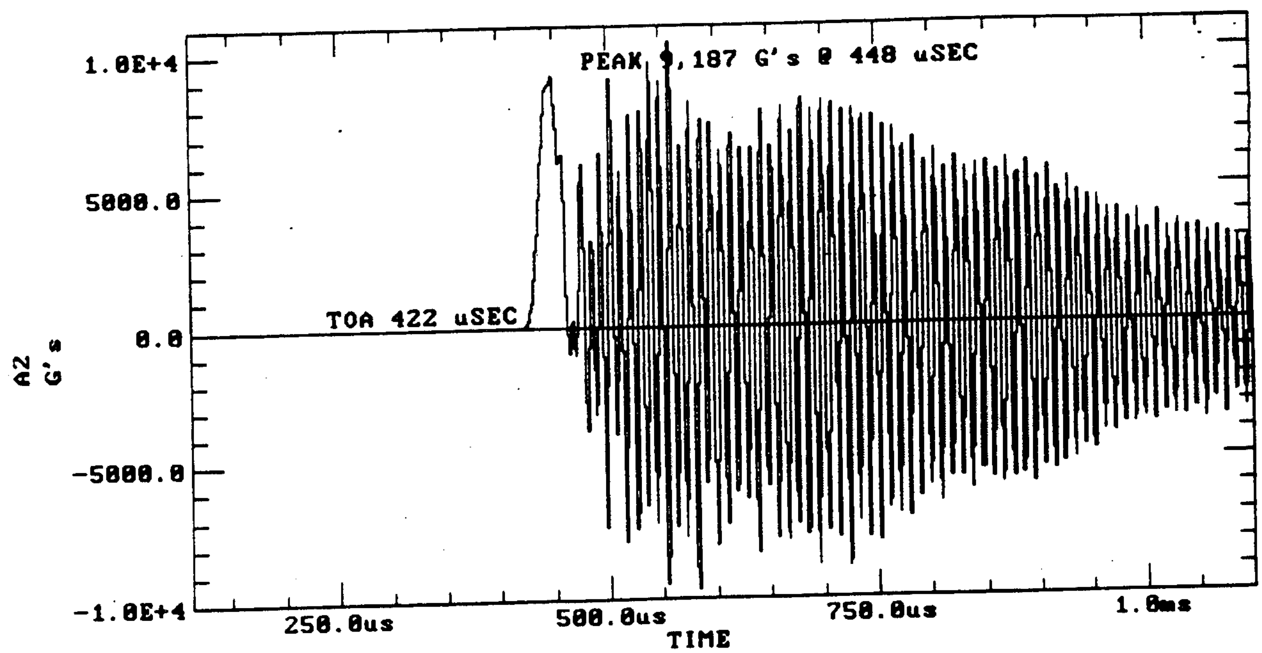

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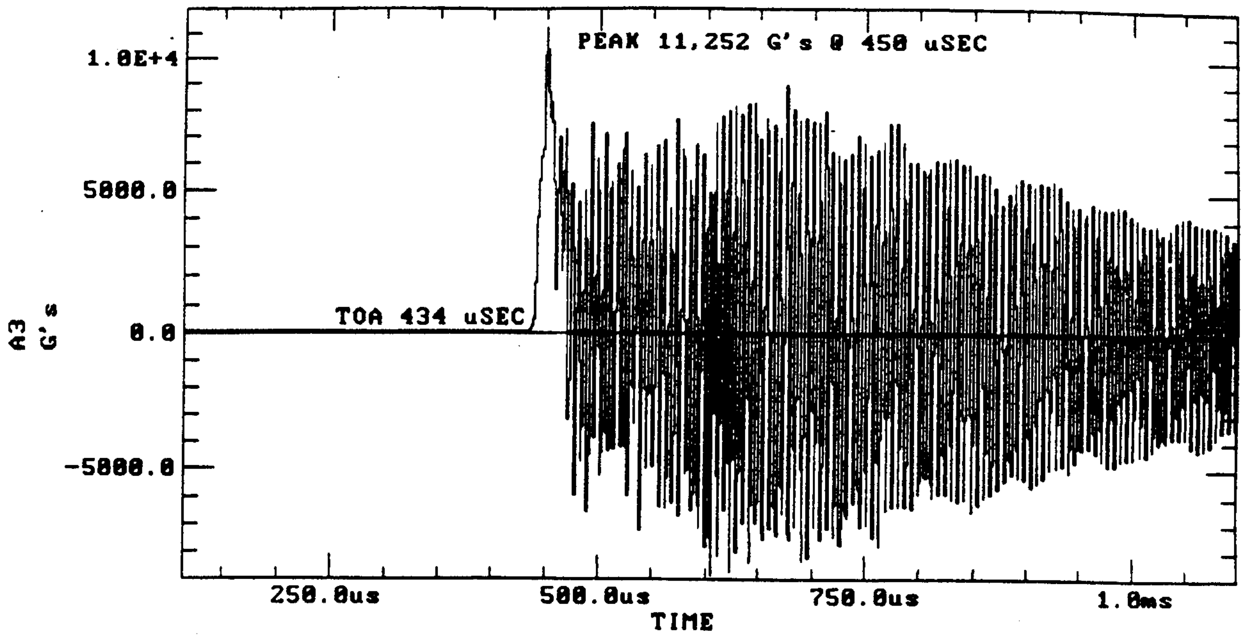

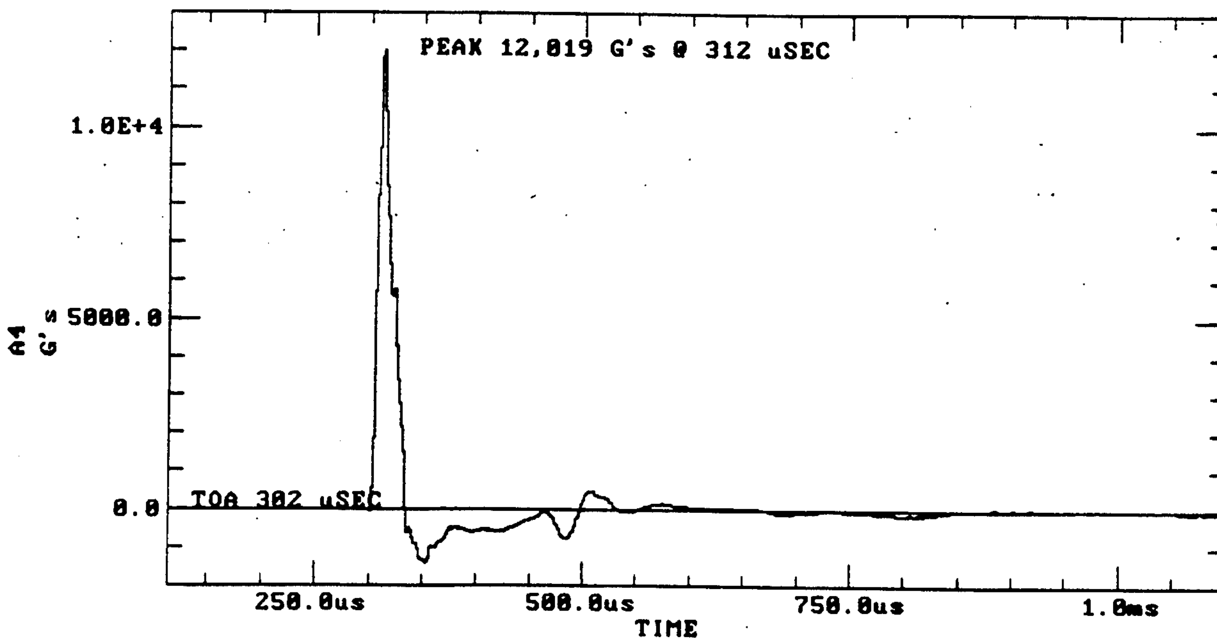

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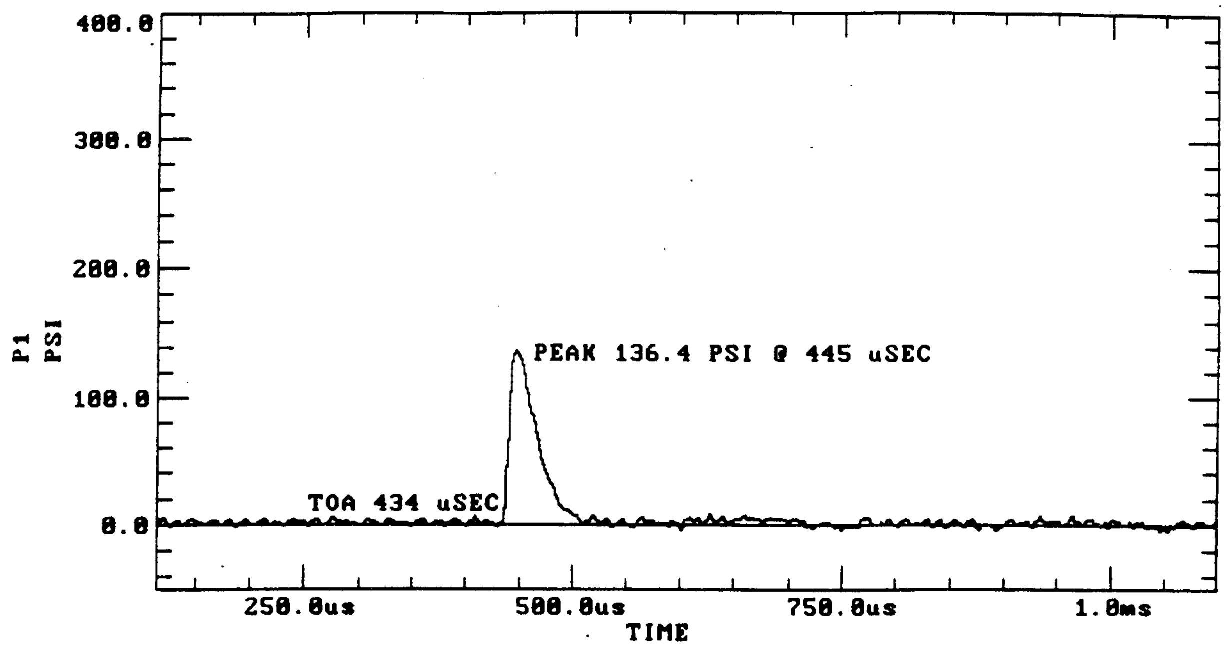

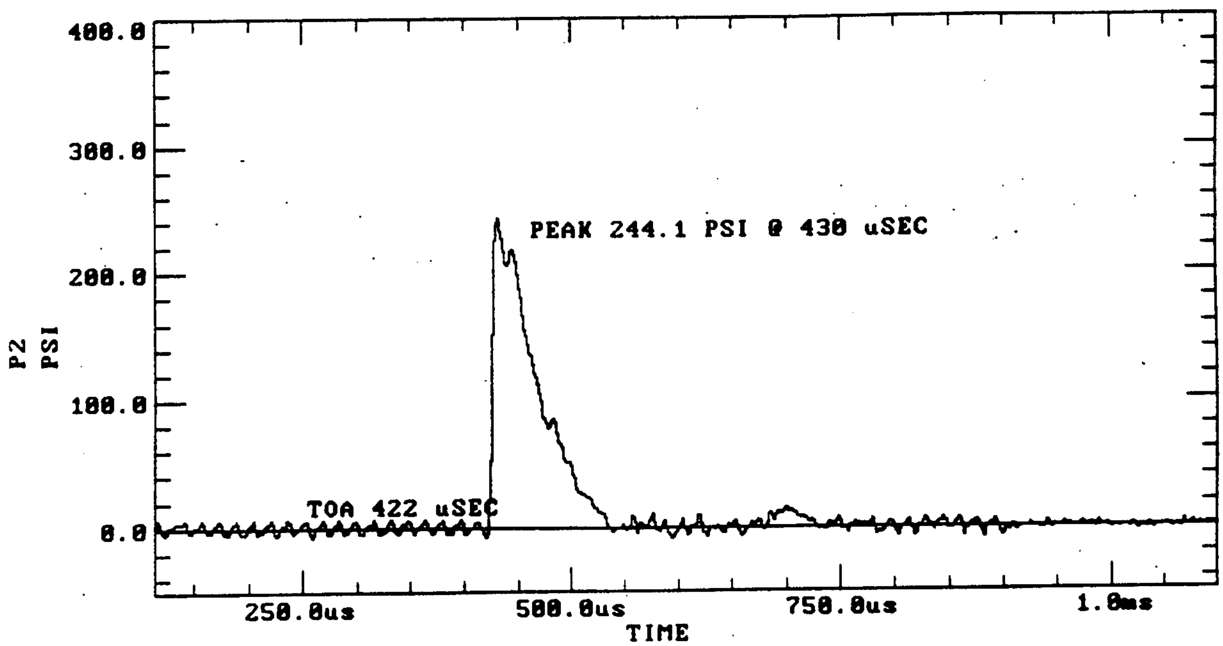

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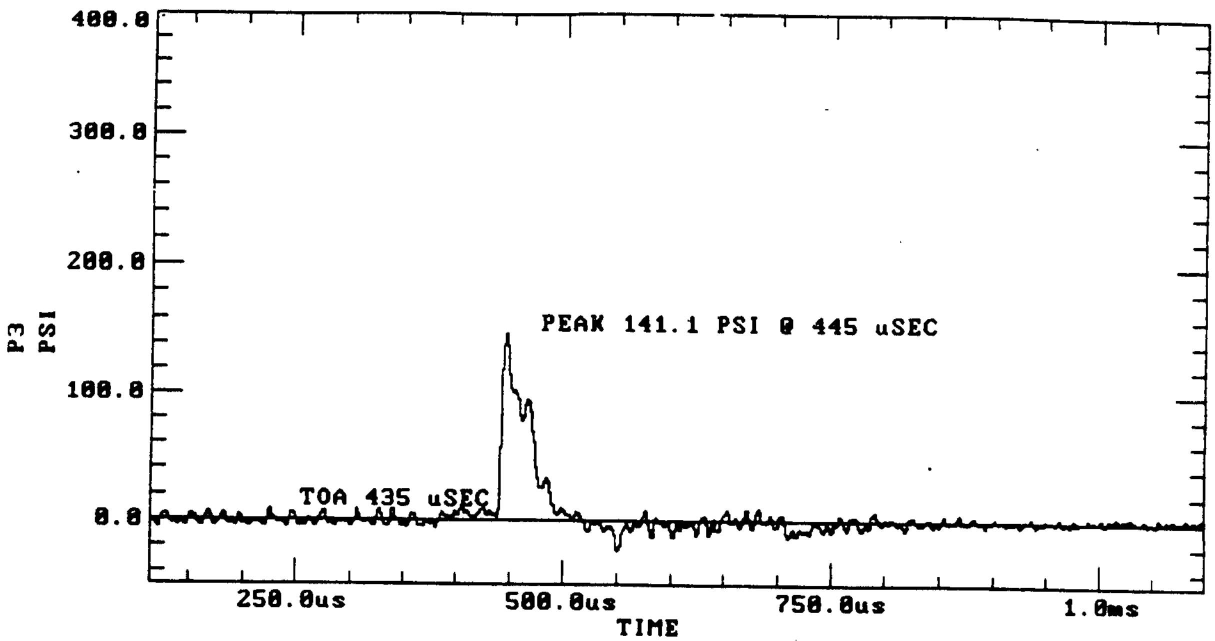

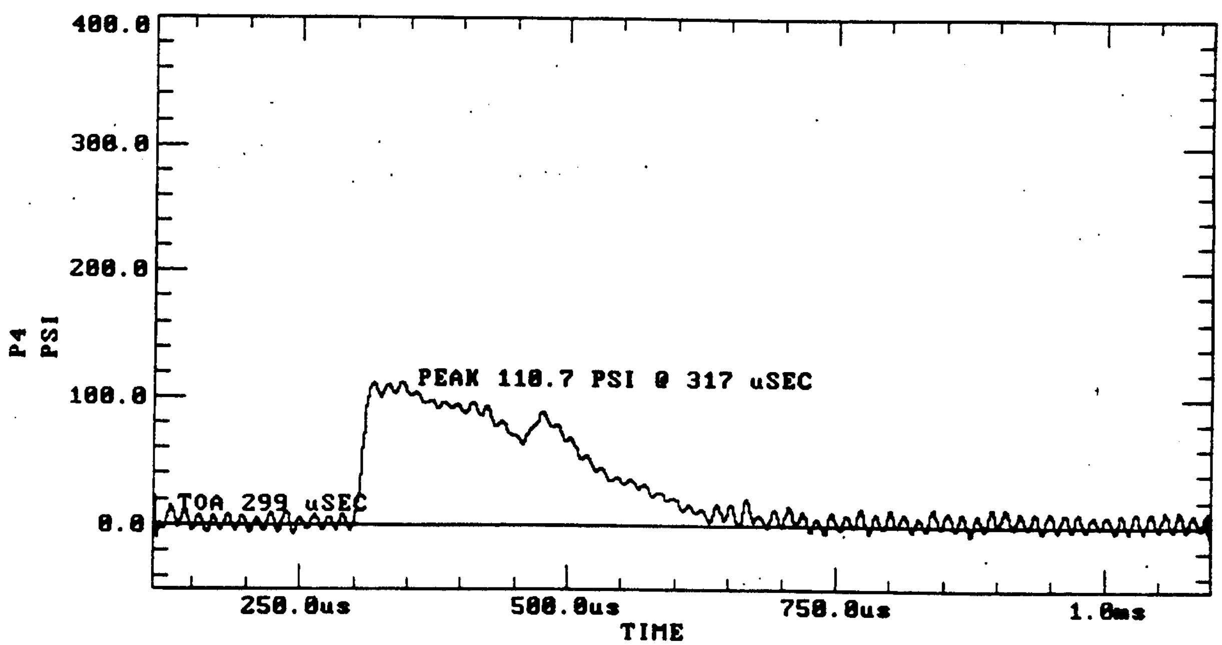

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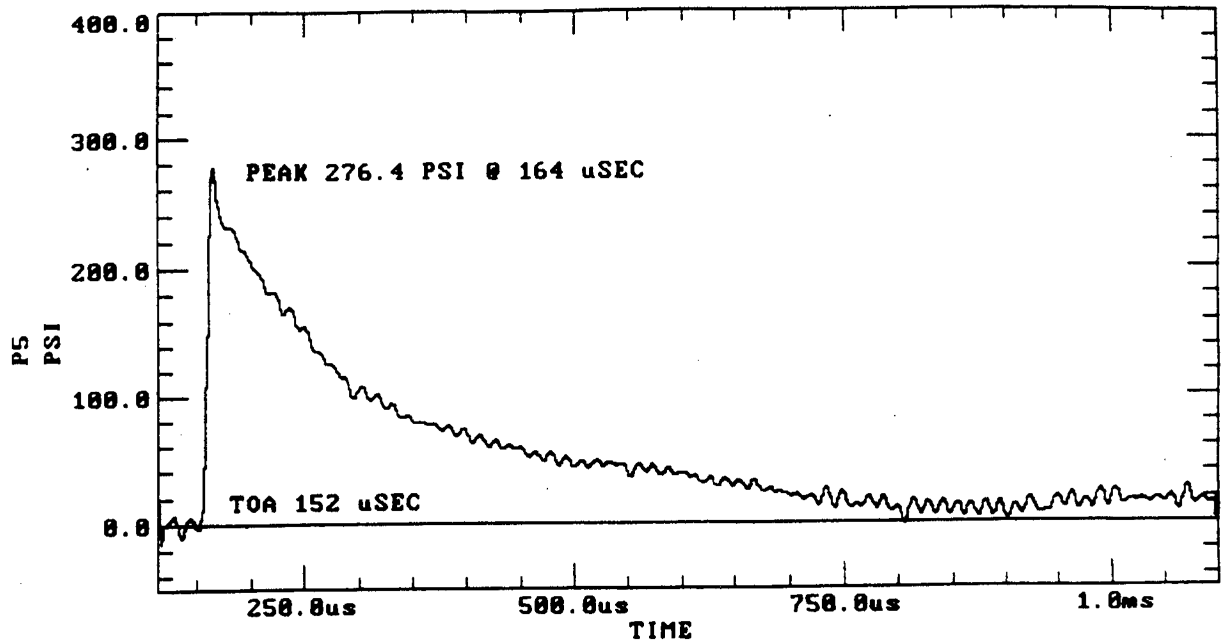

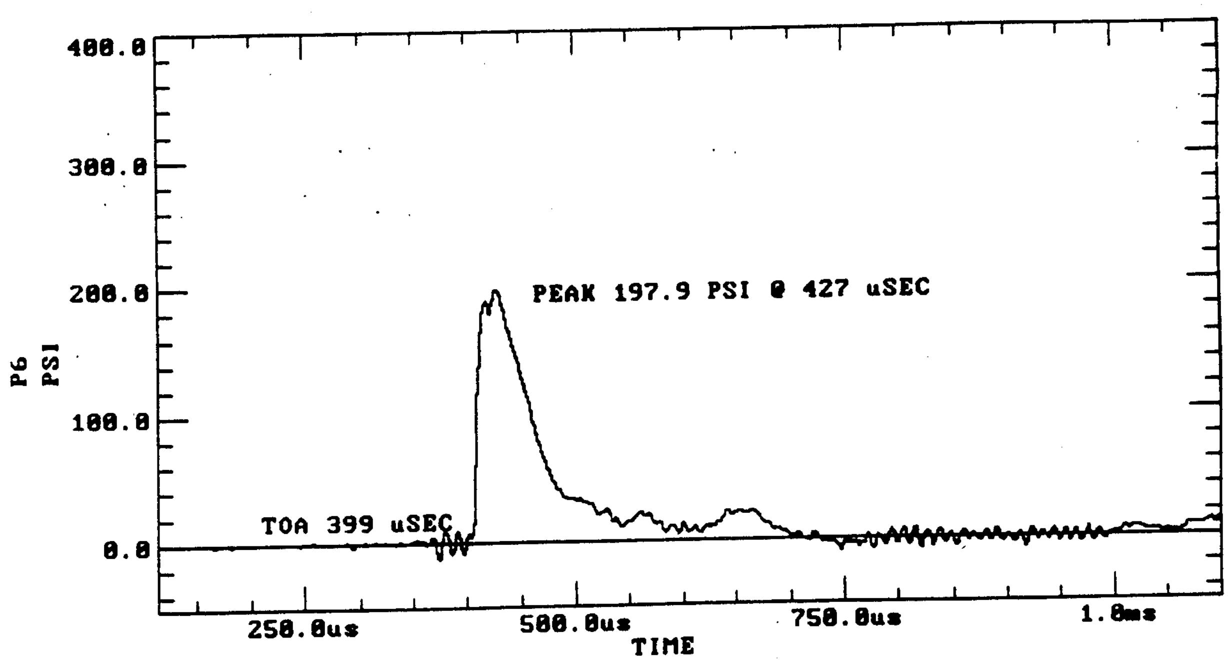

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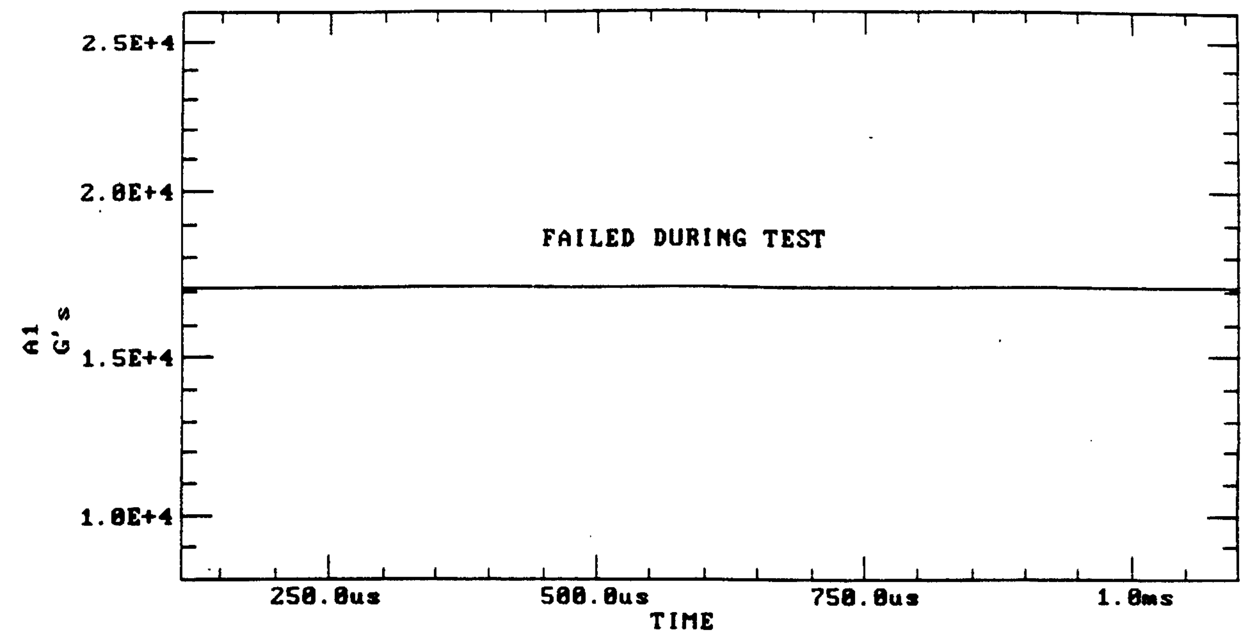

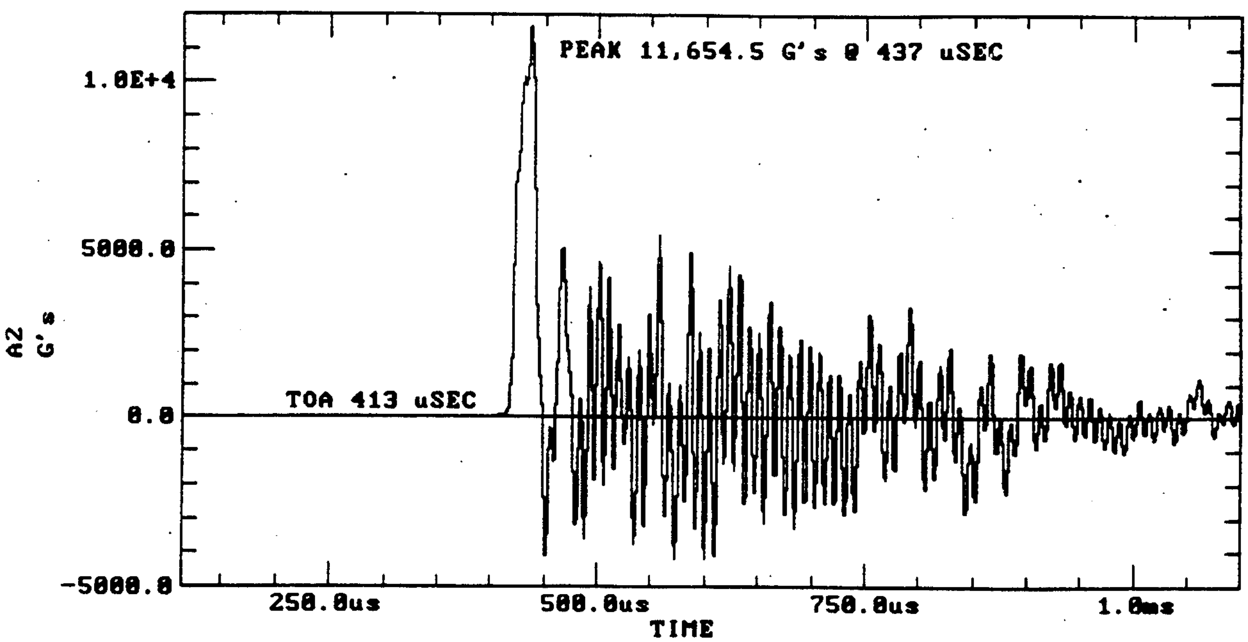

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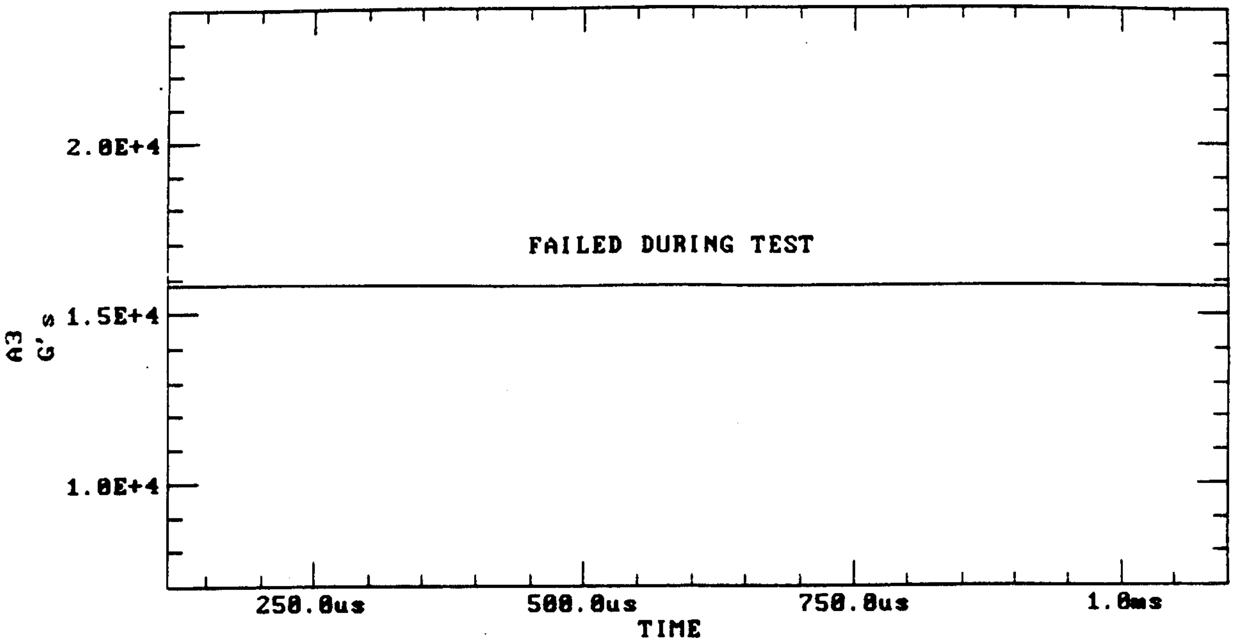

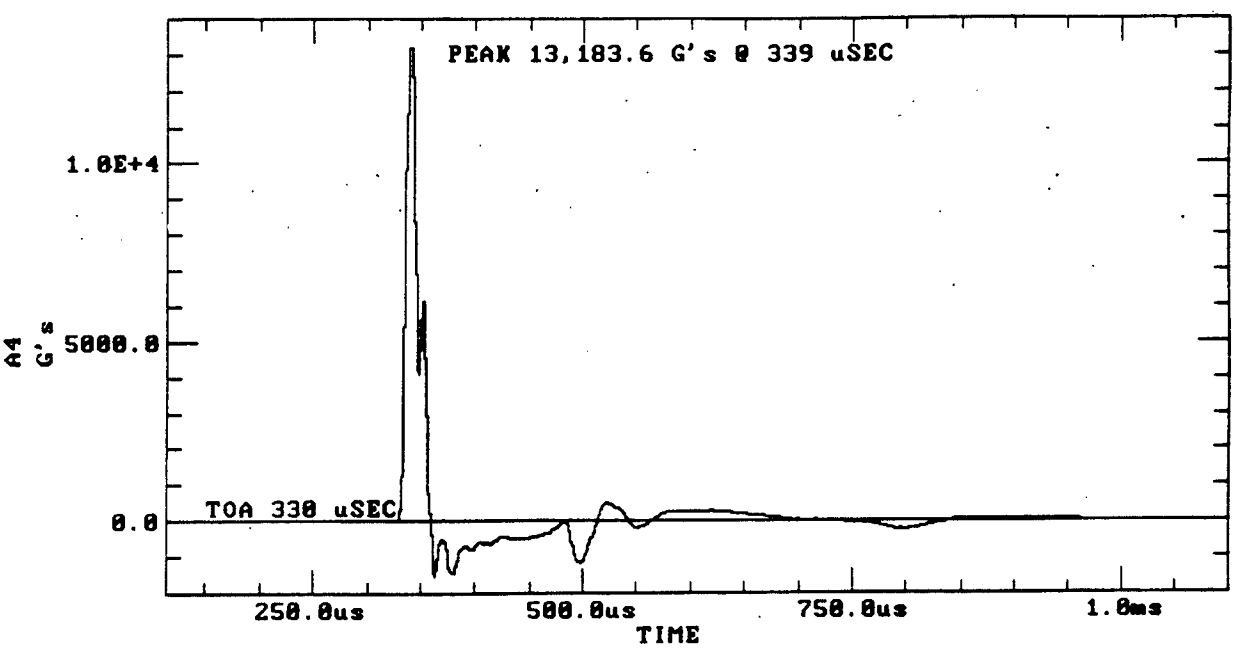

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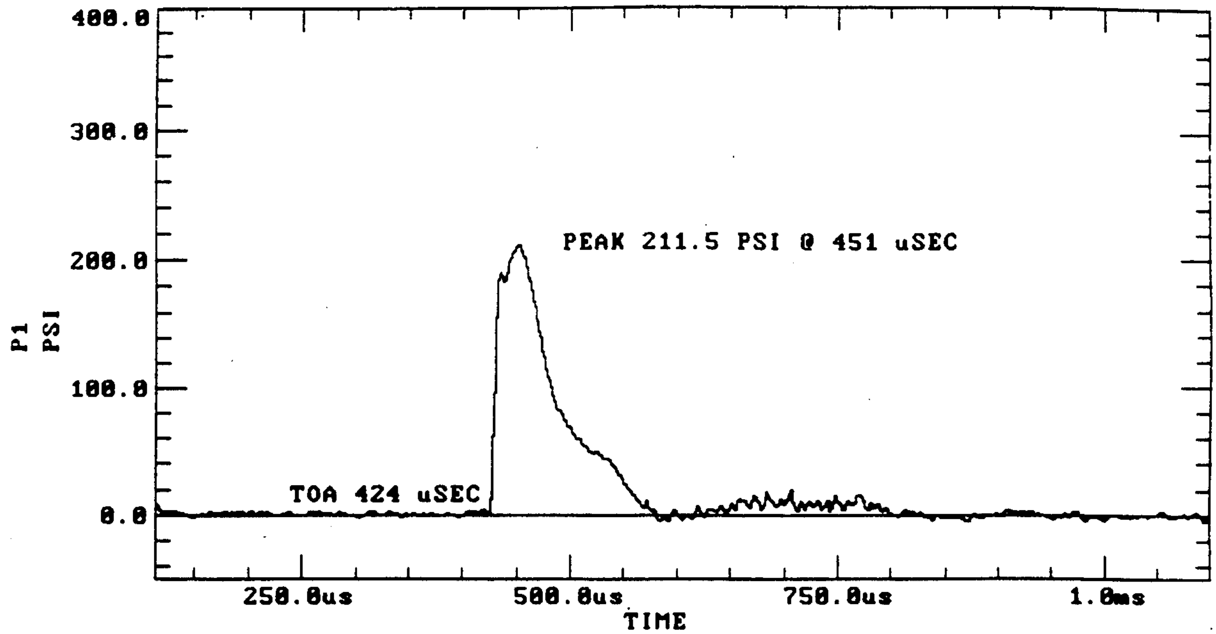

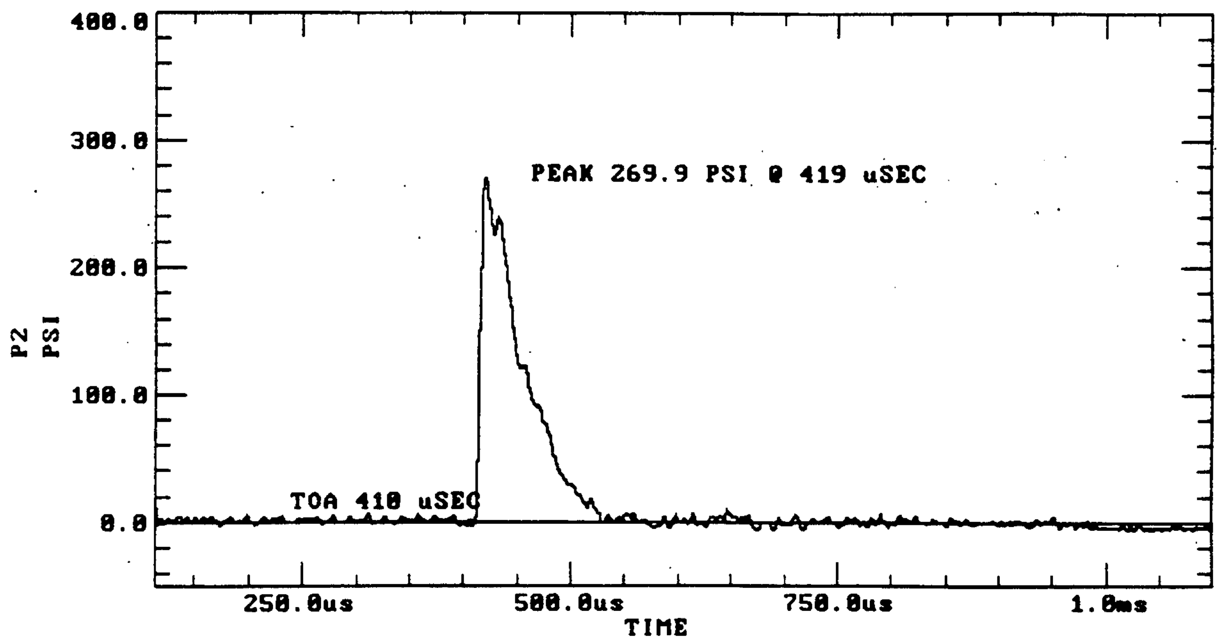

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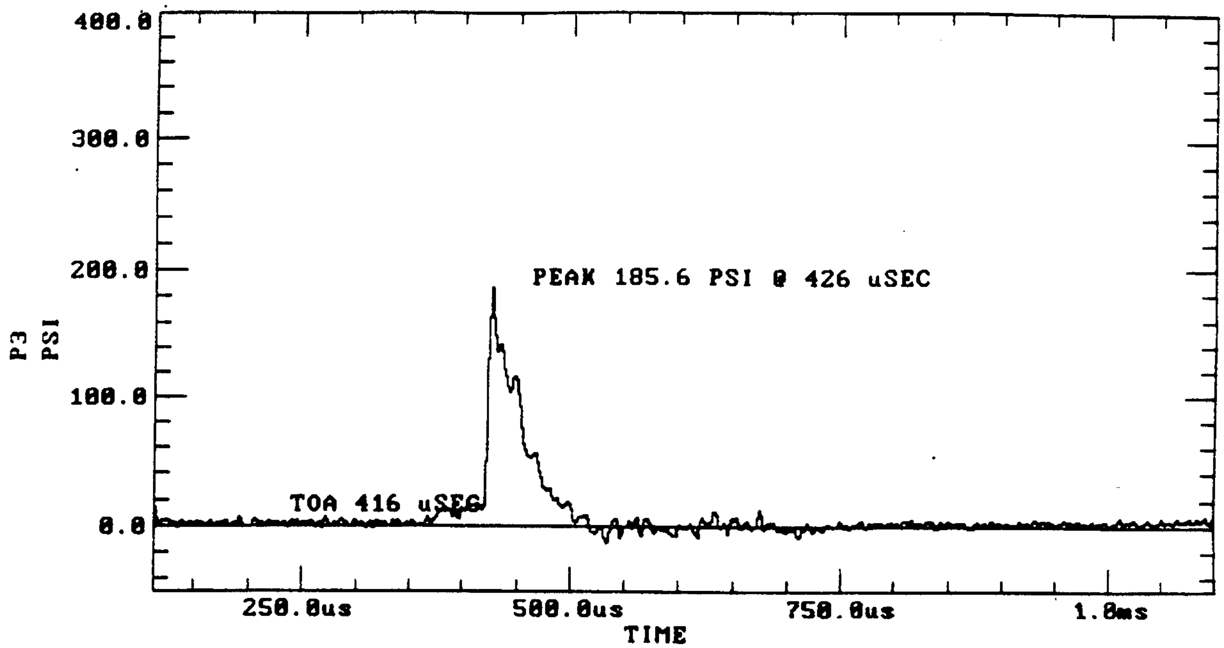

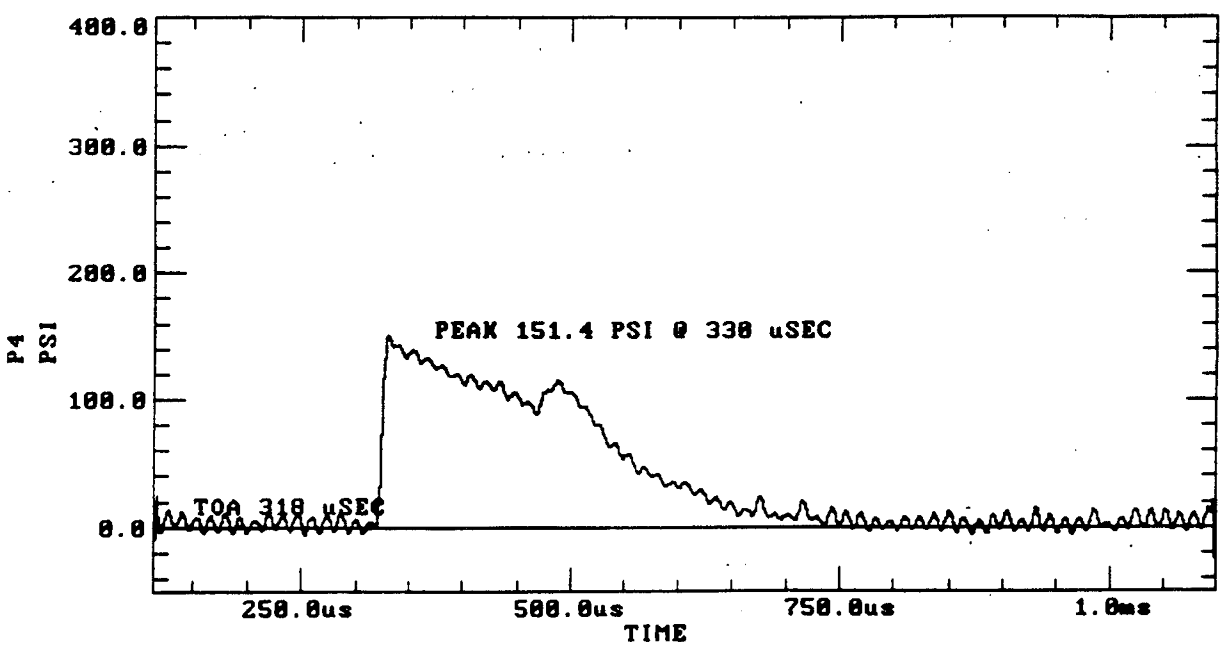

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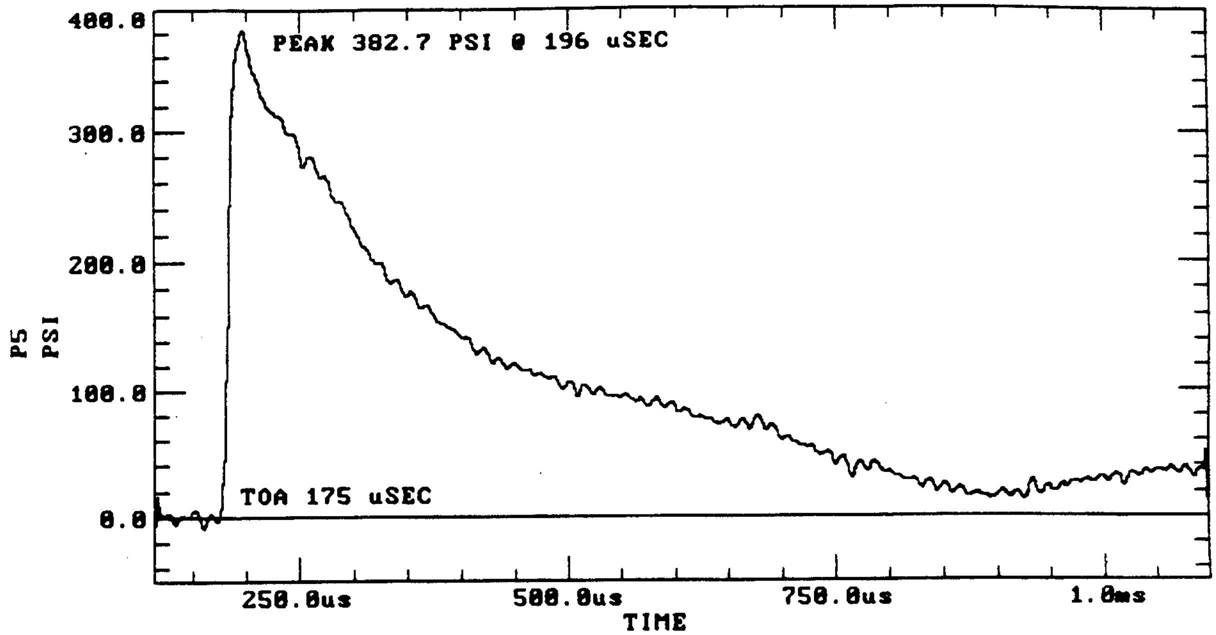

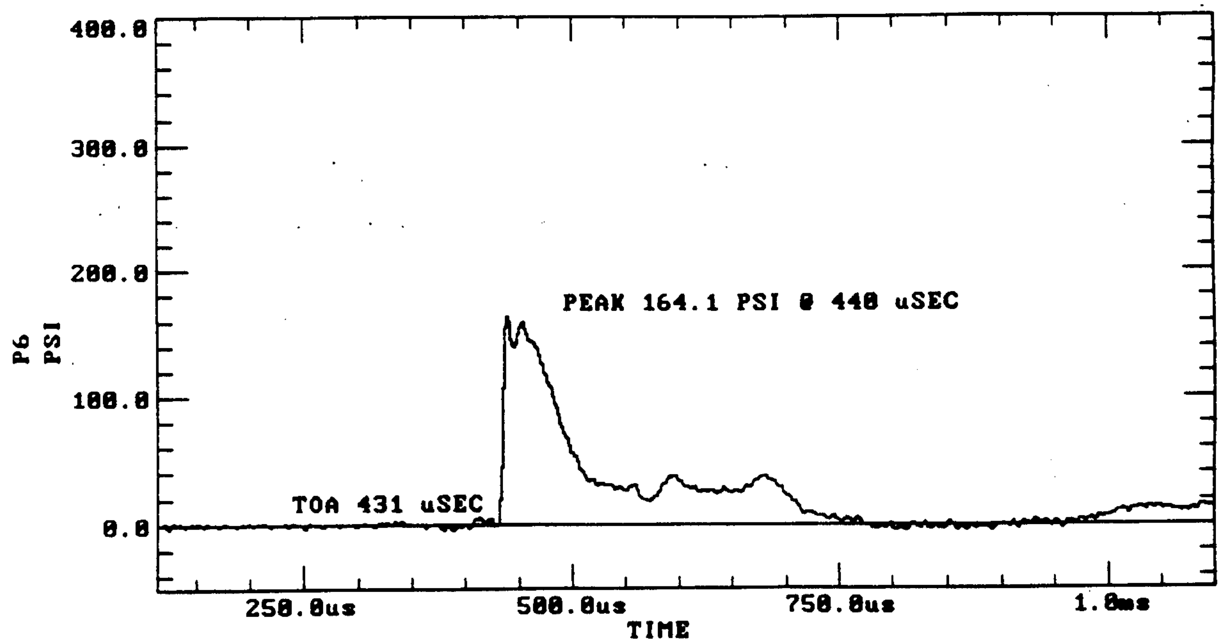

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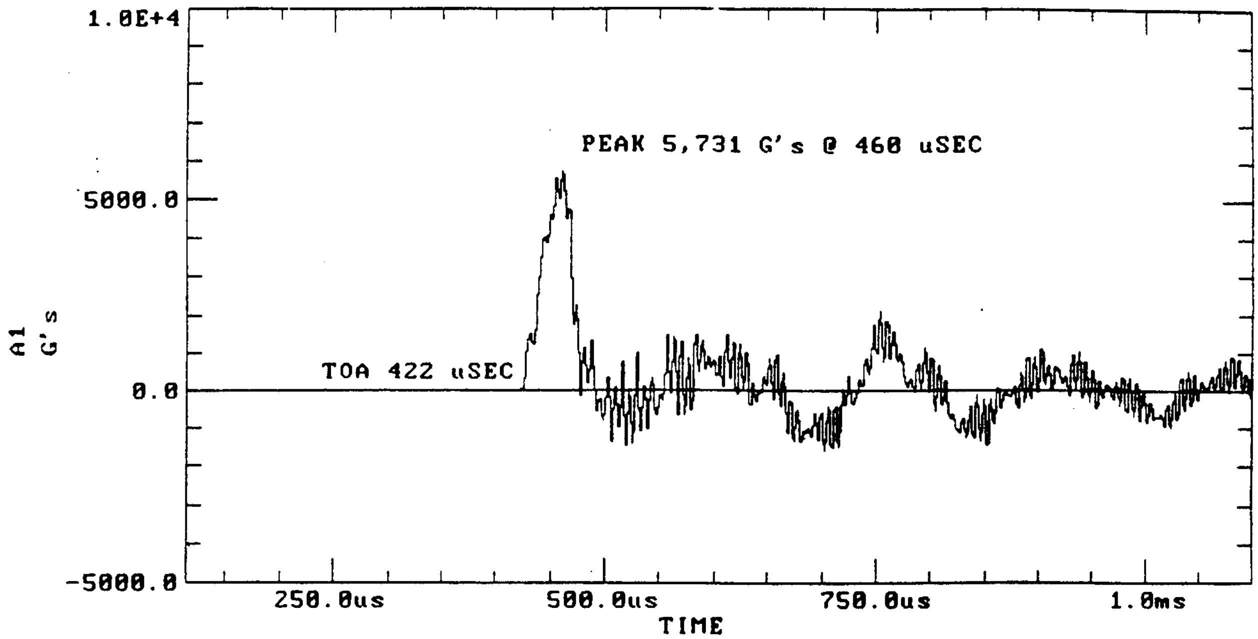

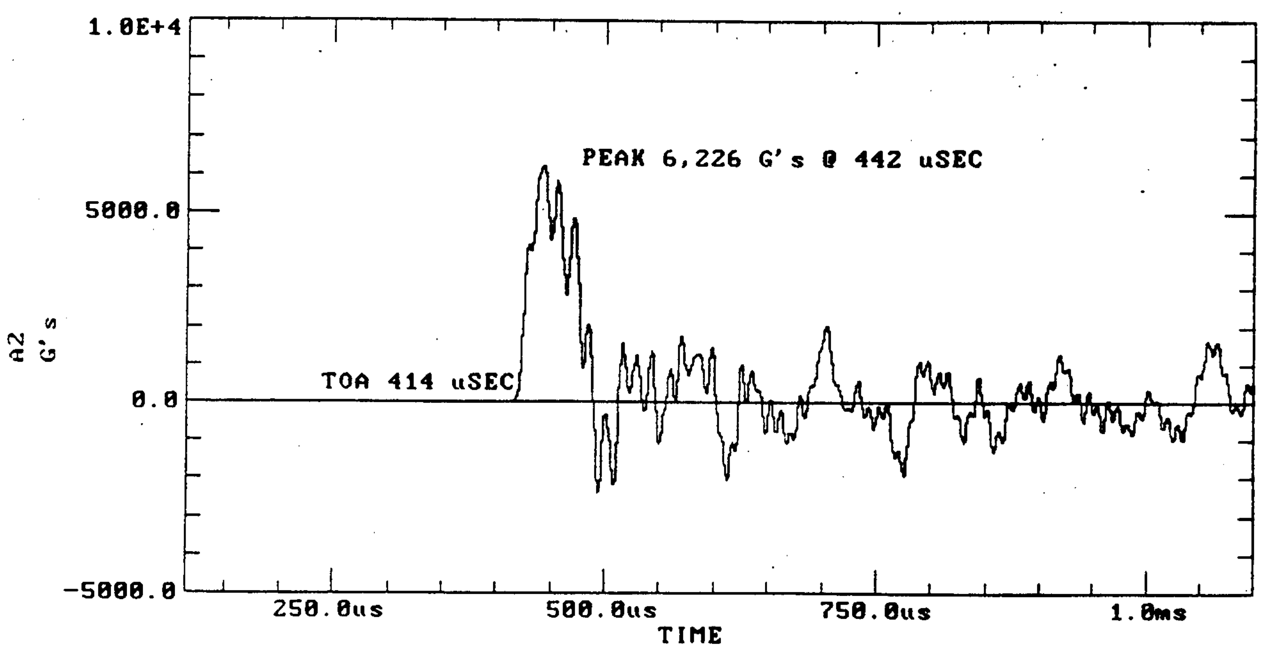

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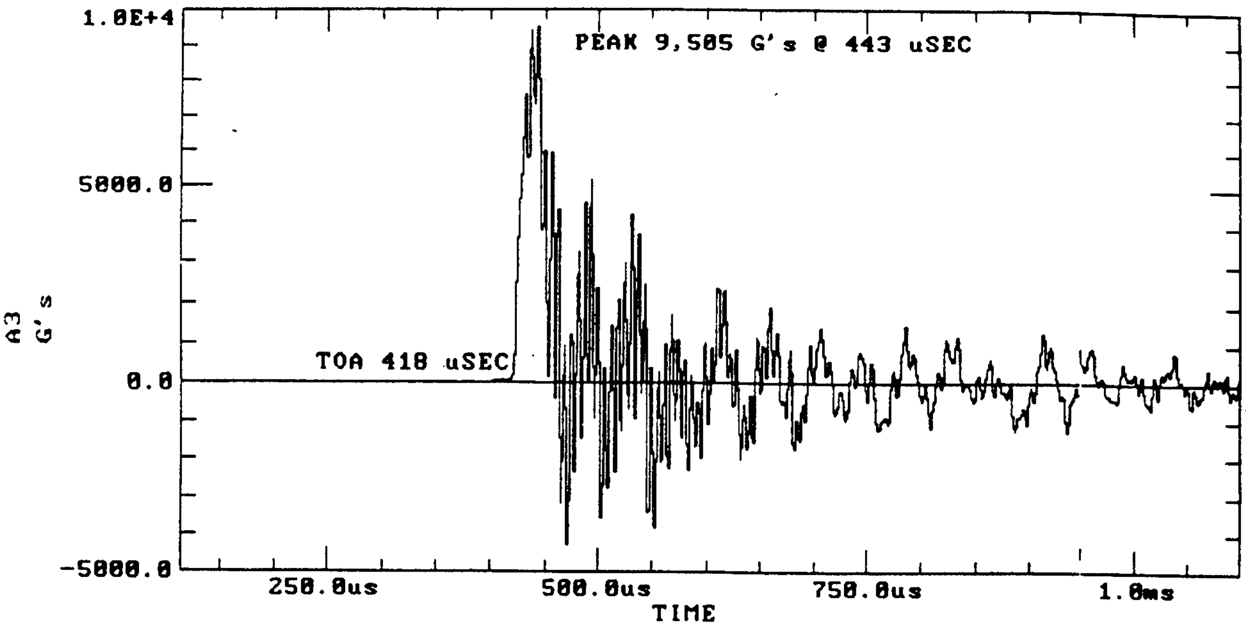

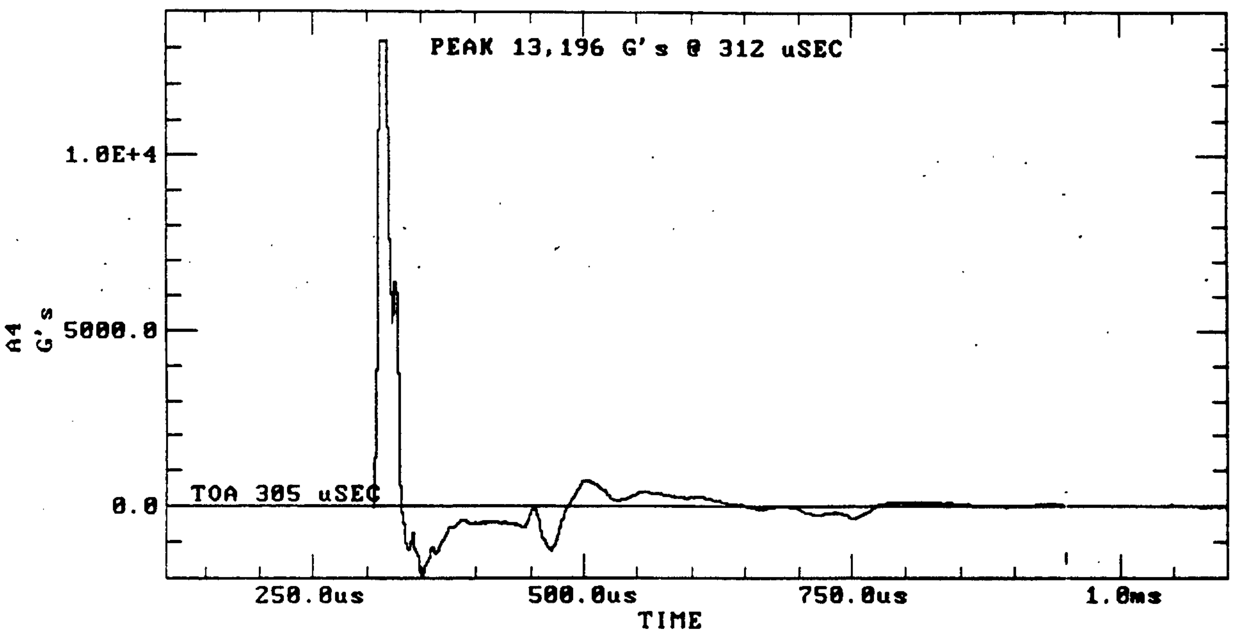

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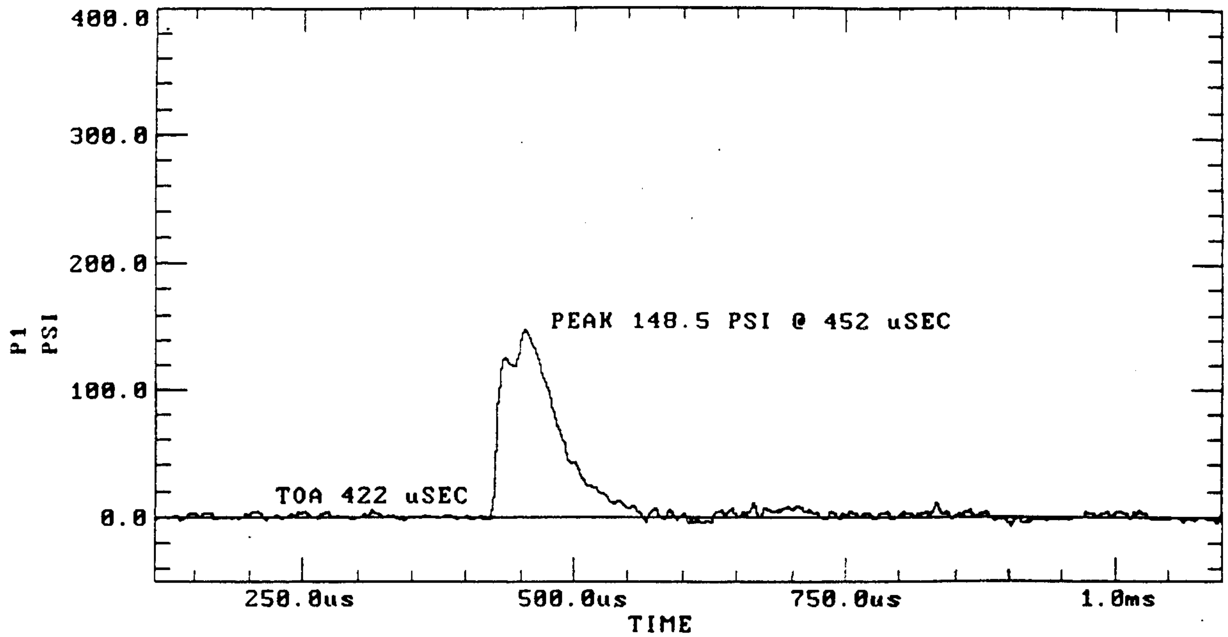

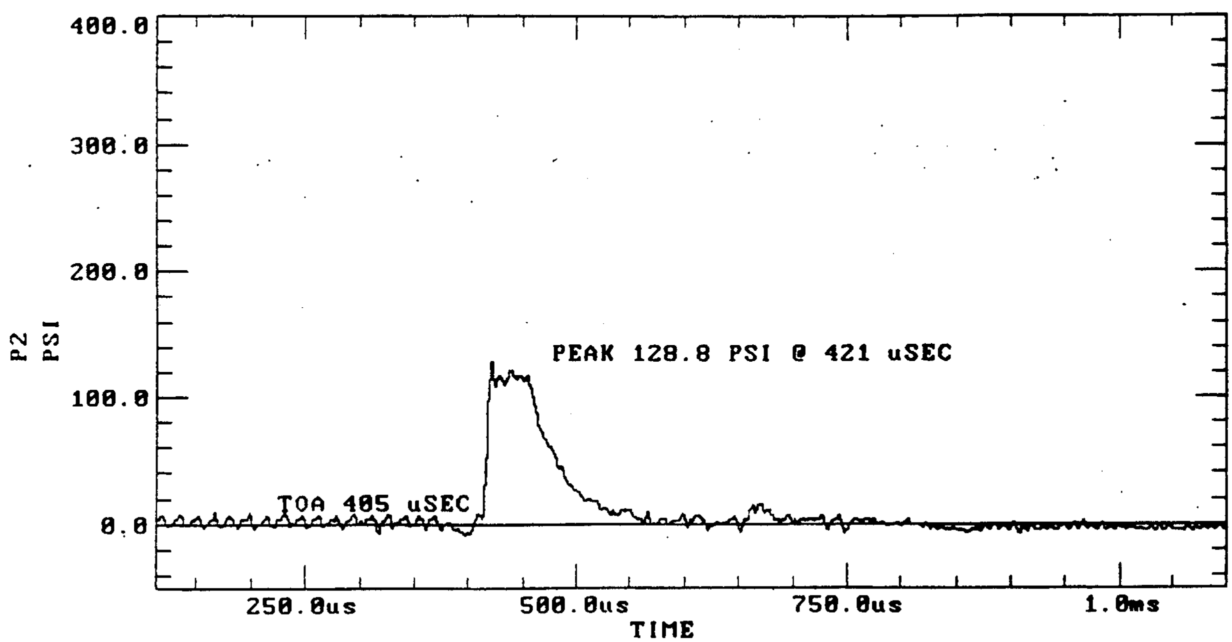

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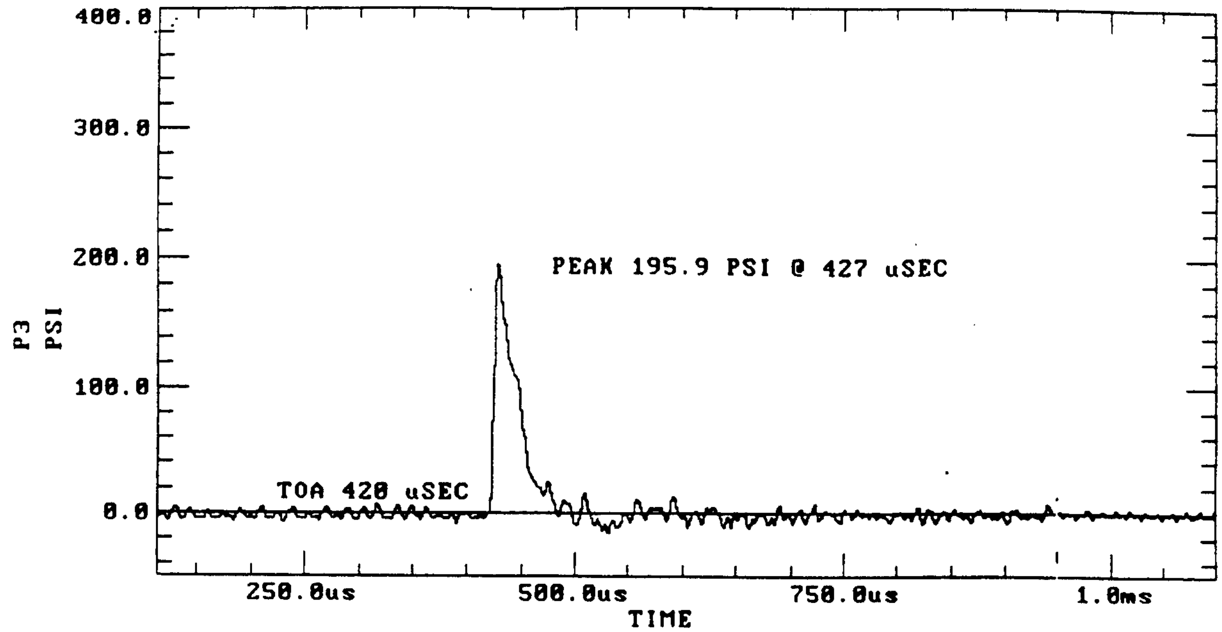

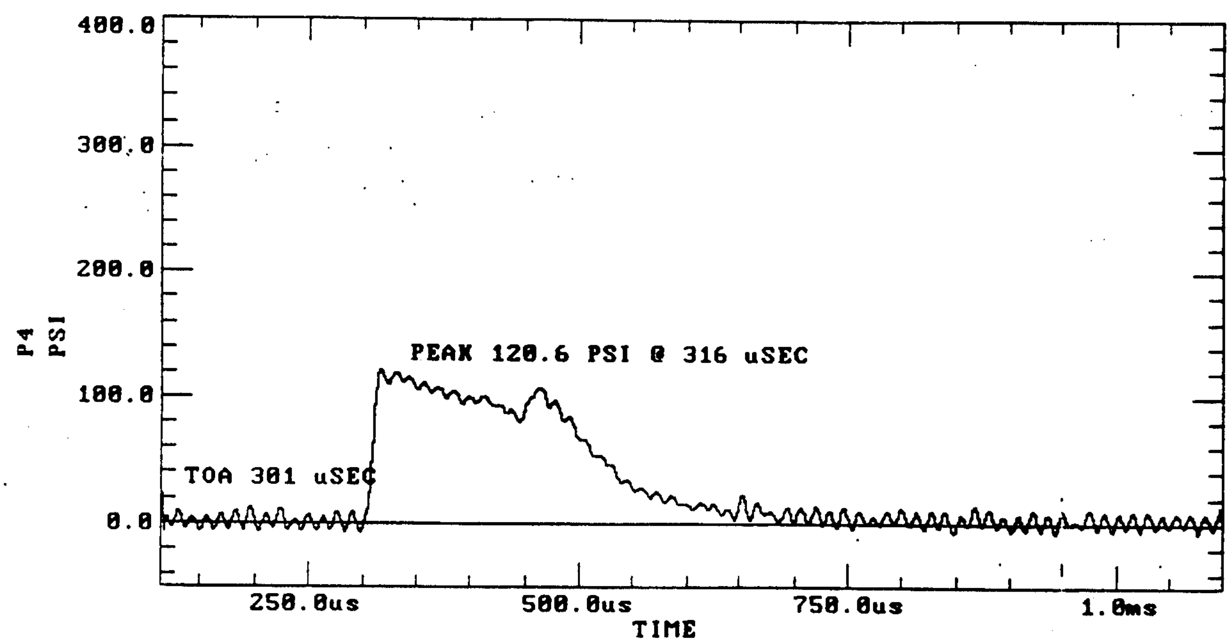

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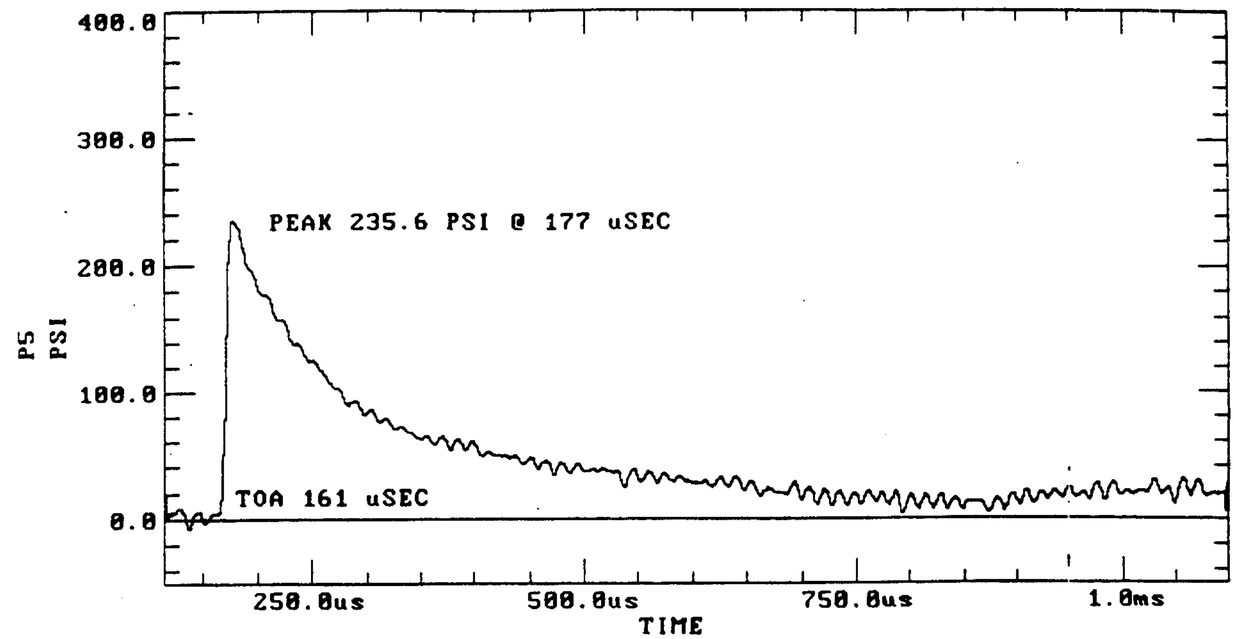

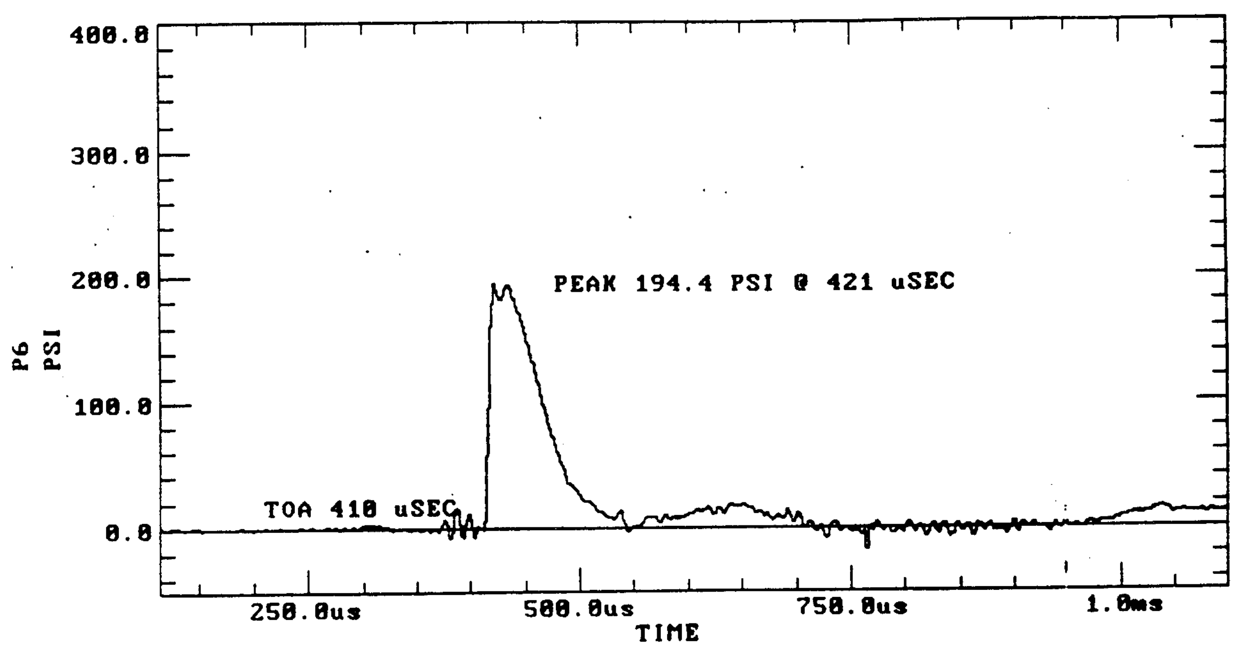

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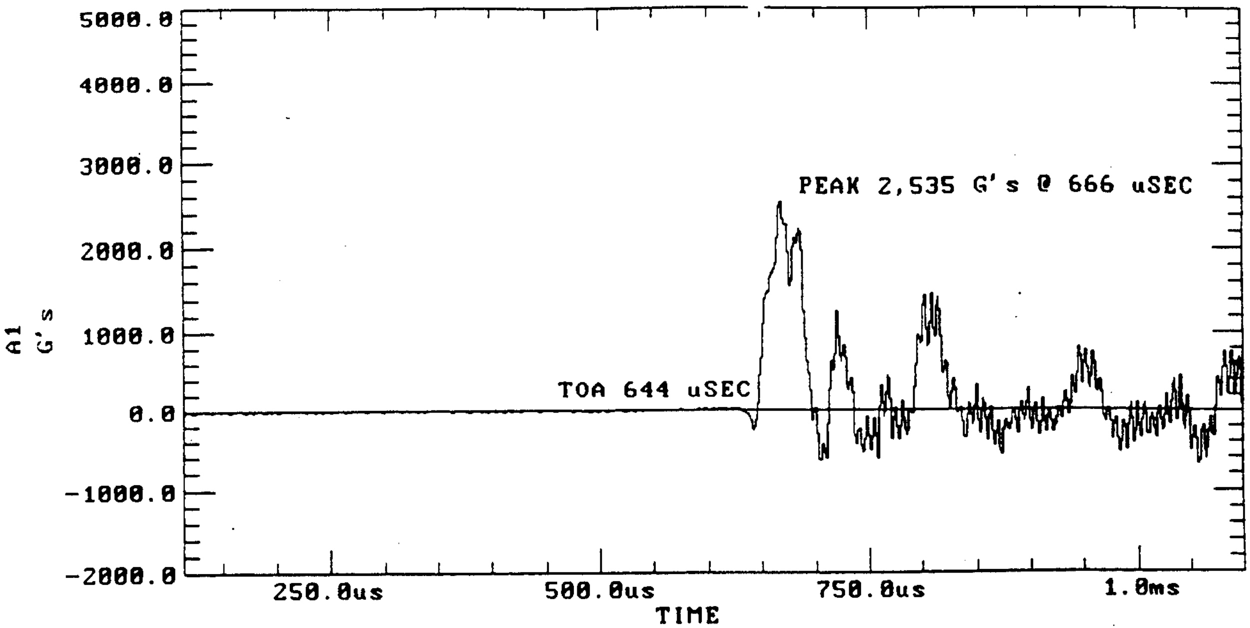

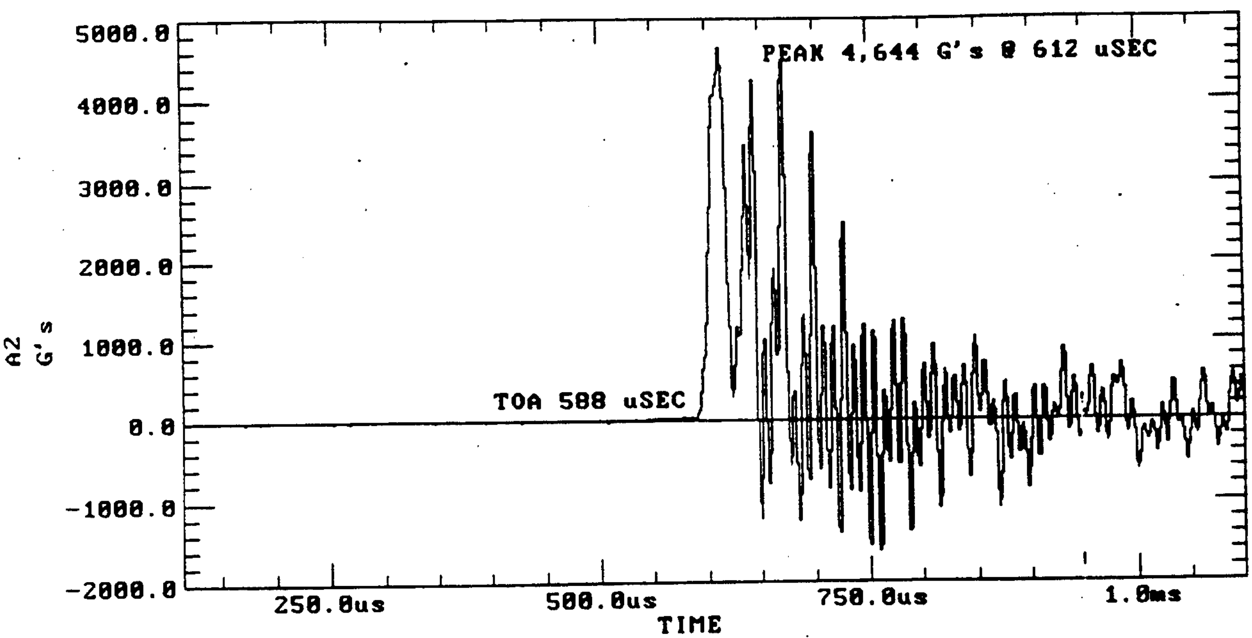

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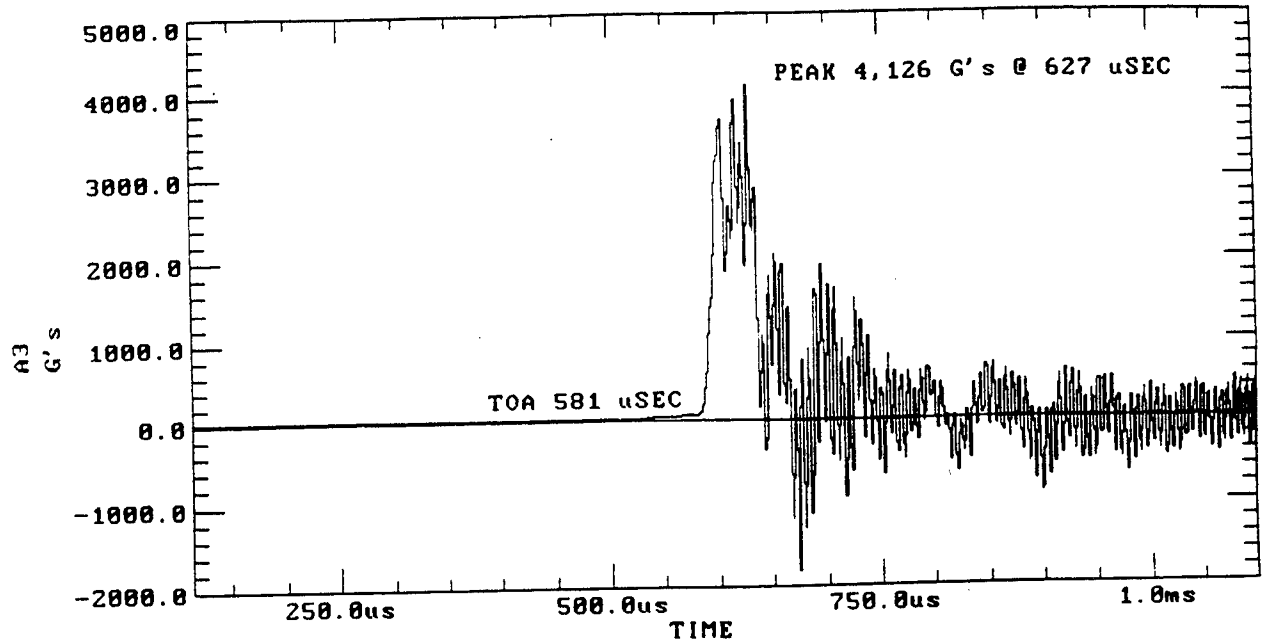

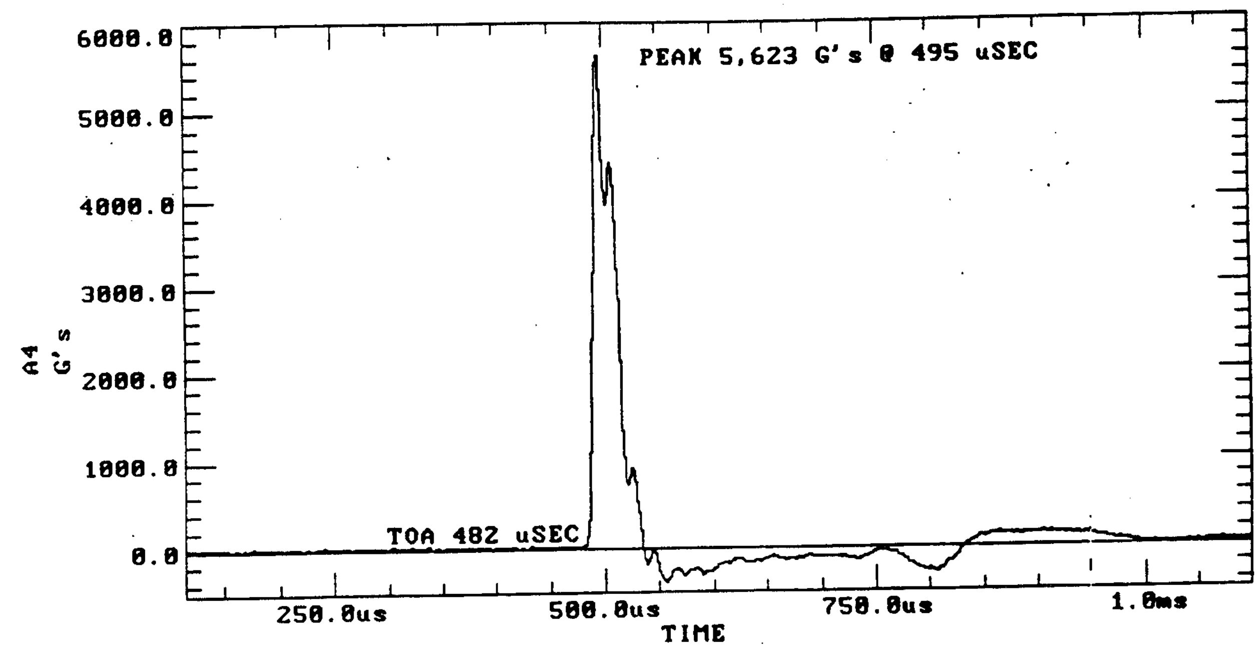

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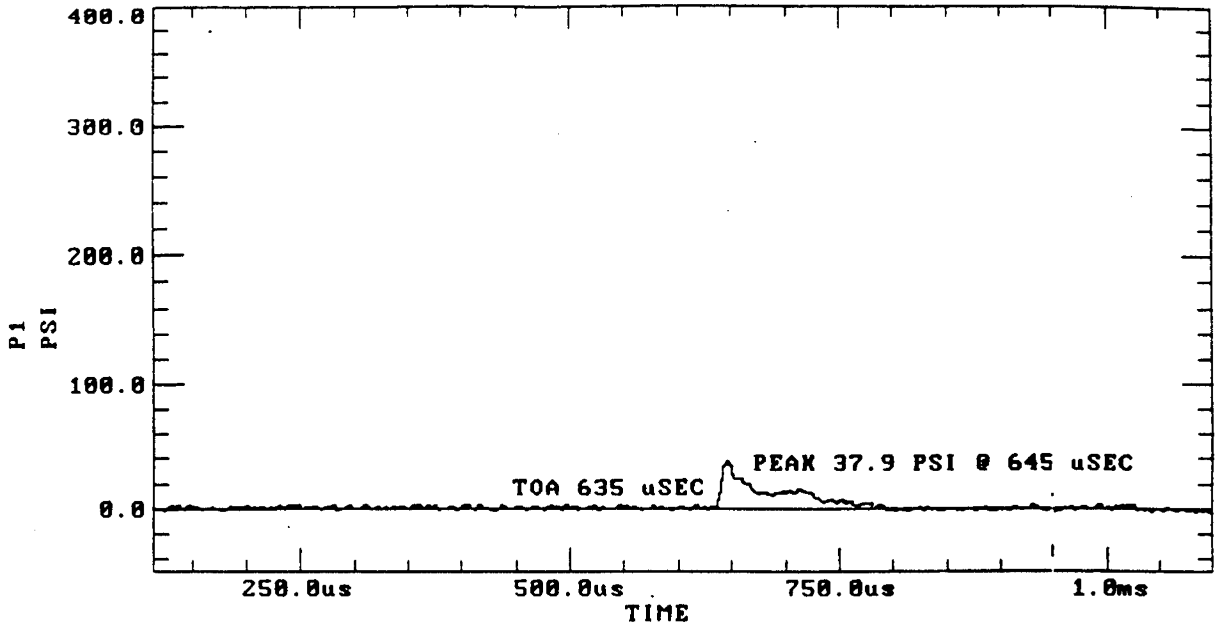

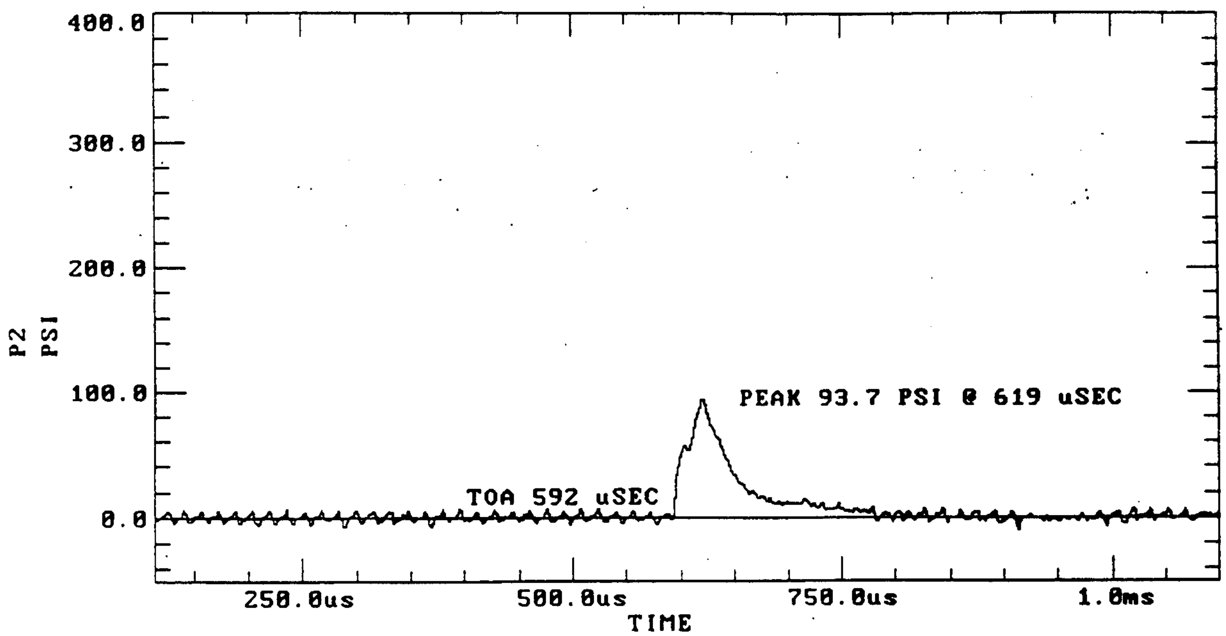

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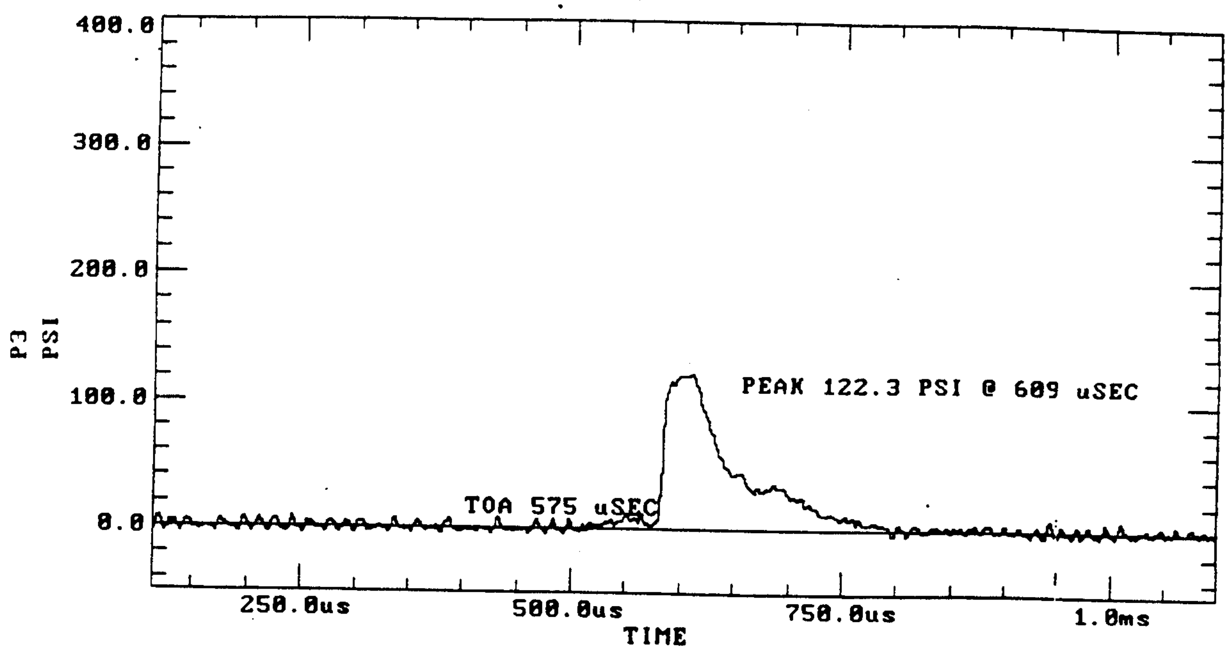

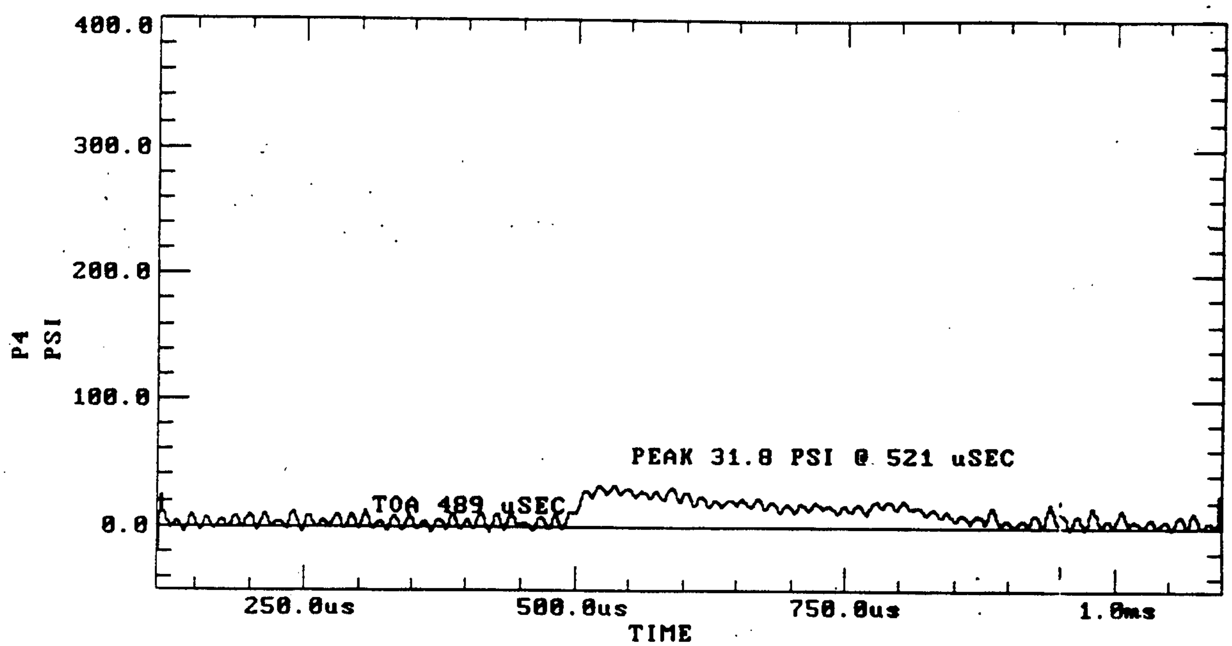

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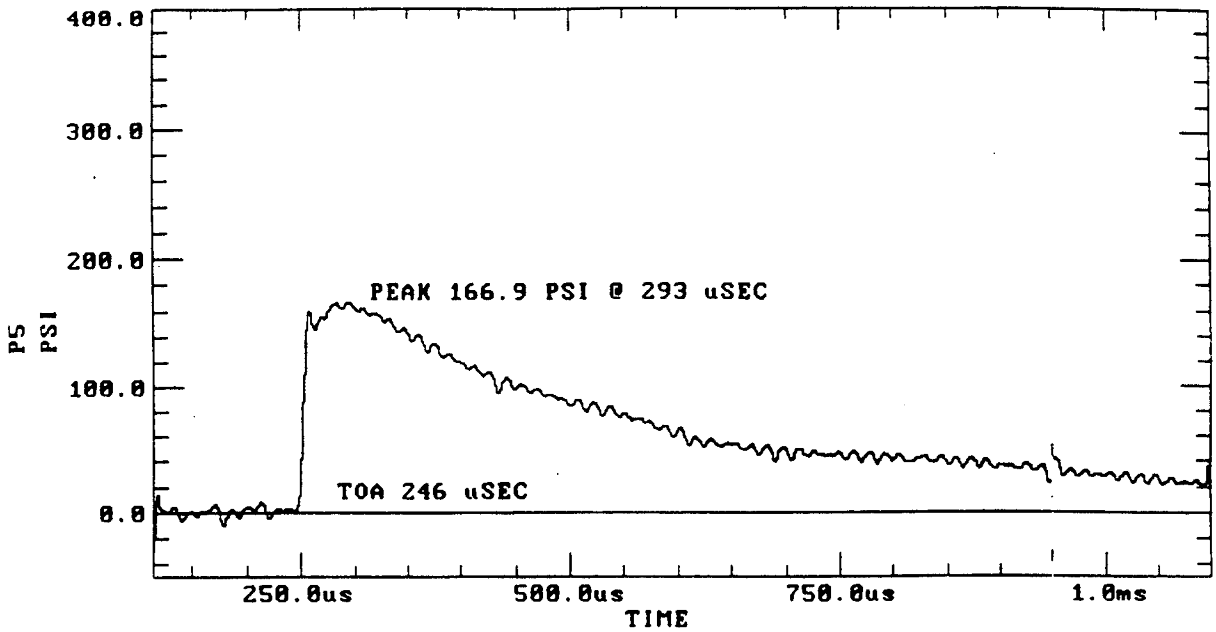

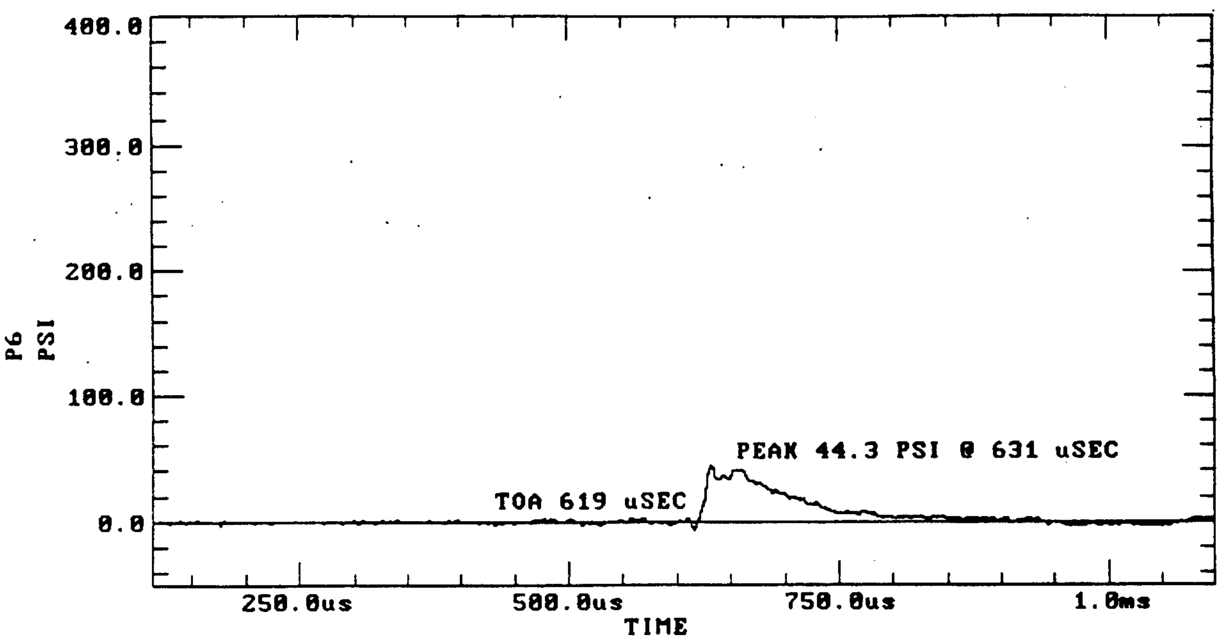

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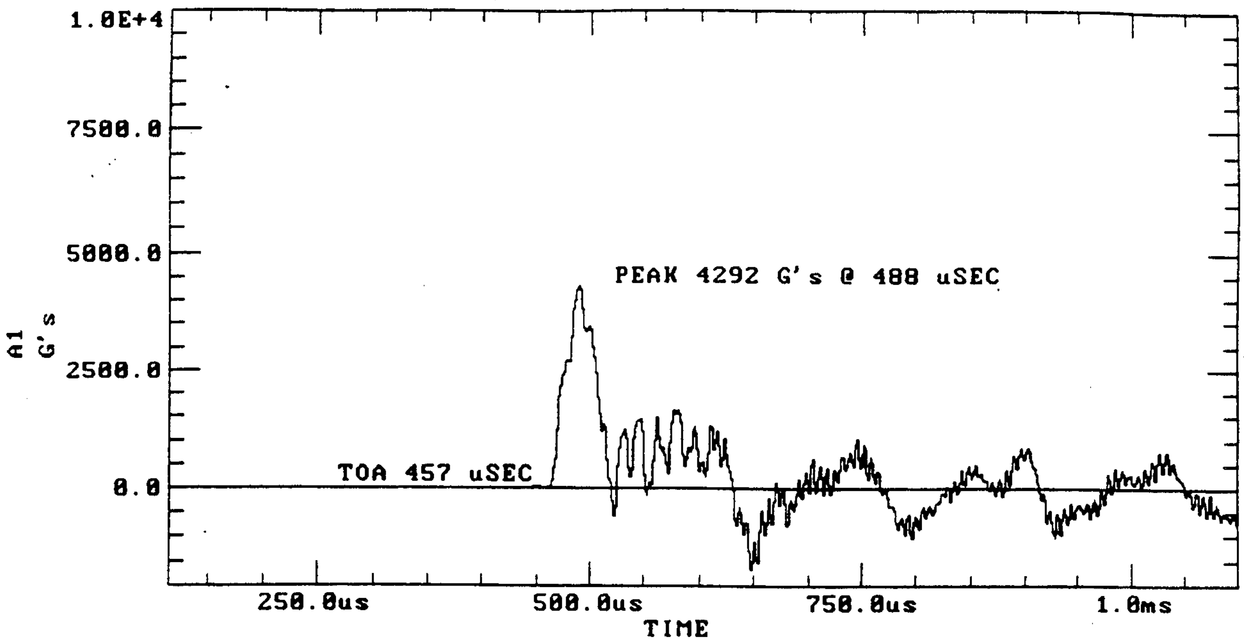

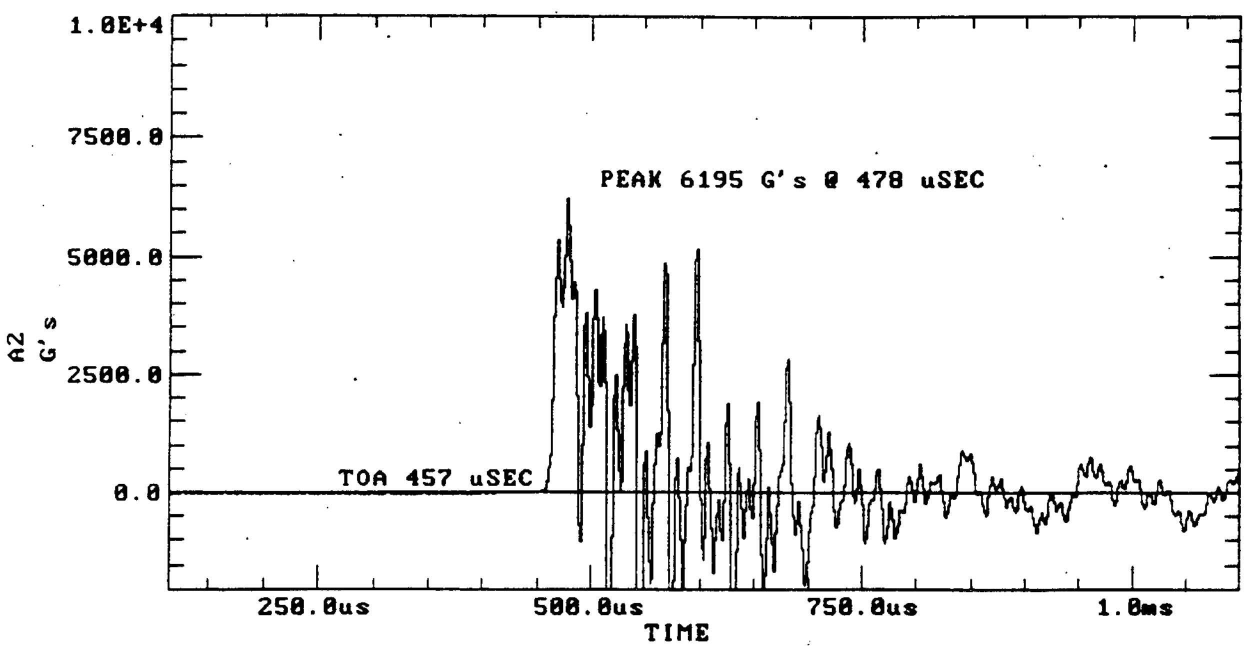

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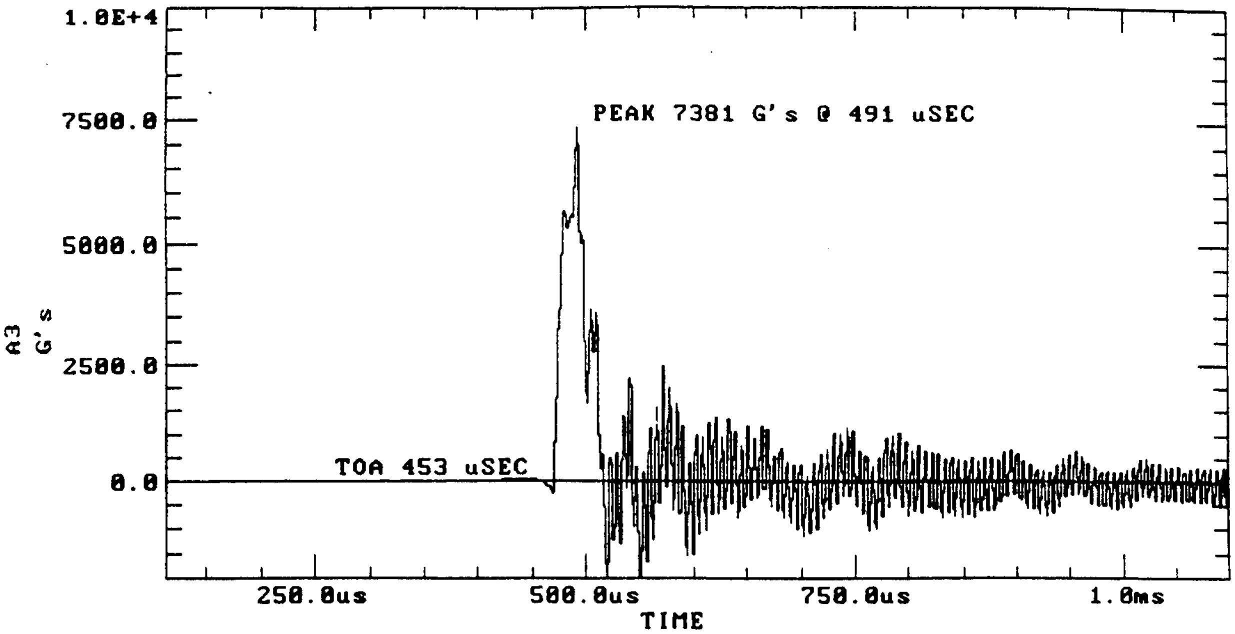

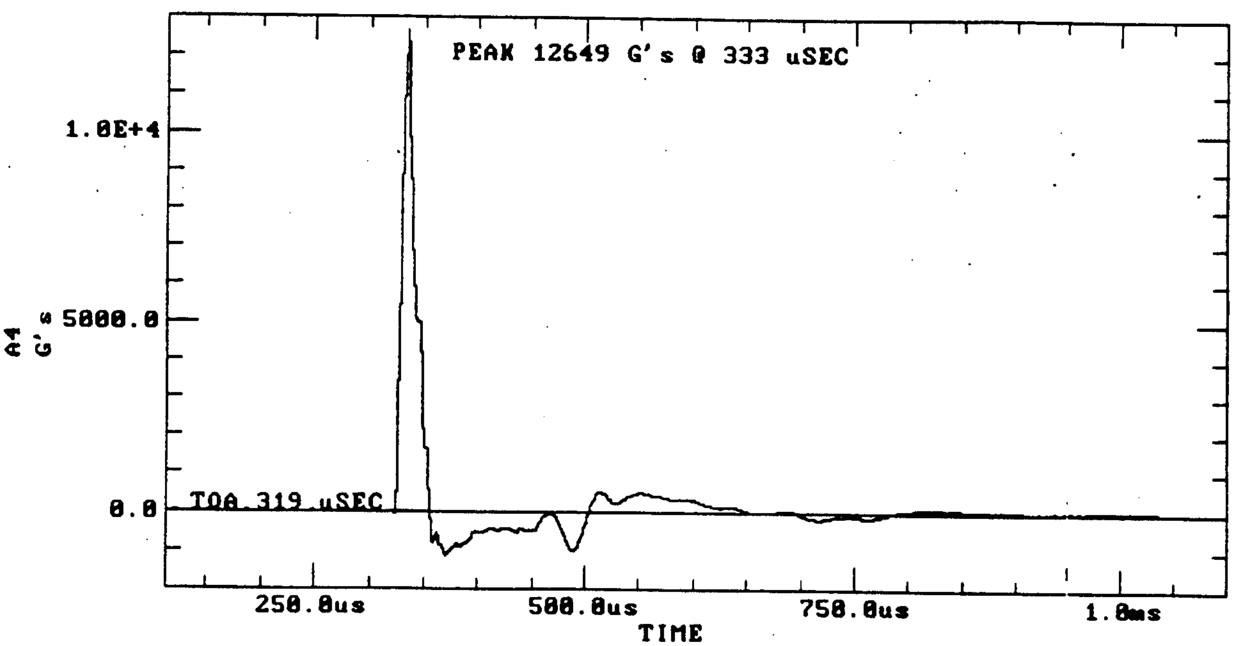

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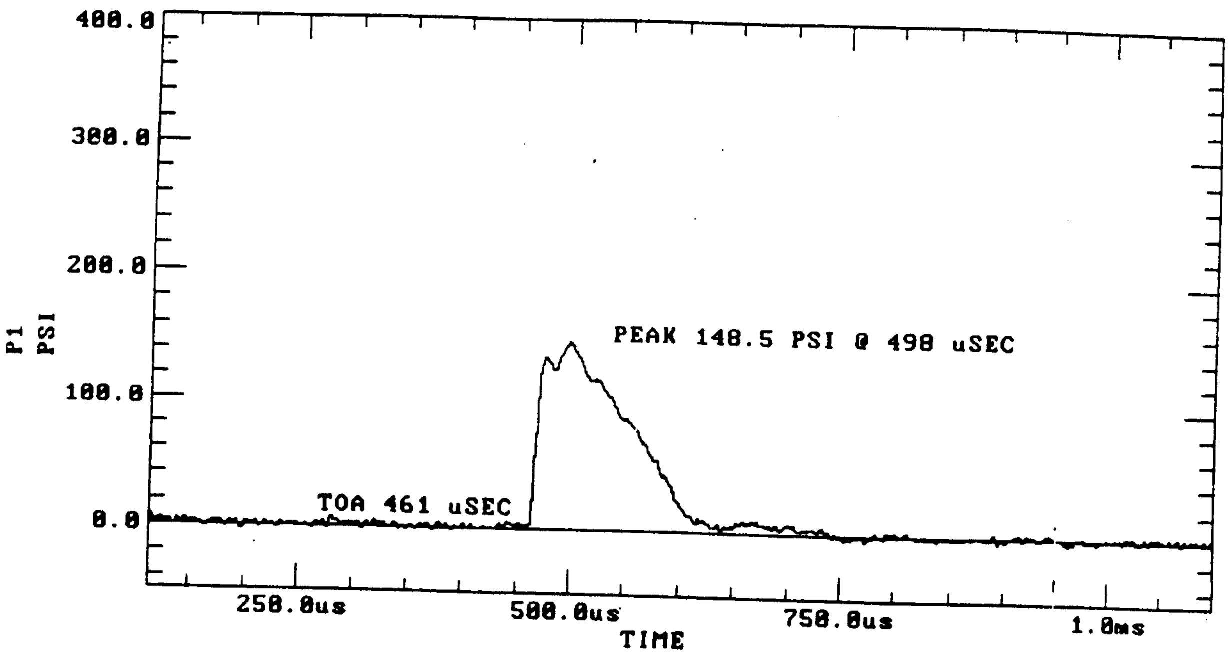

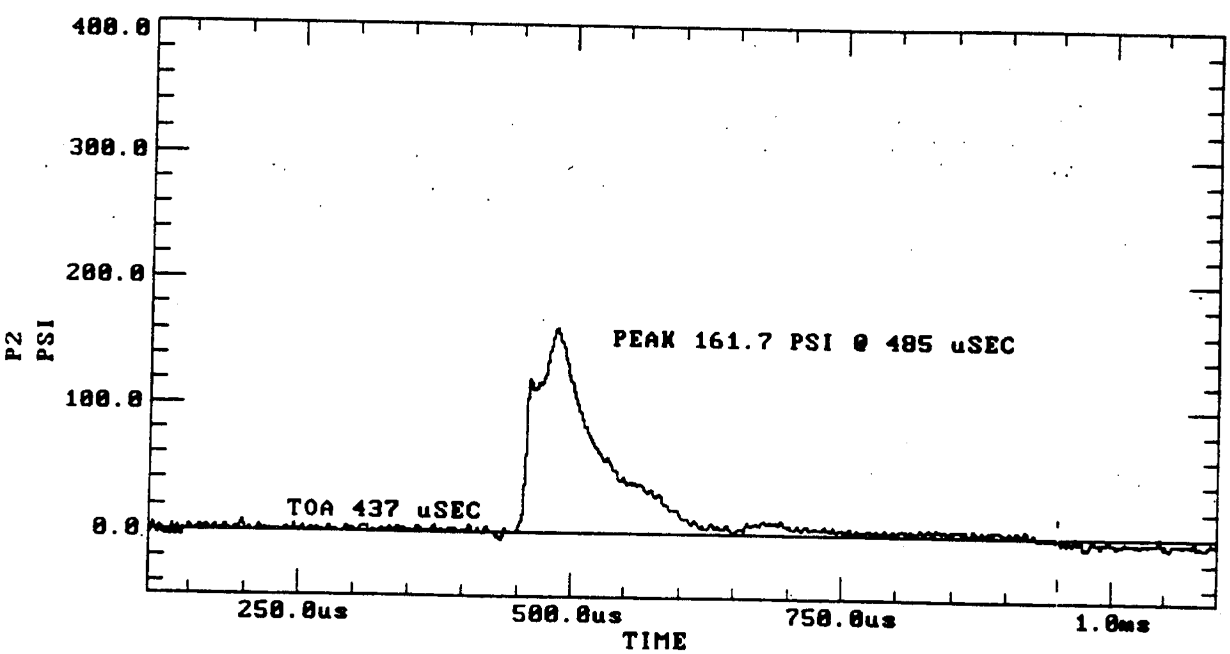

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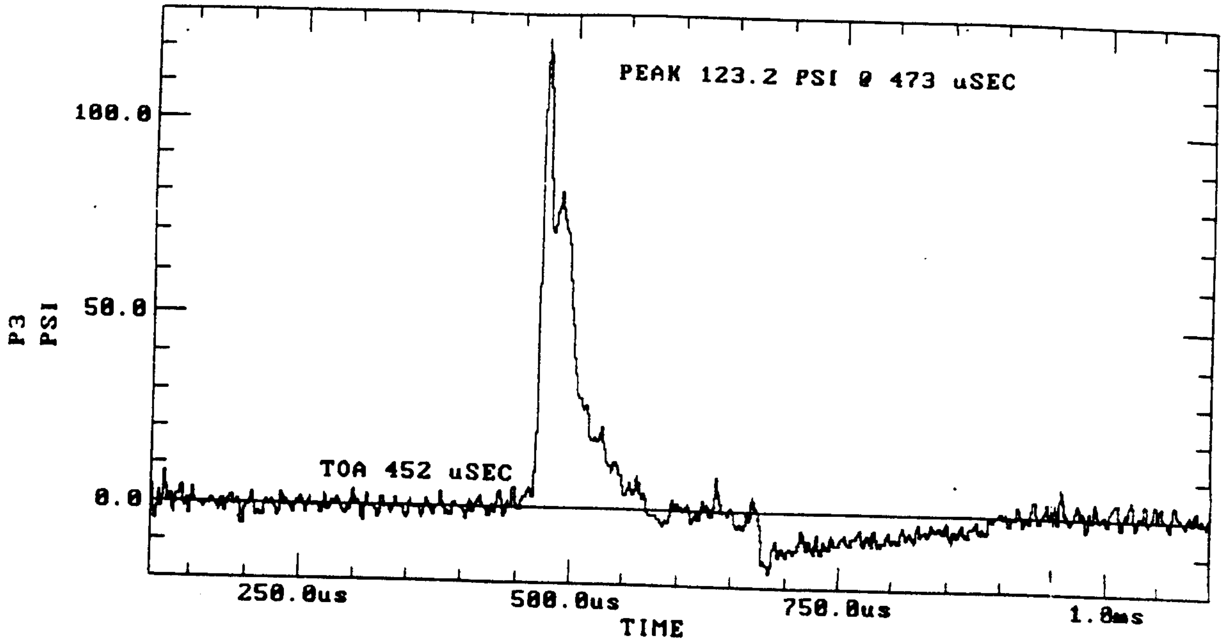

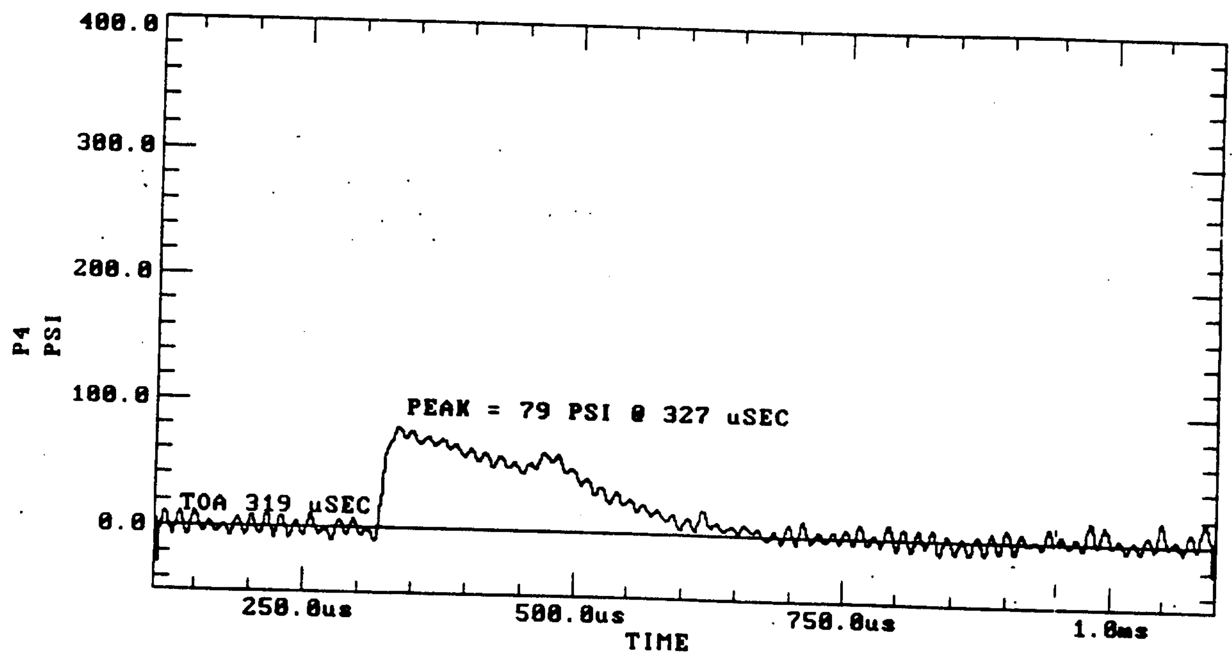

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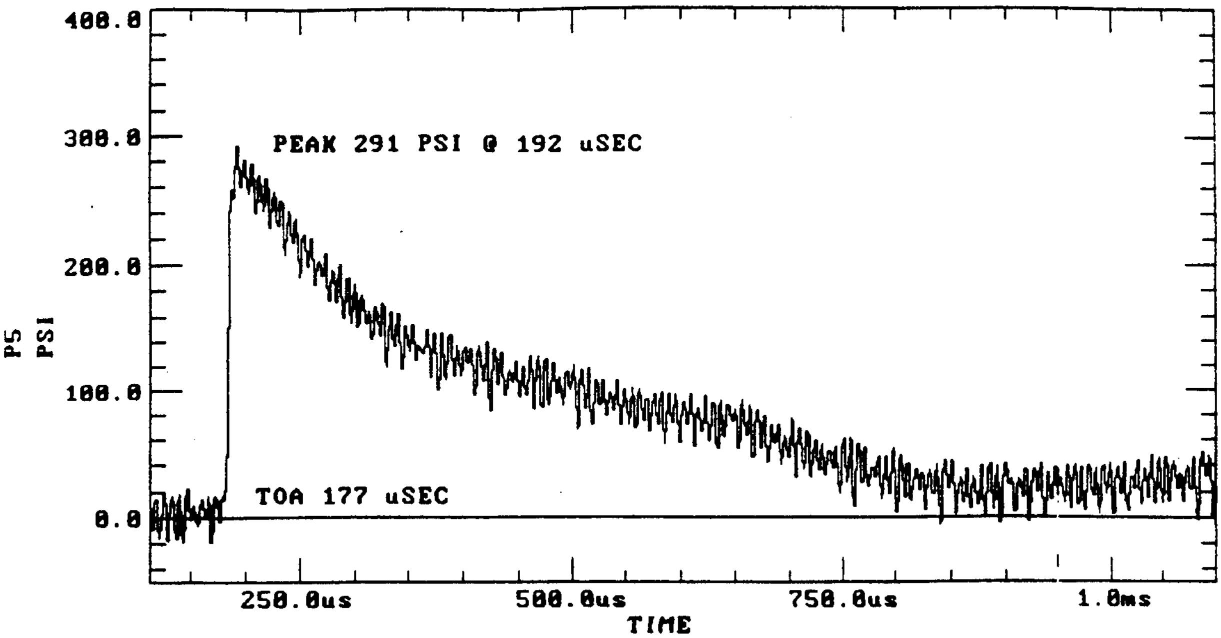
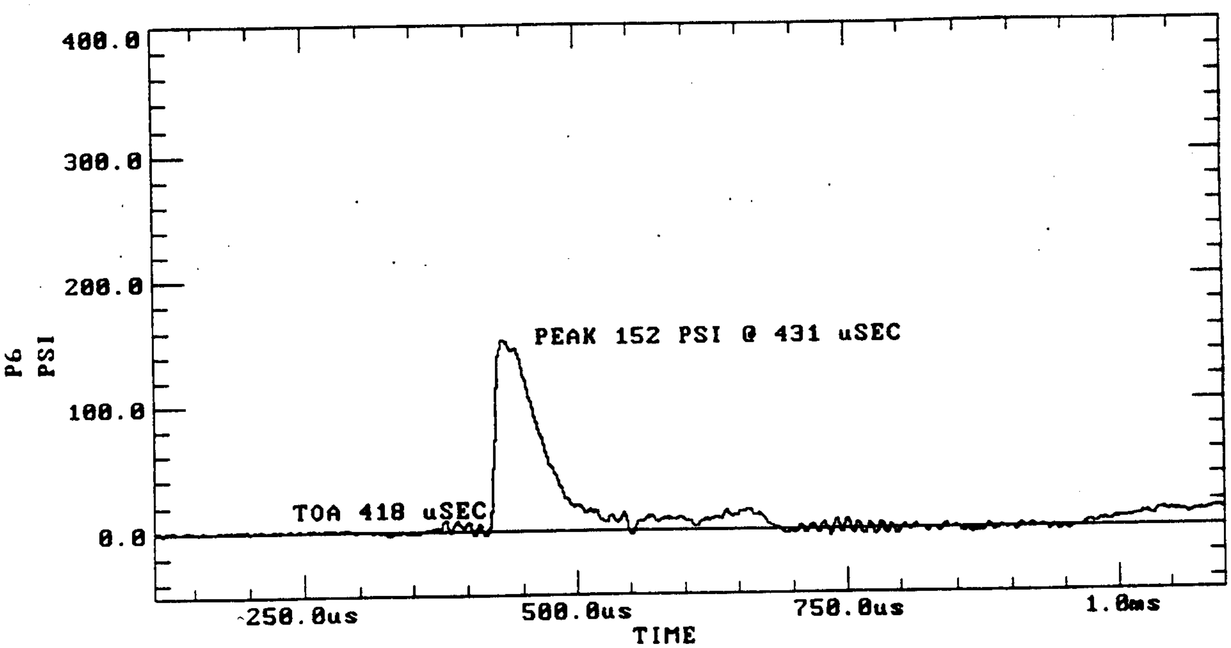
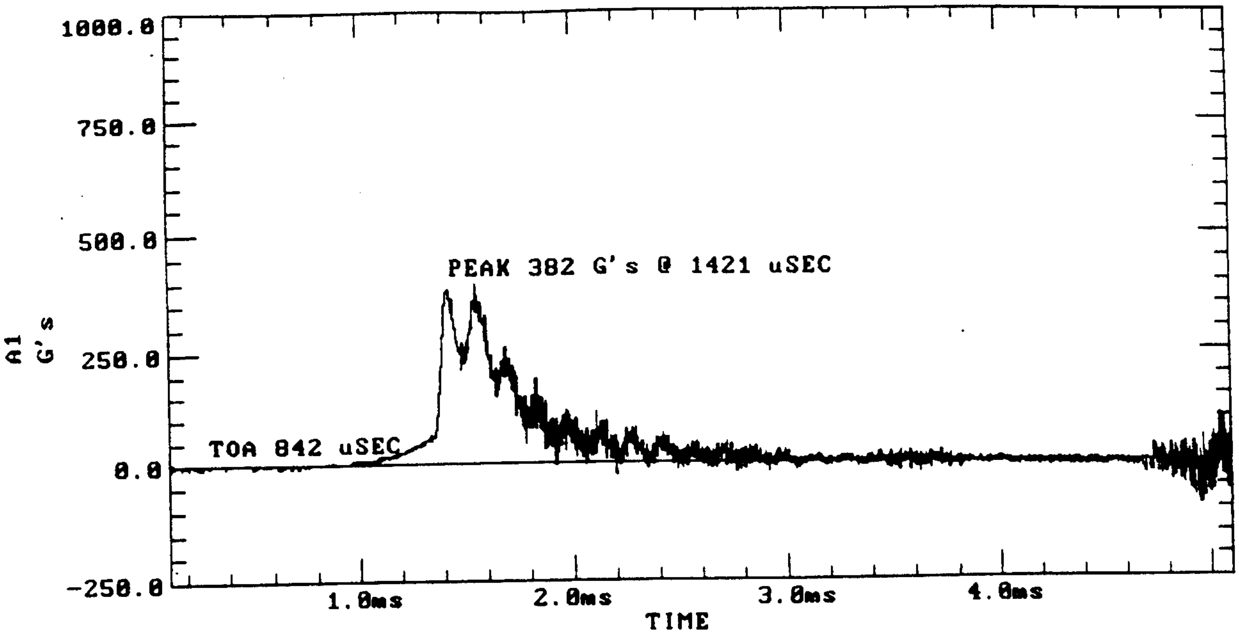
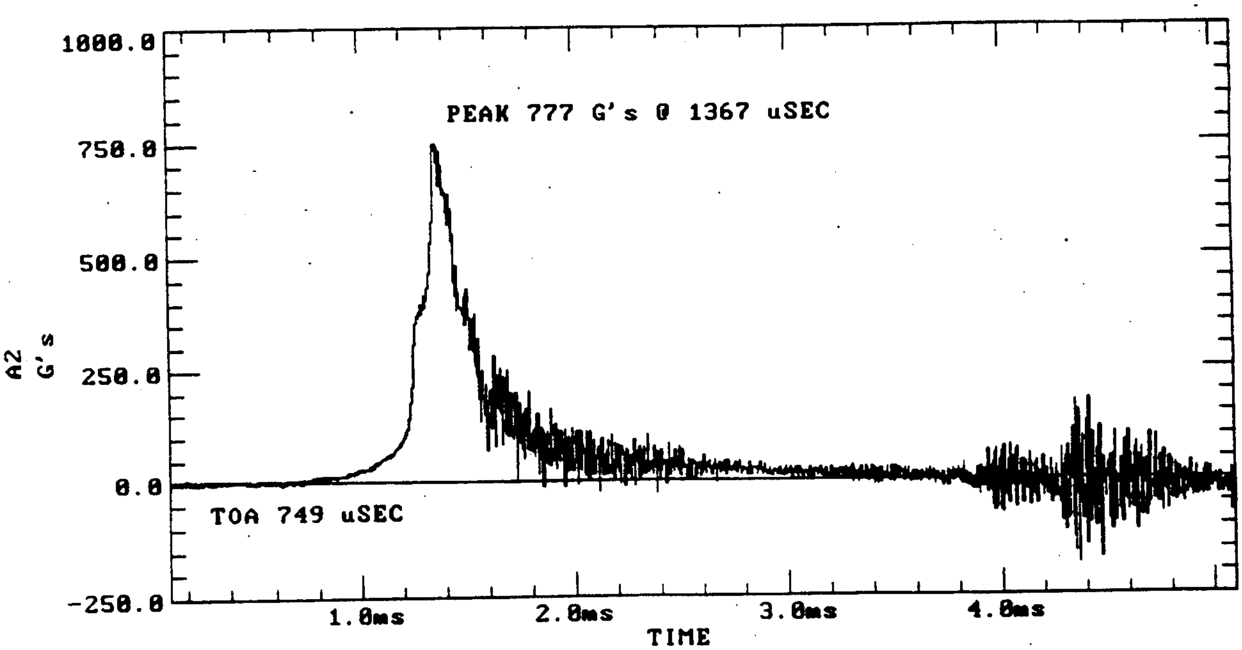




Test 19. Gage Data (continued)





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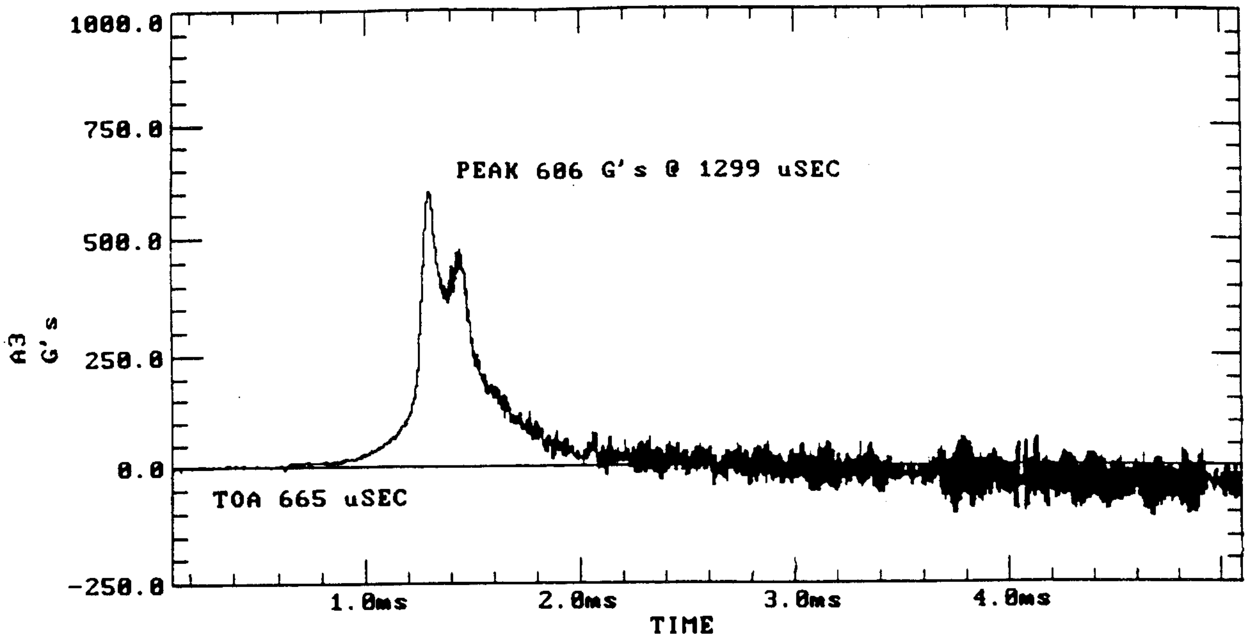

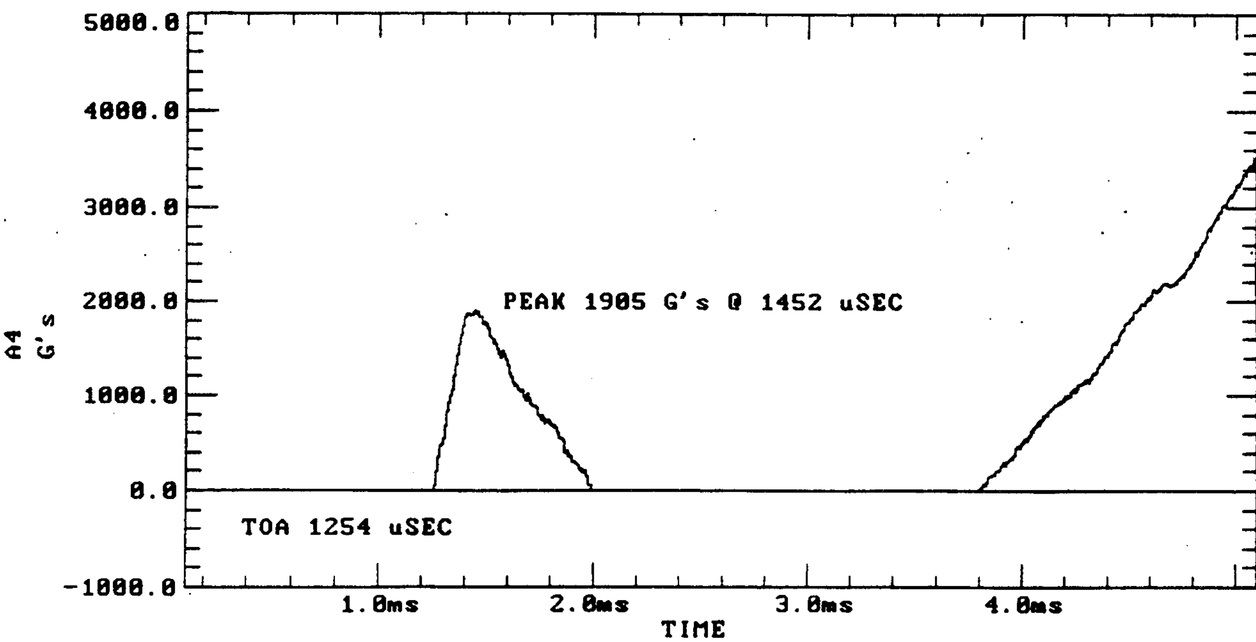

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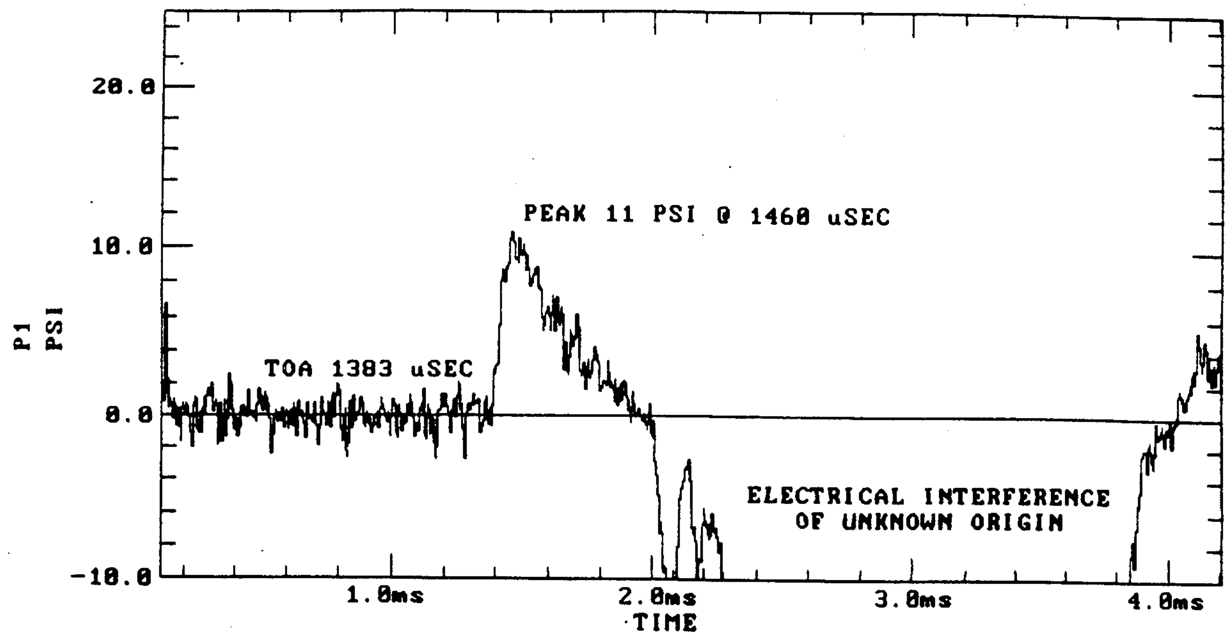

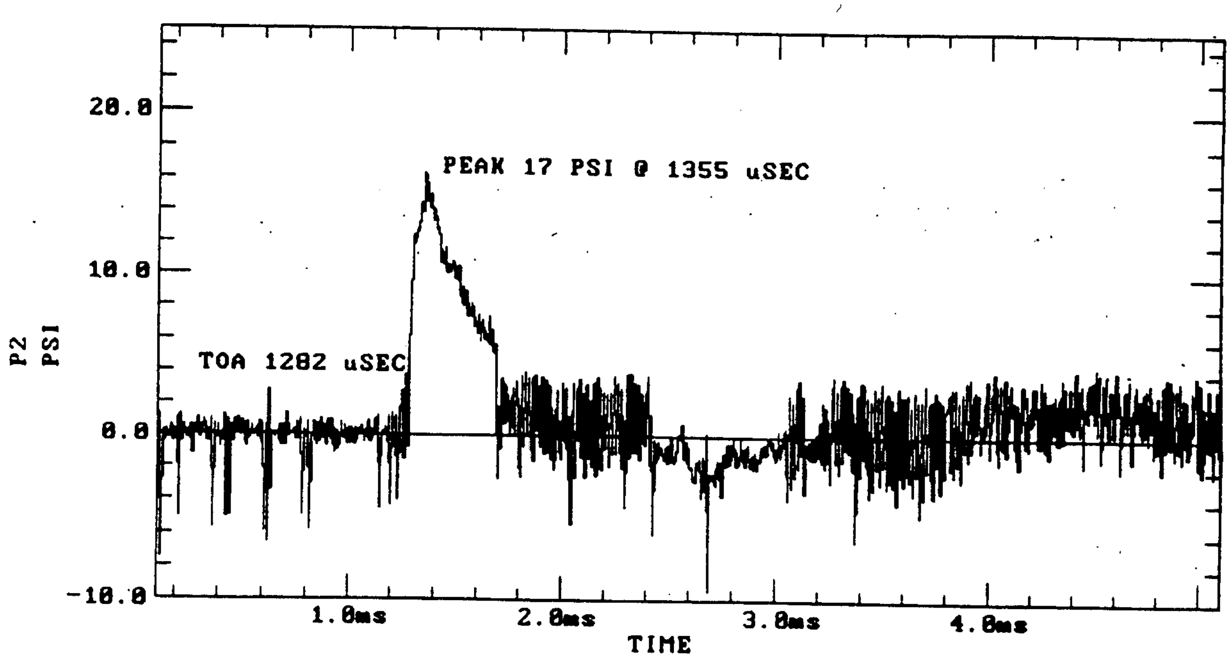

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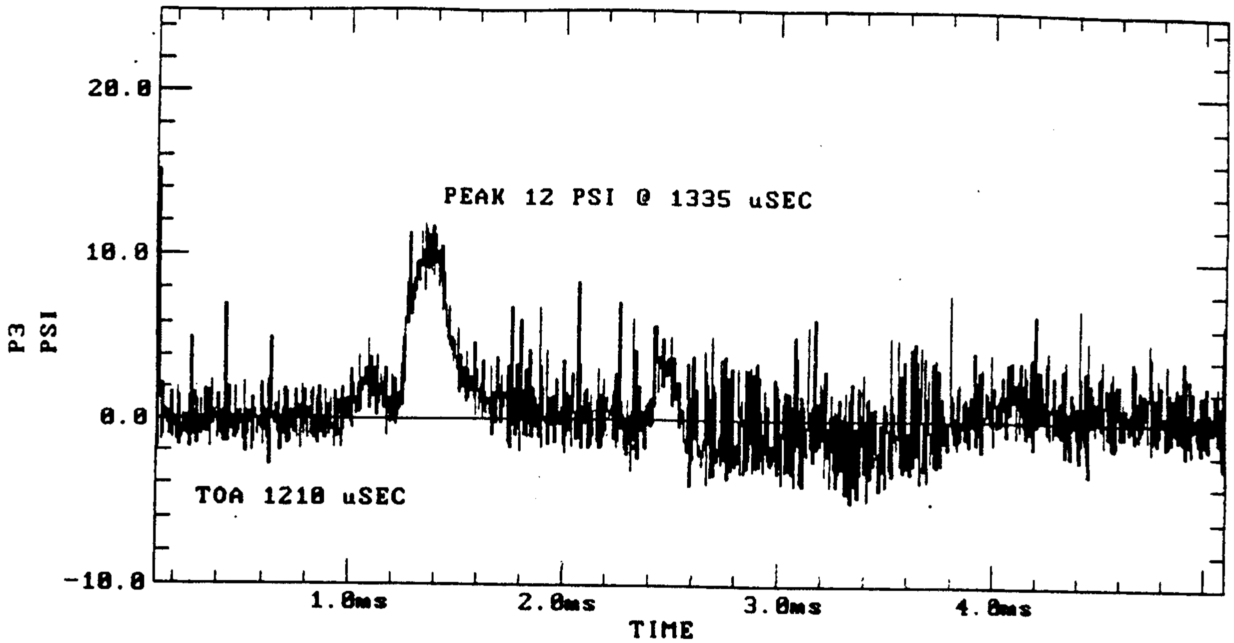

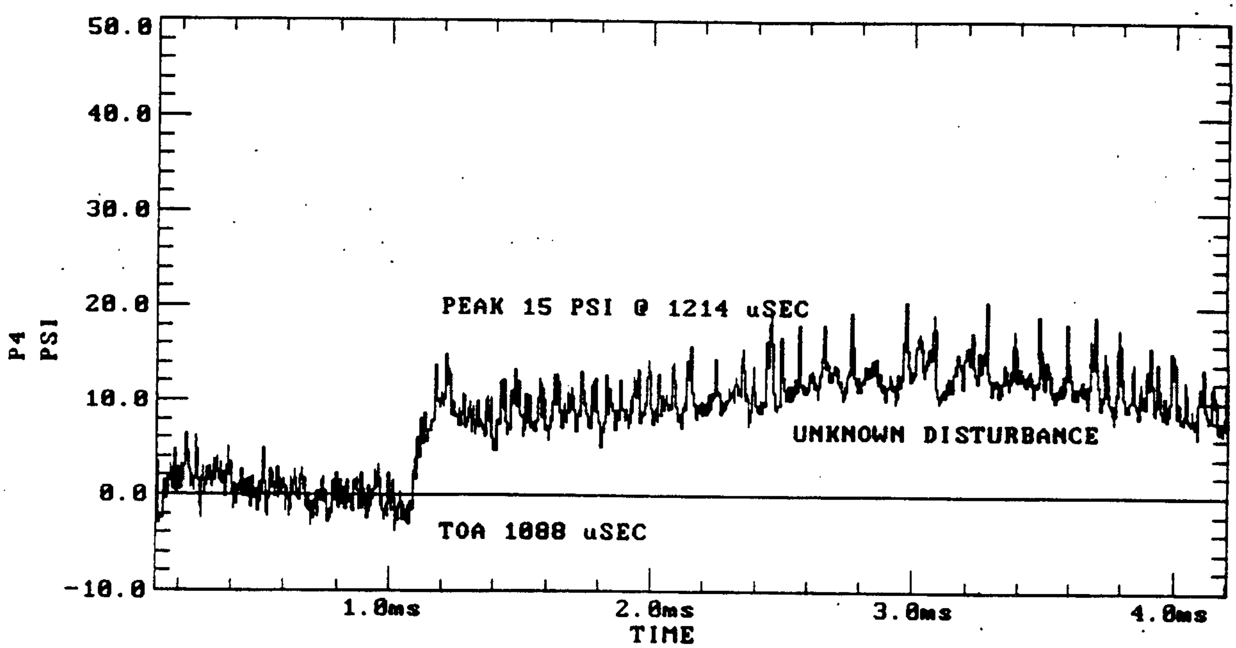

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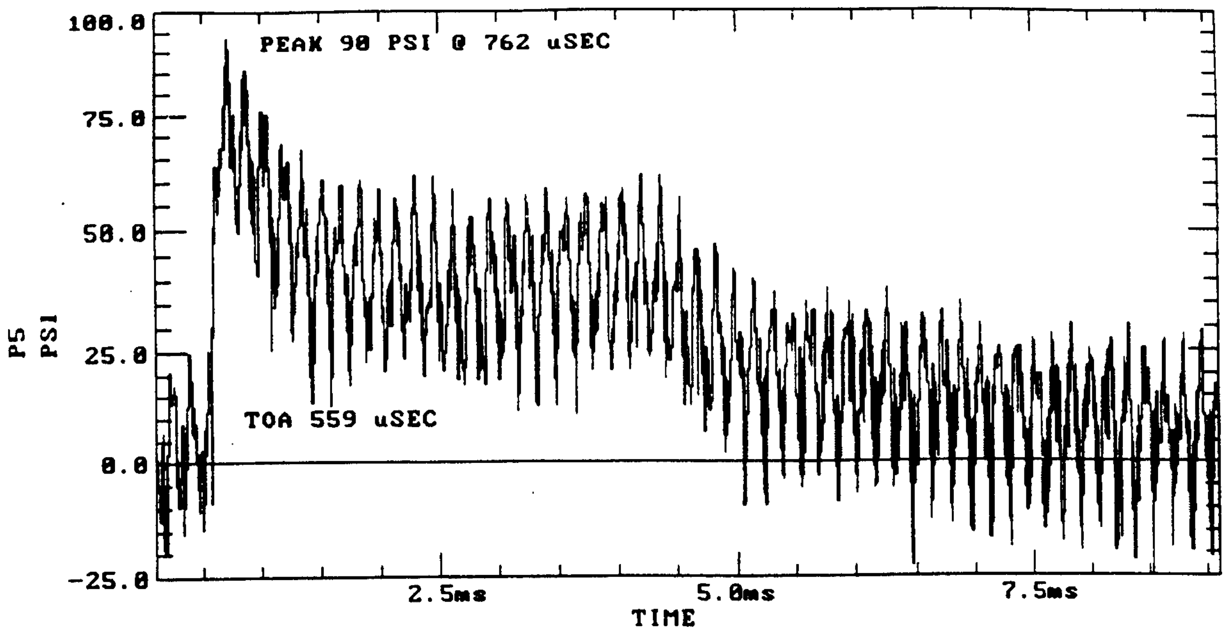

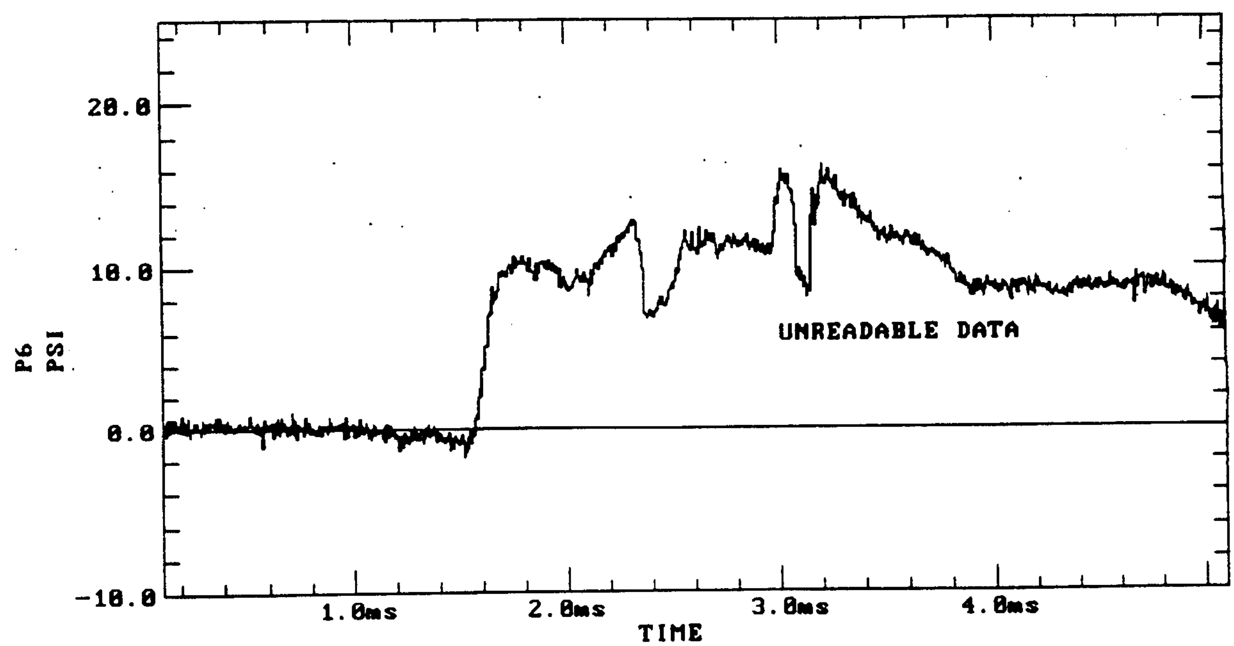

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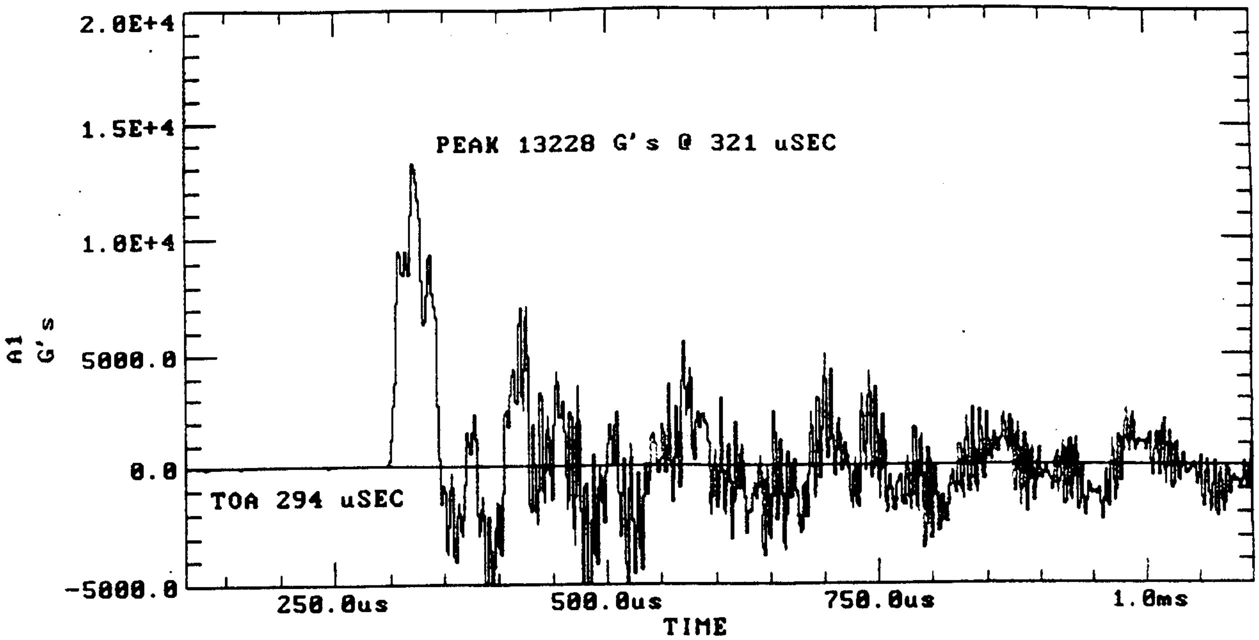

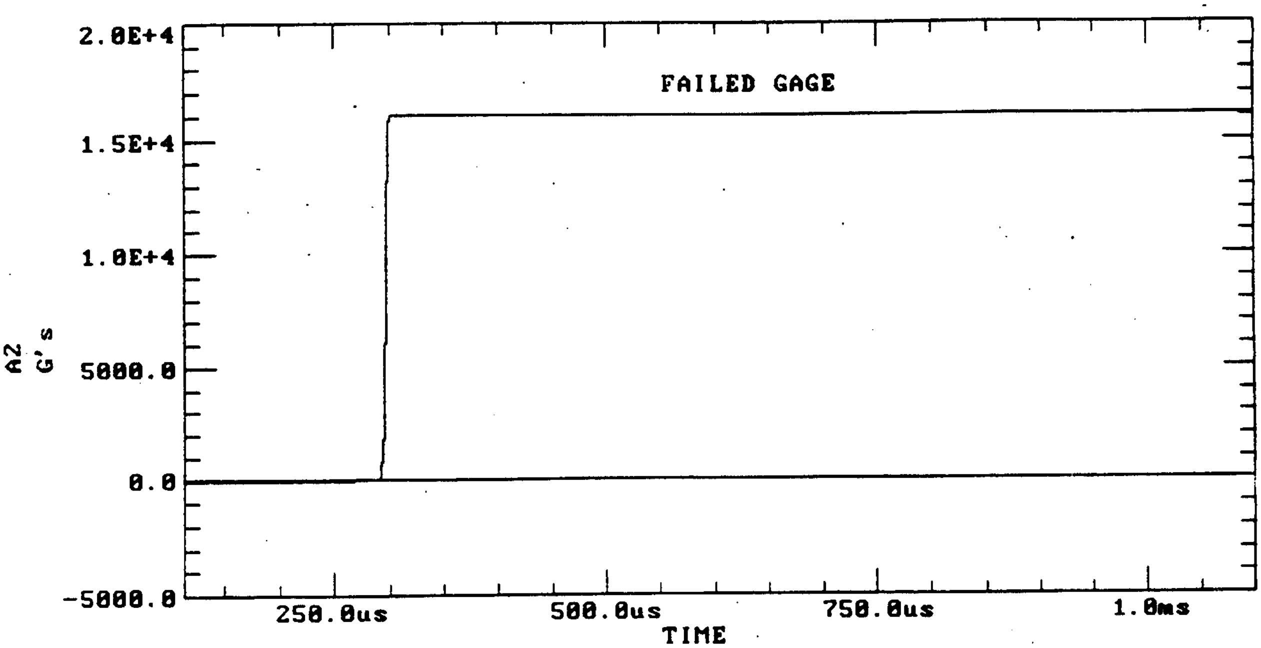

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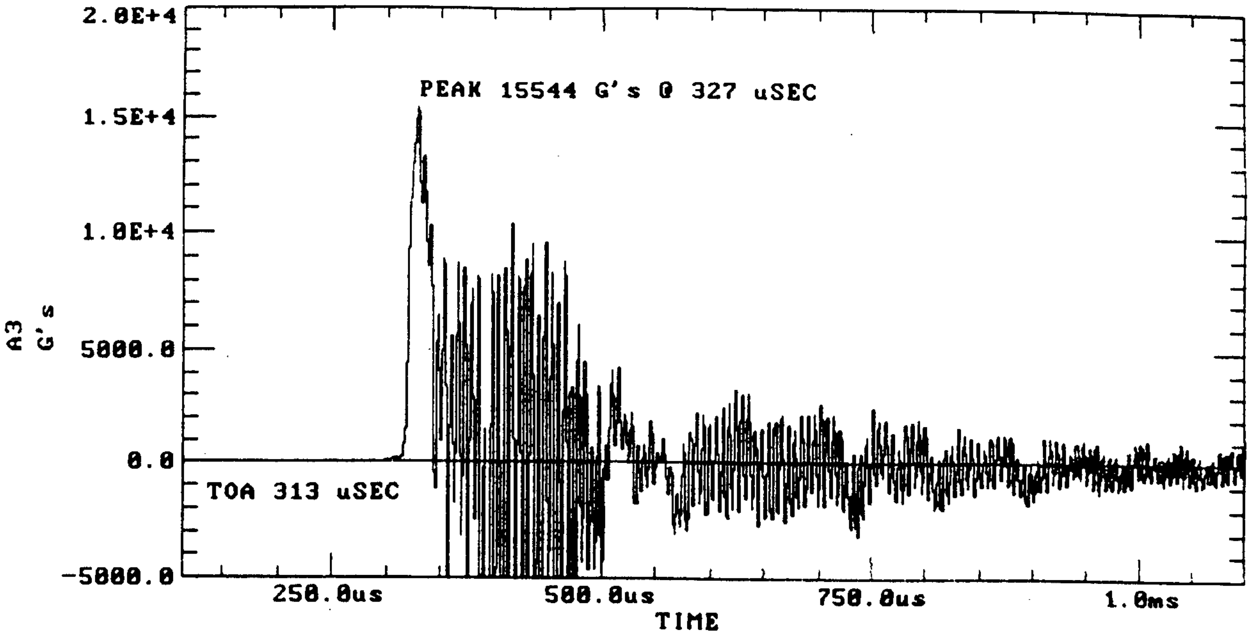

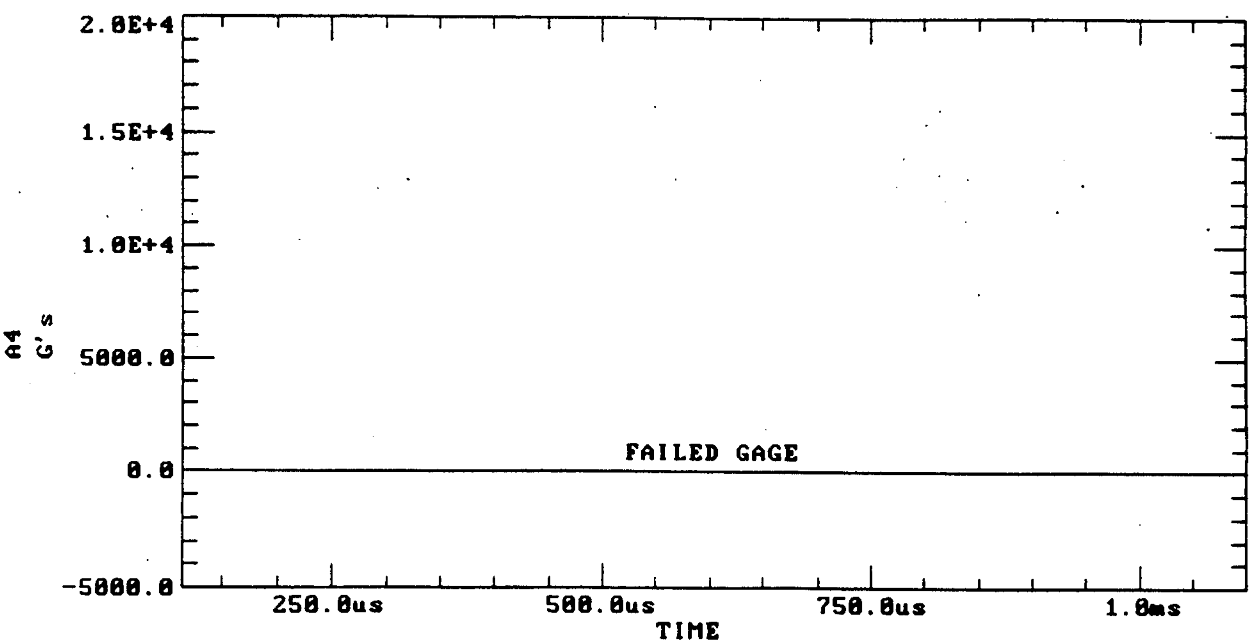

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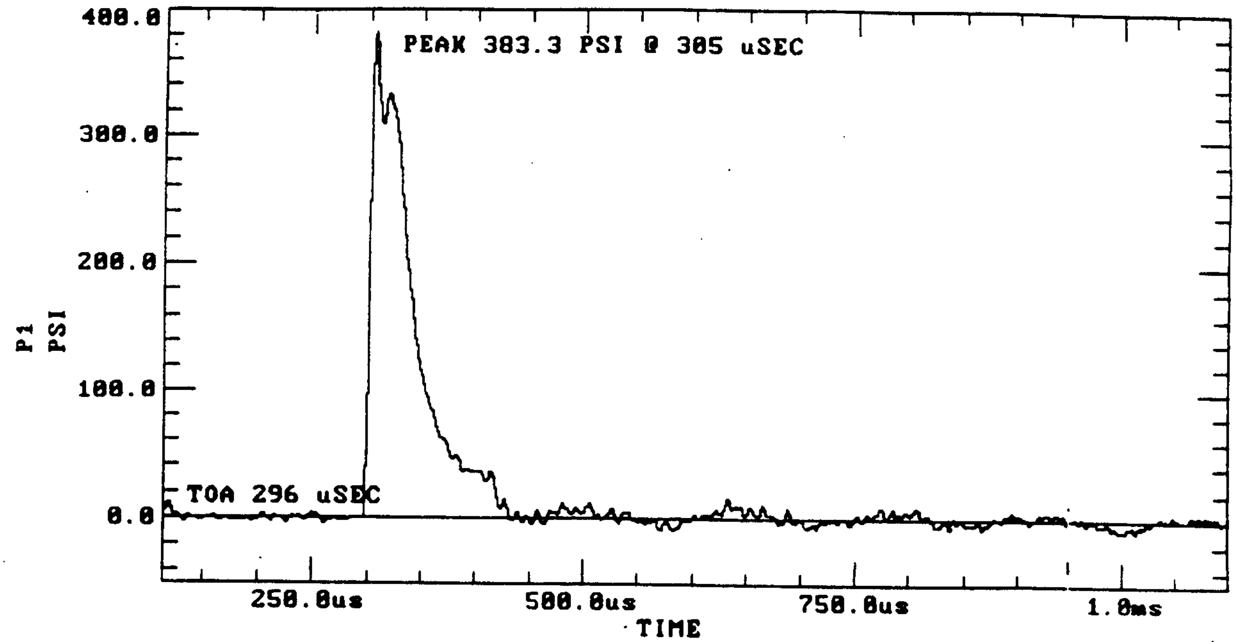

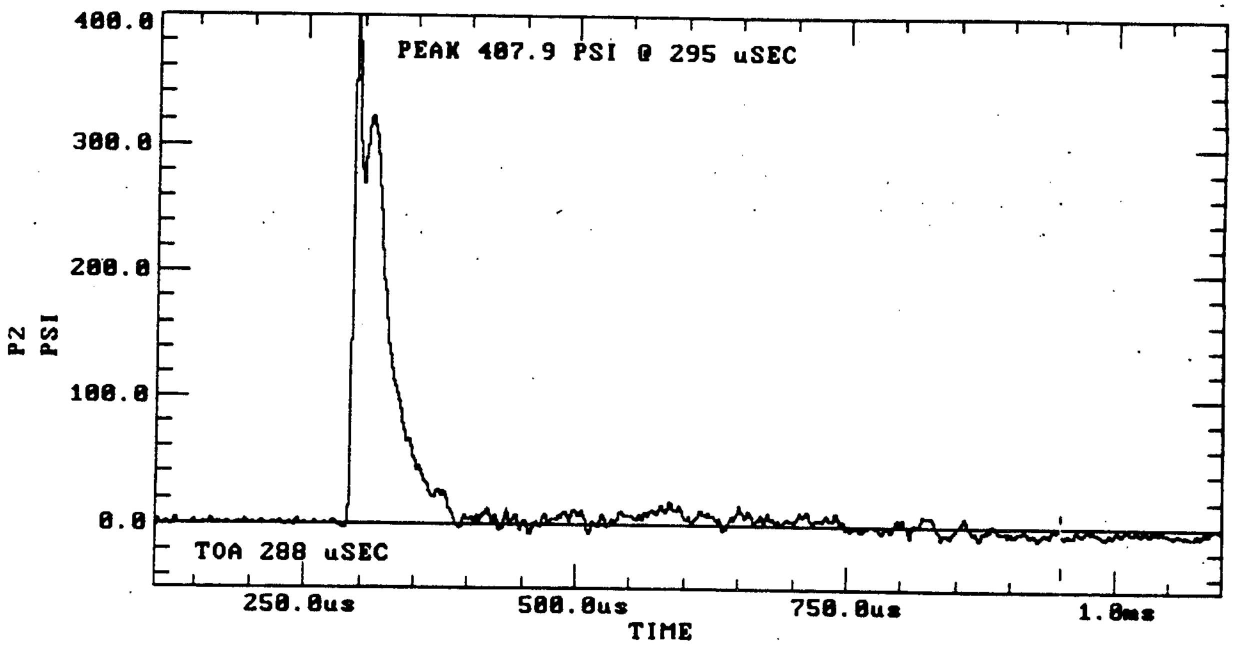

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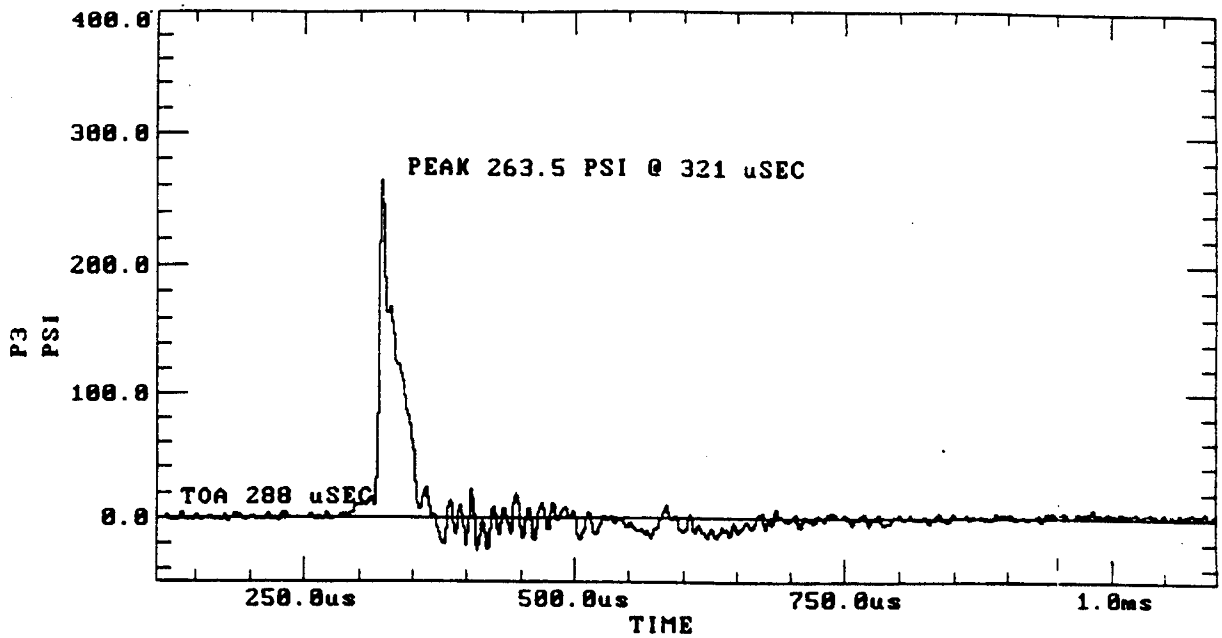

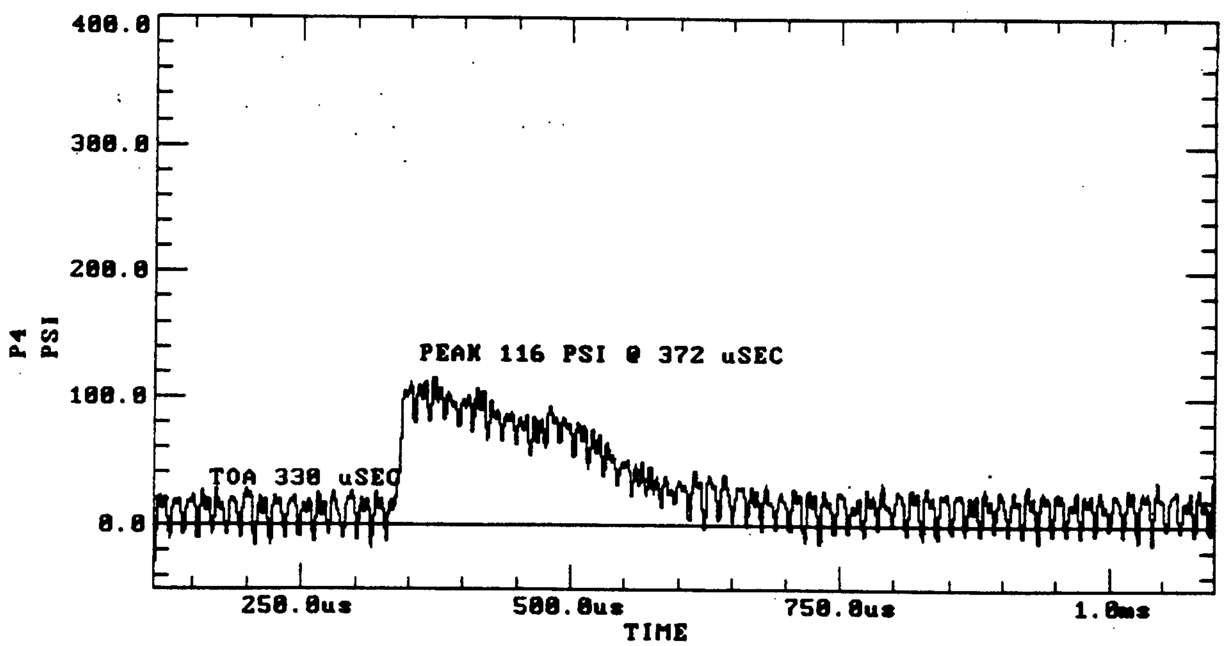

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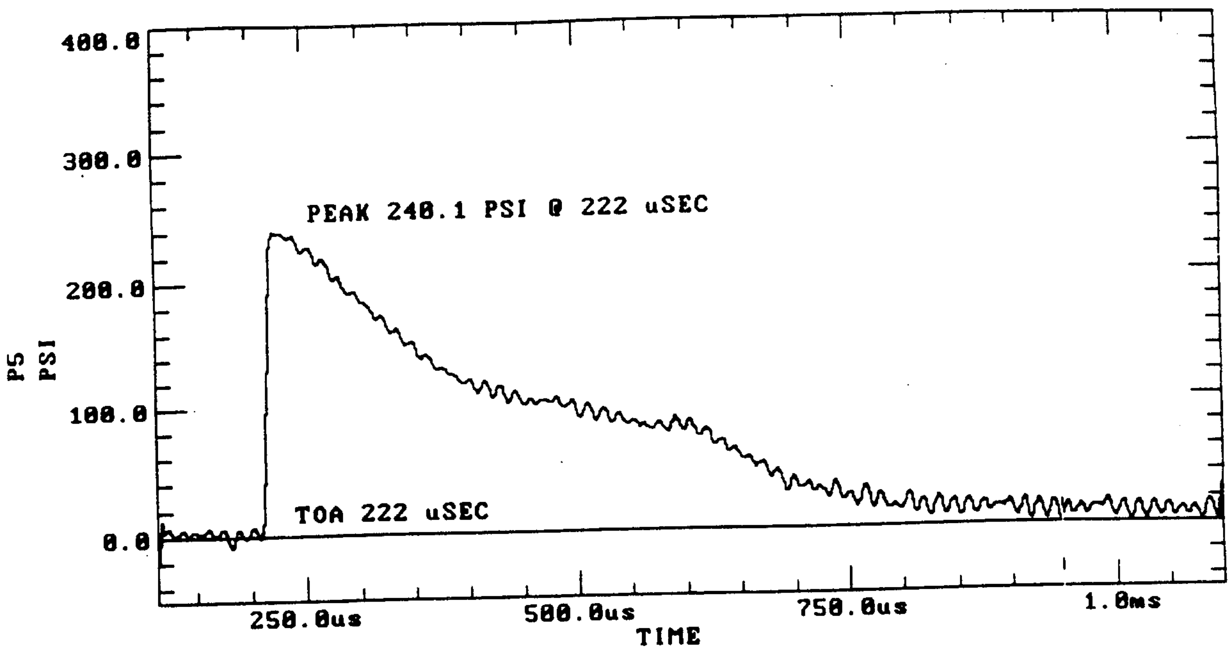

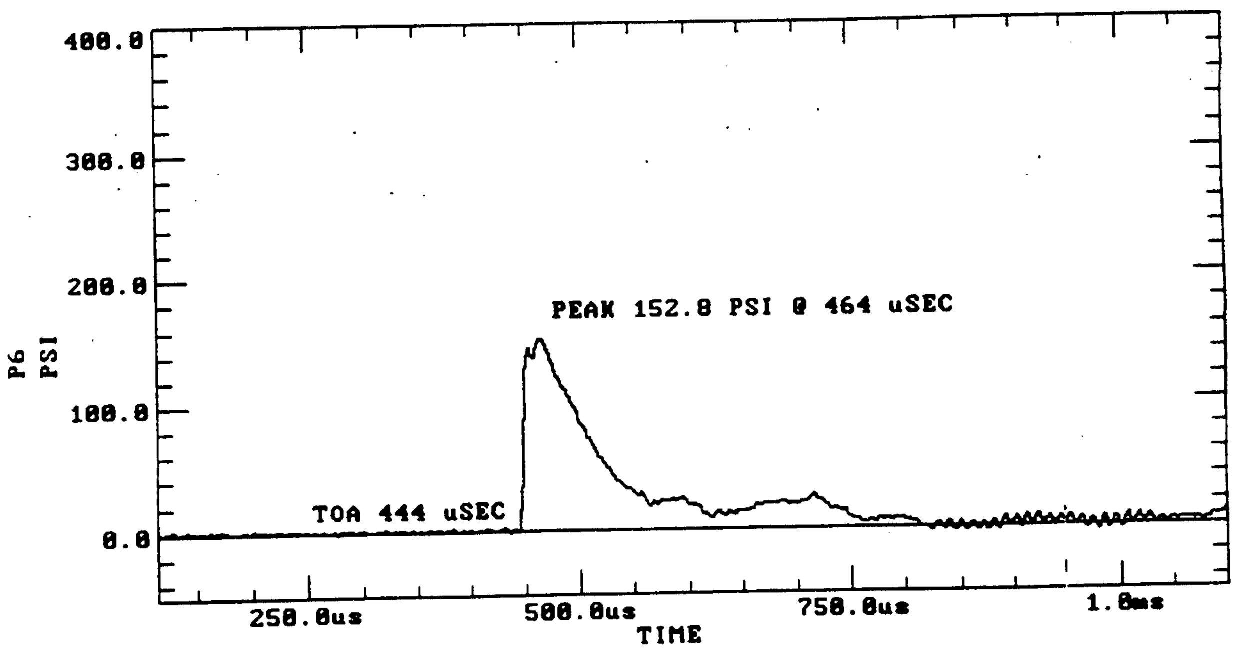

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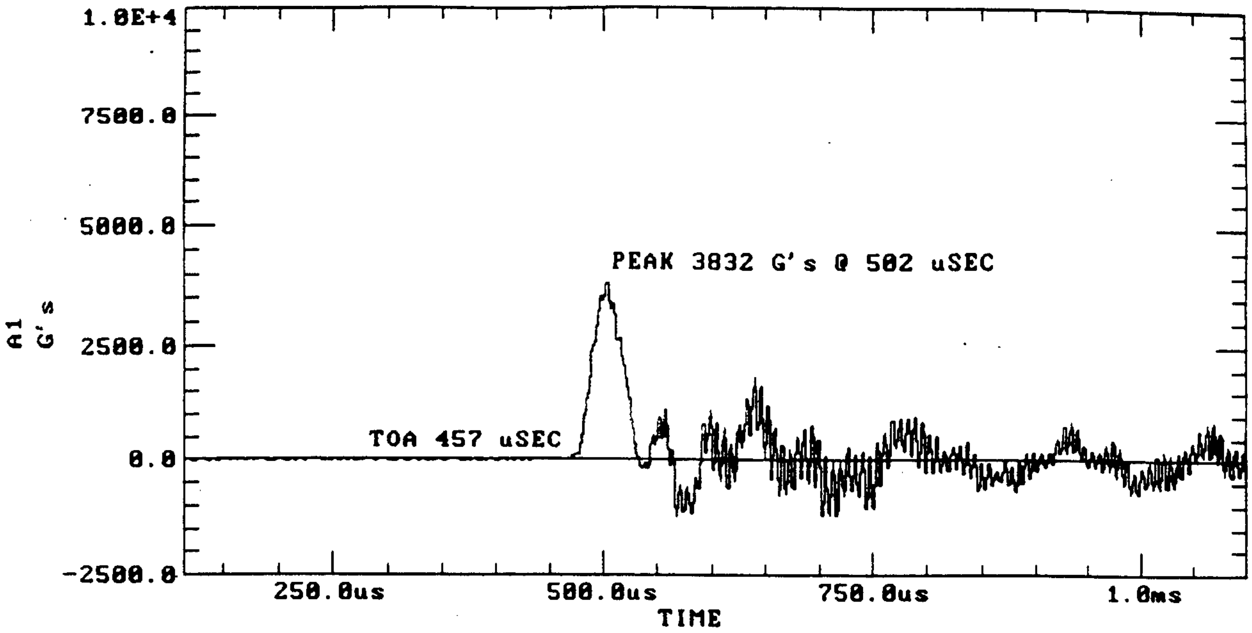

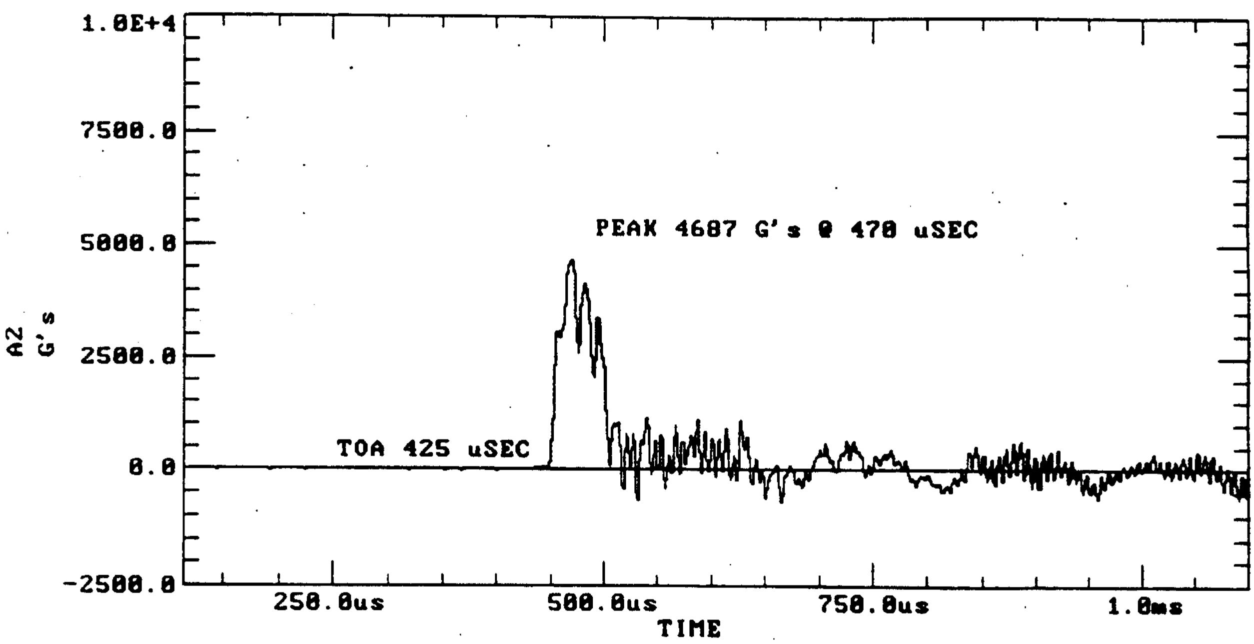

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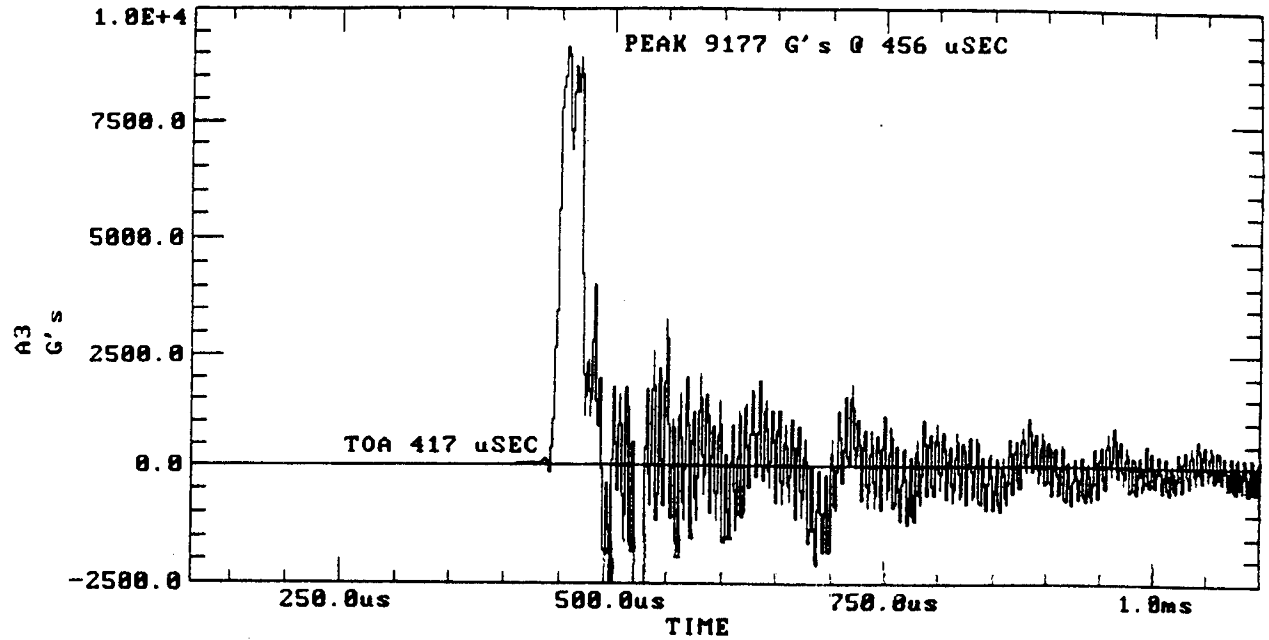

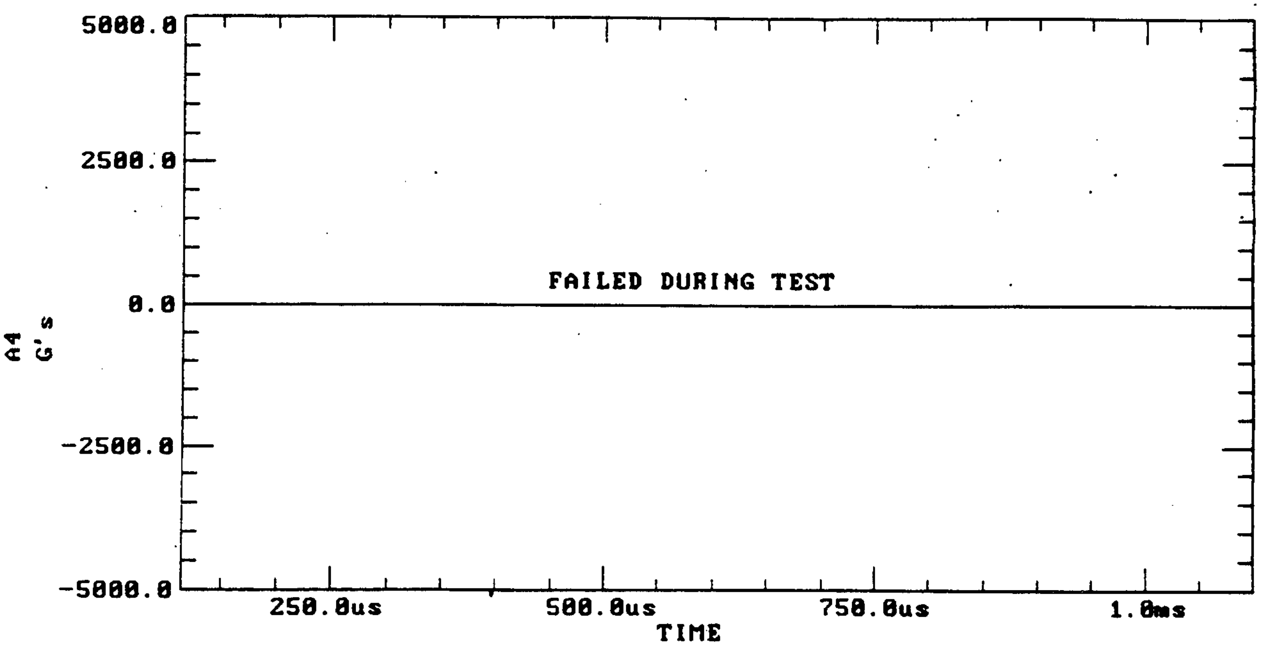

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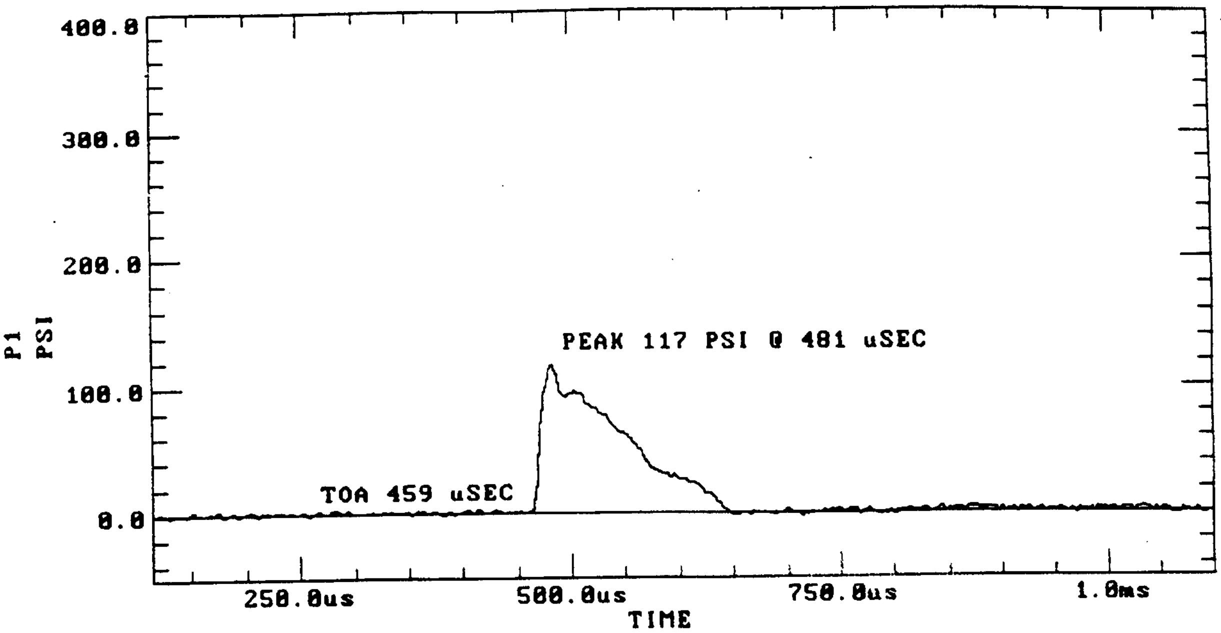

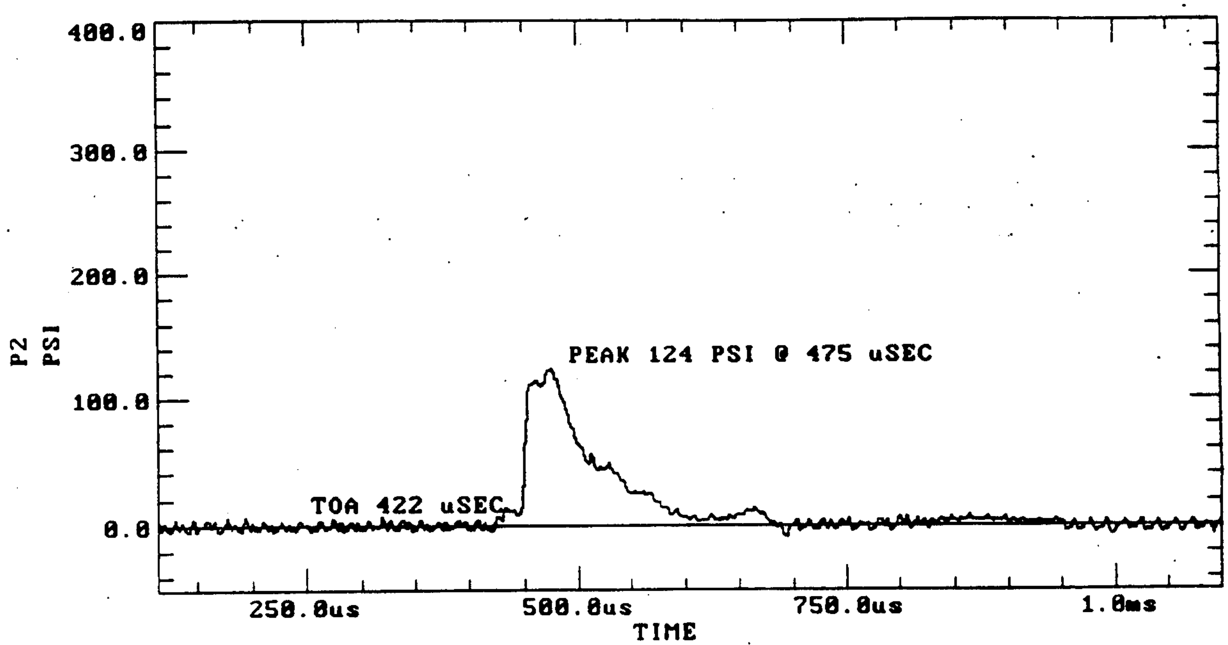

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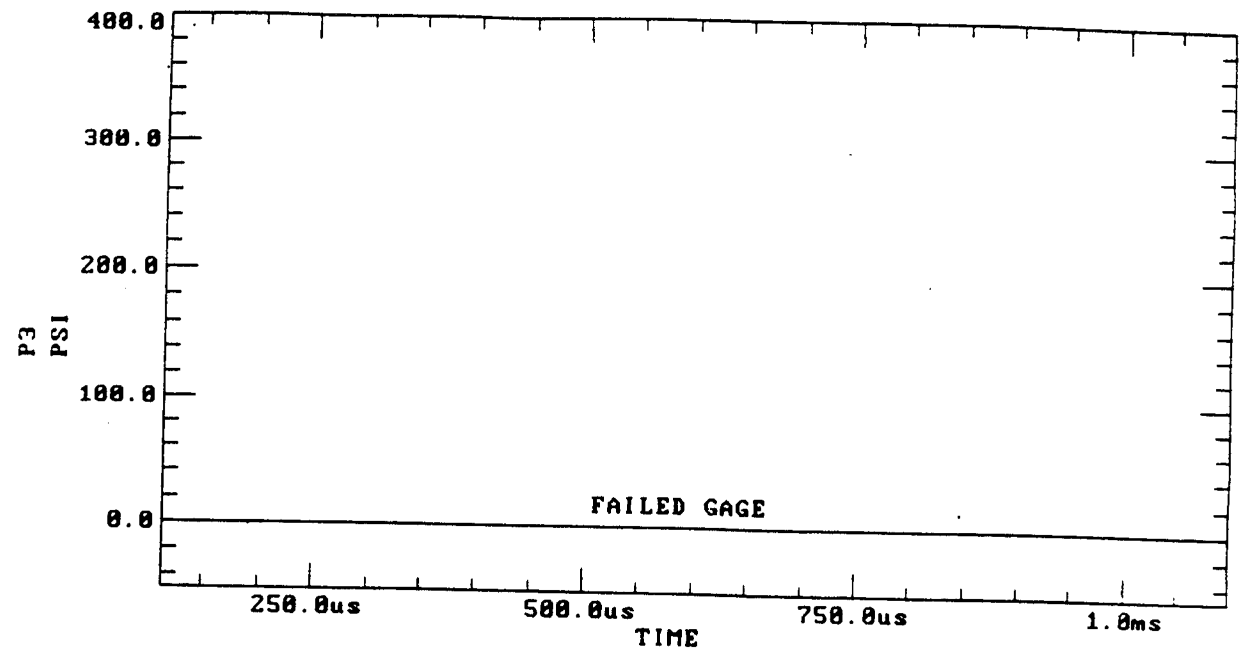

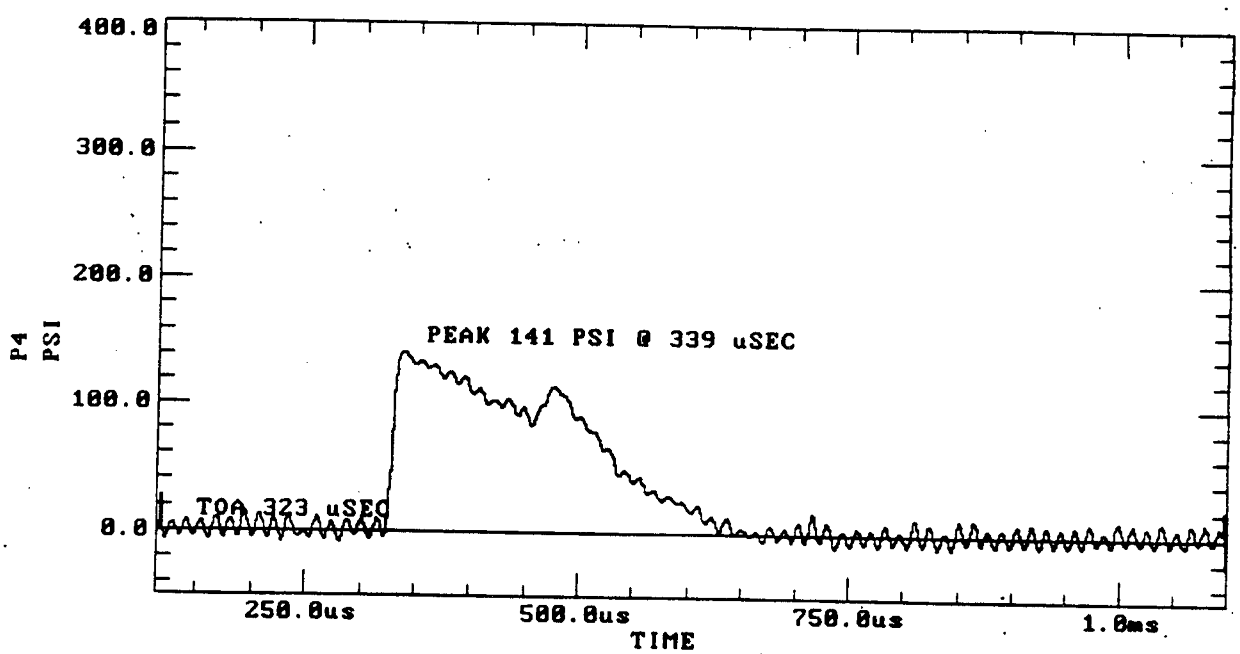

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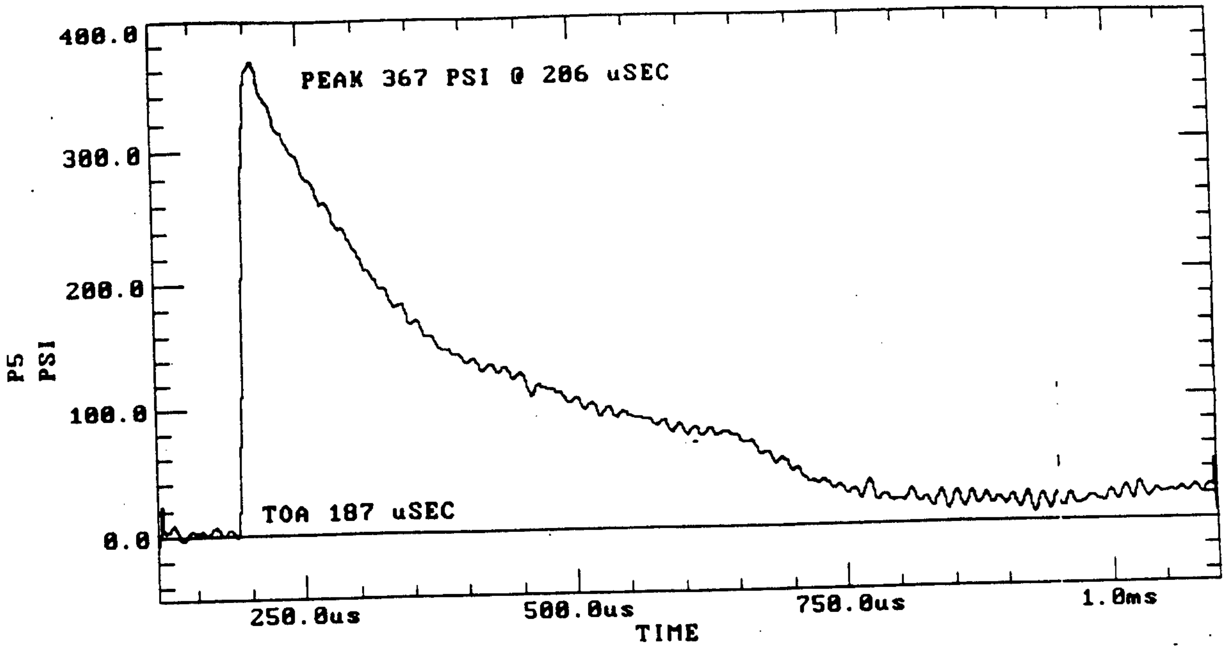

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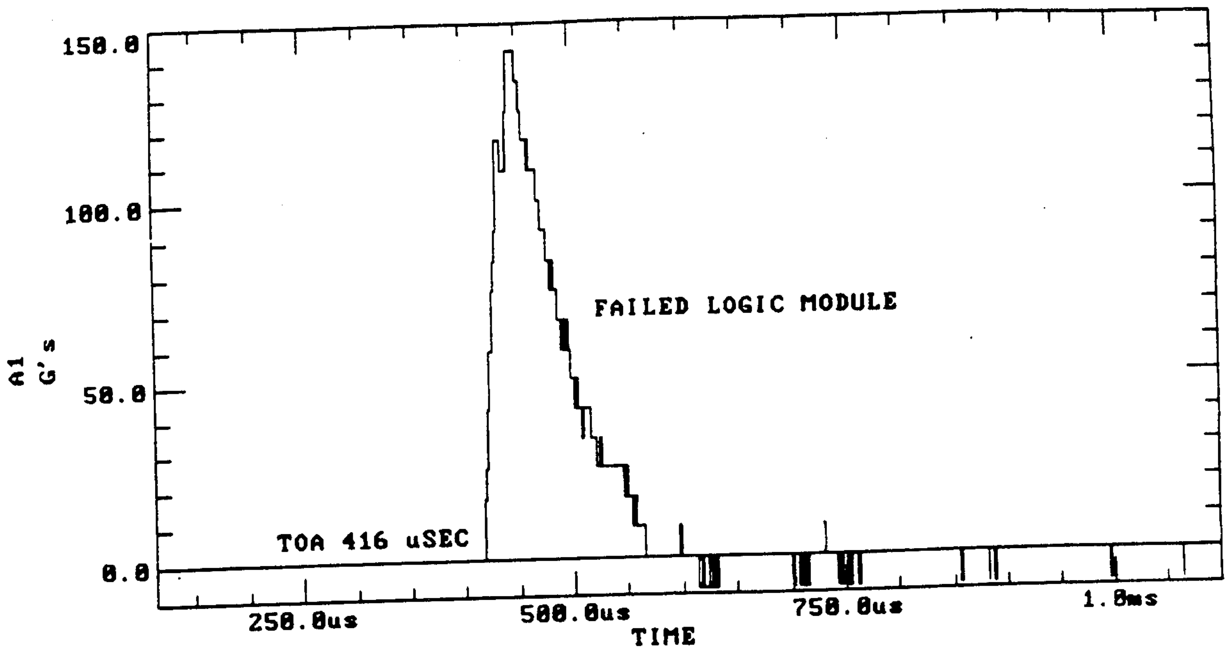

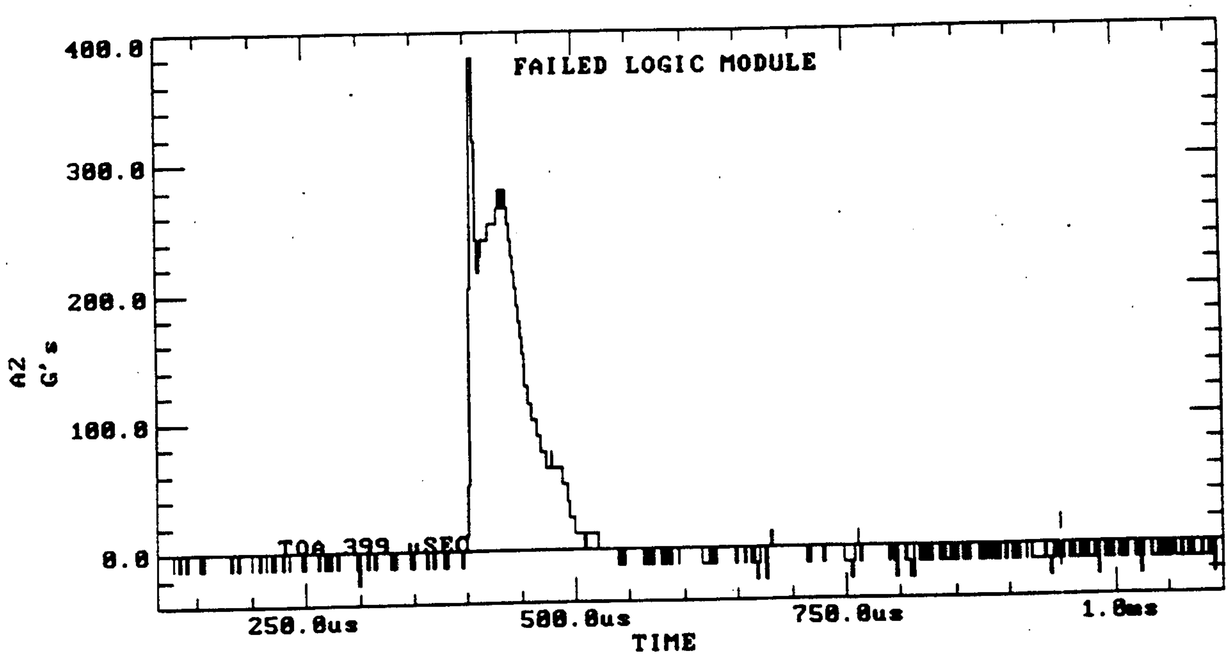

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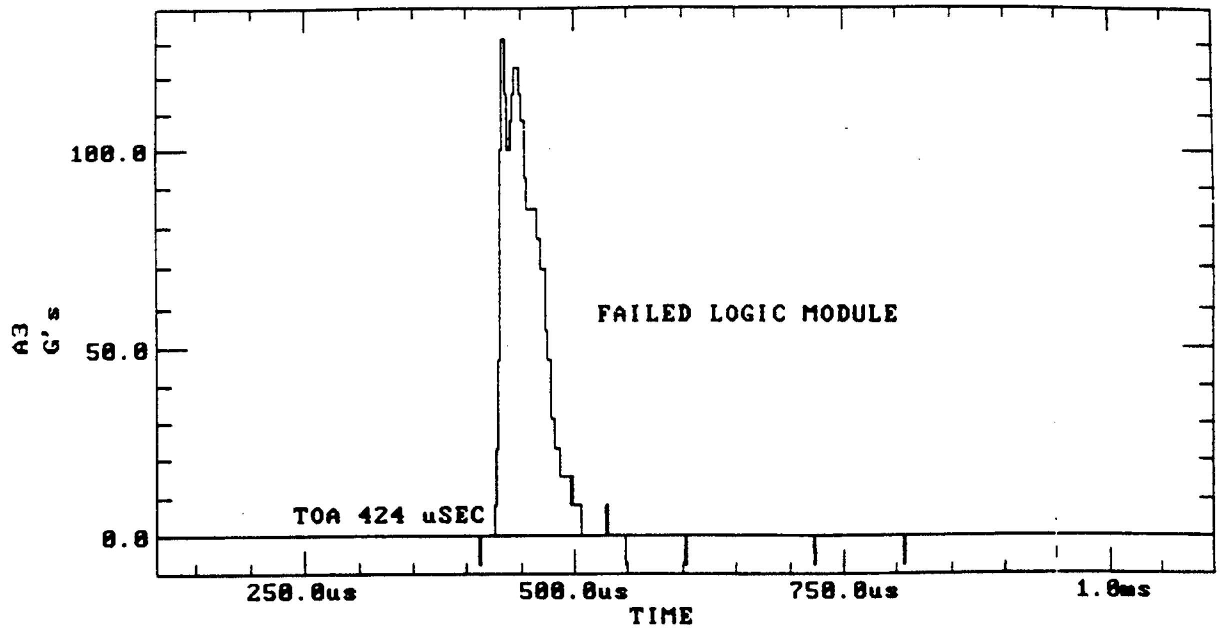

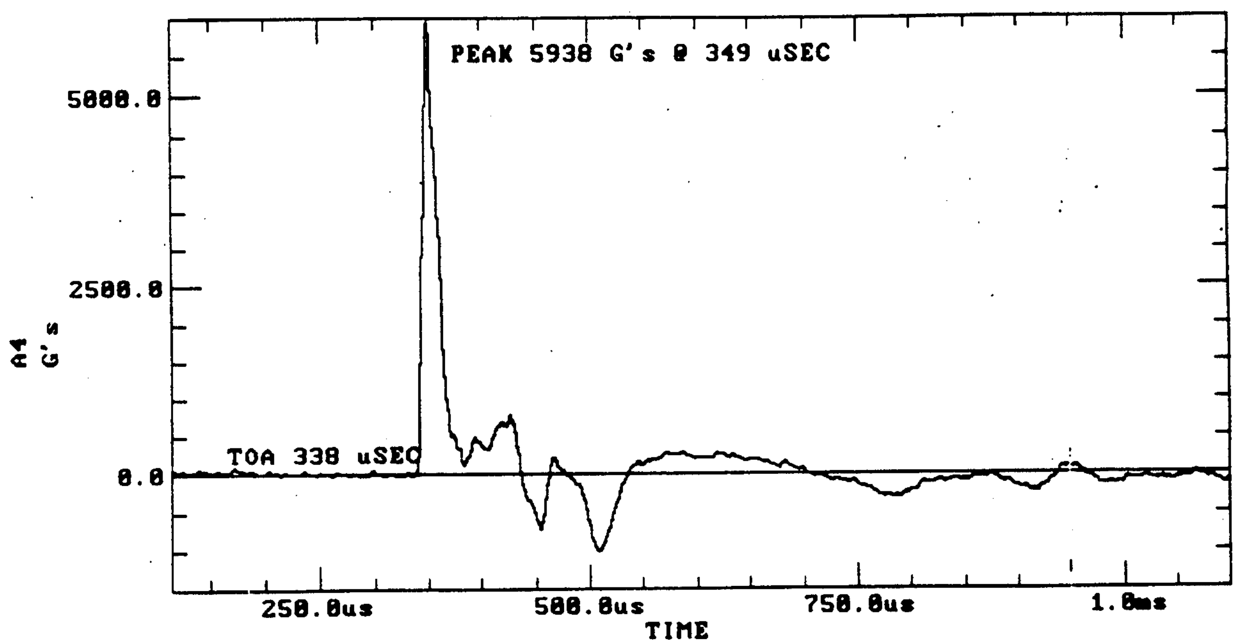

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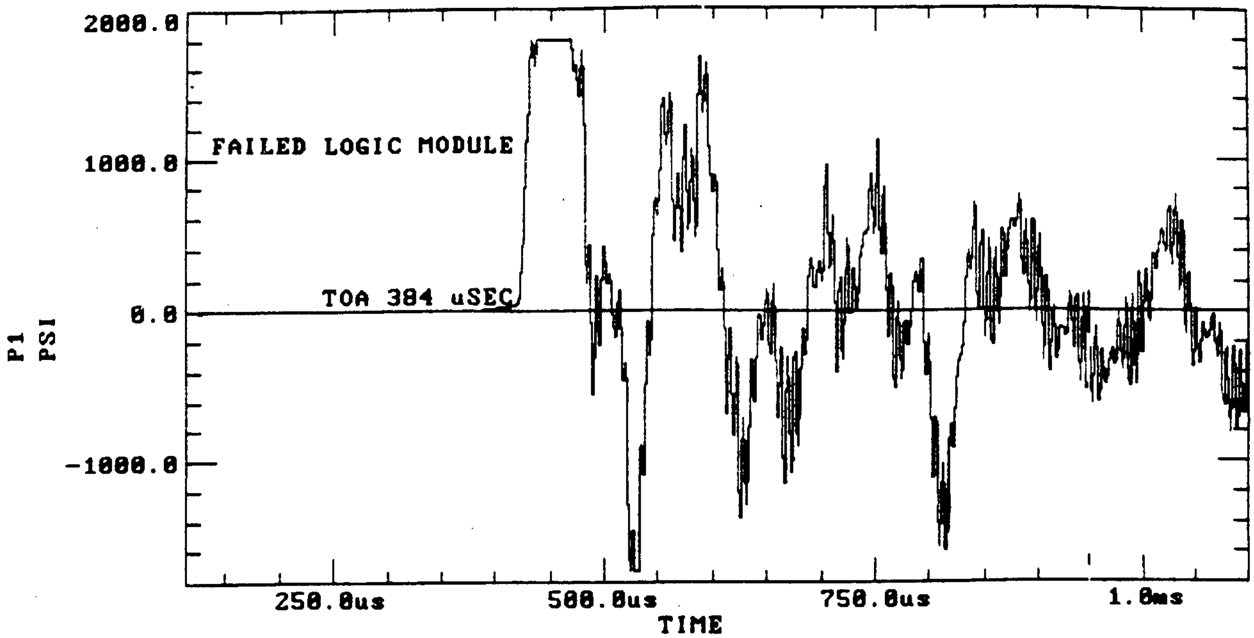

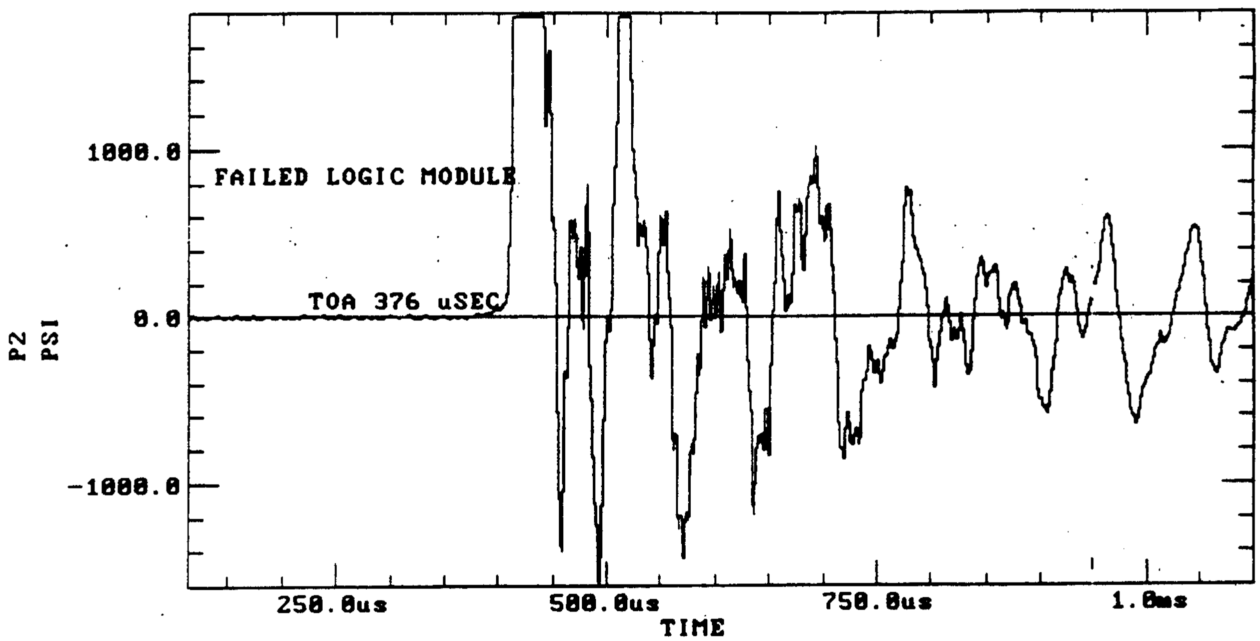

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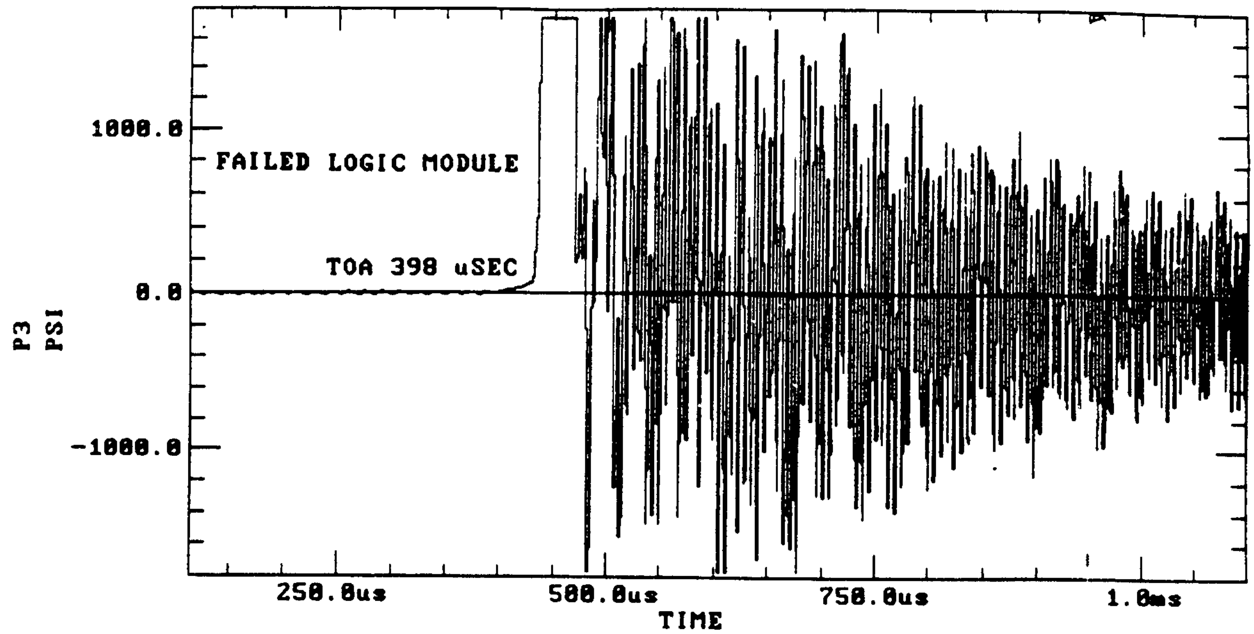

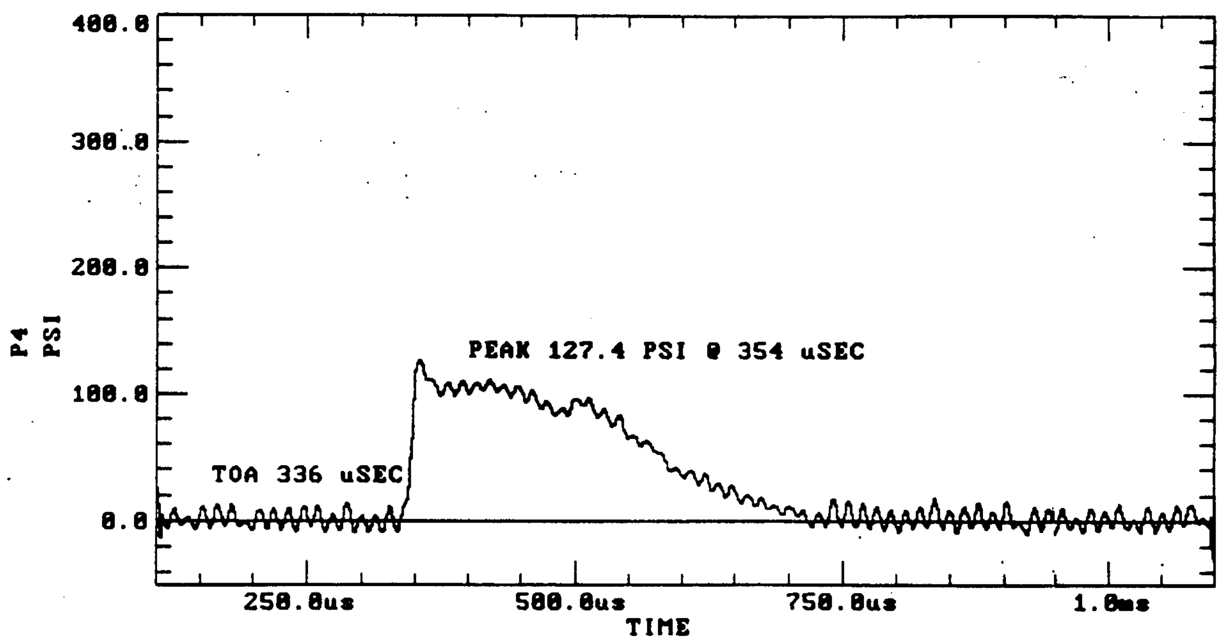

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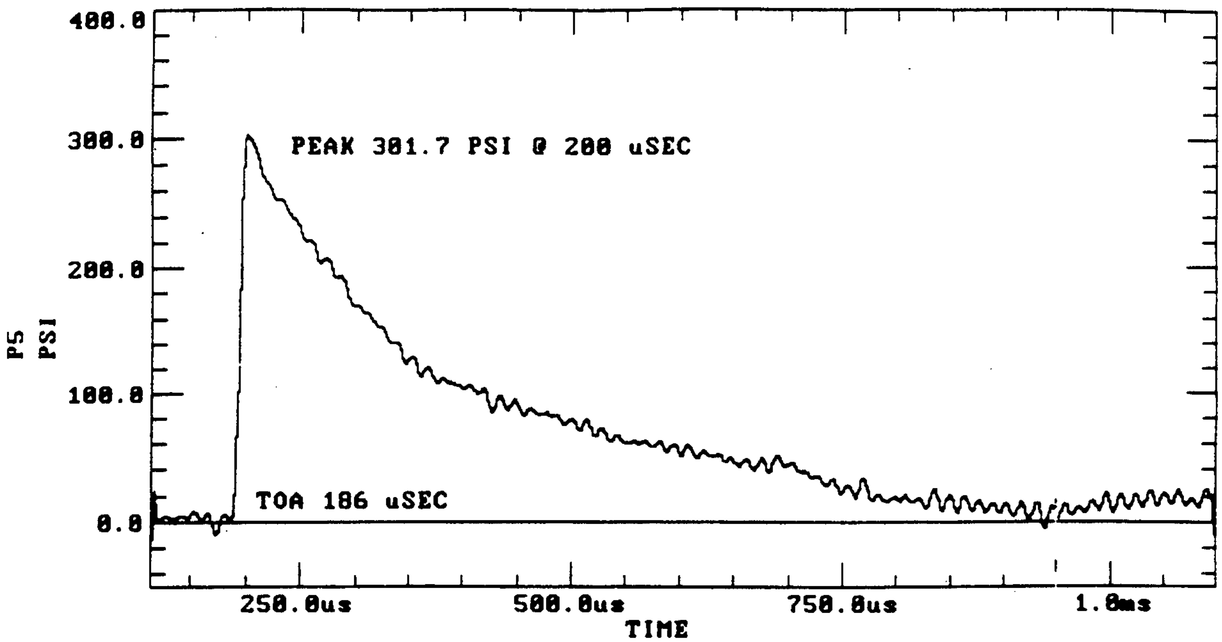

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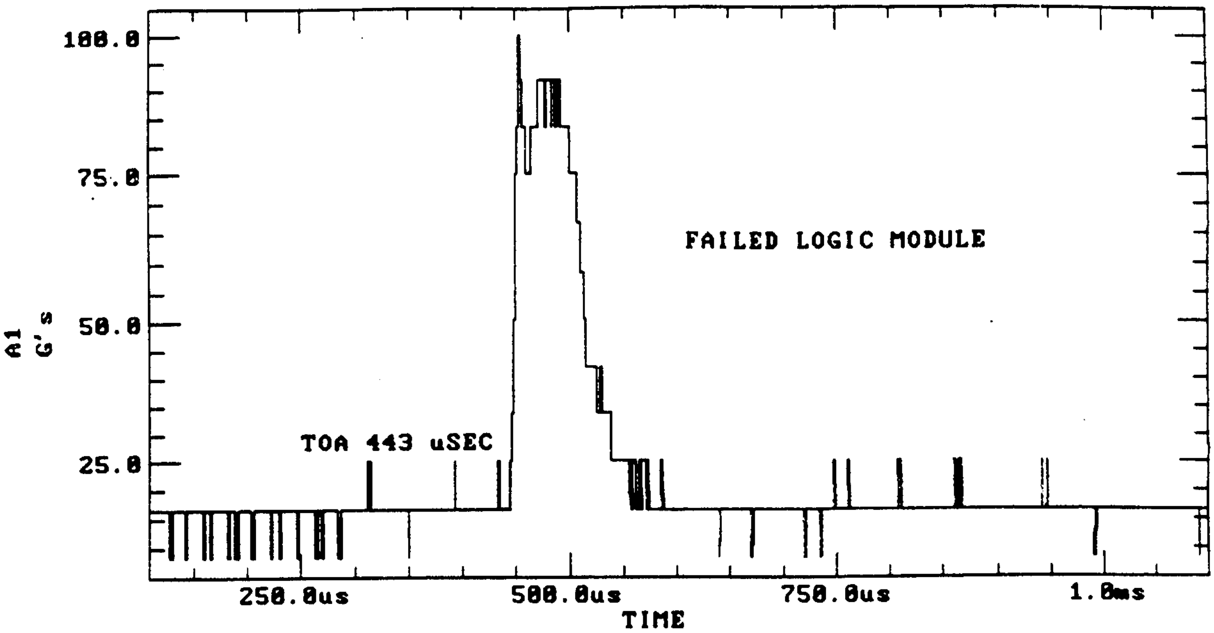

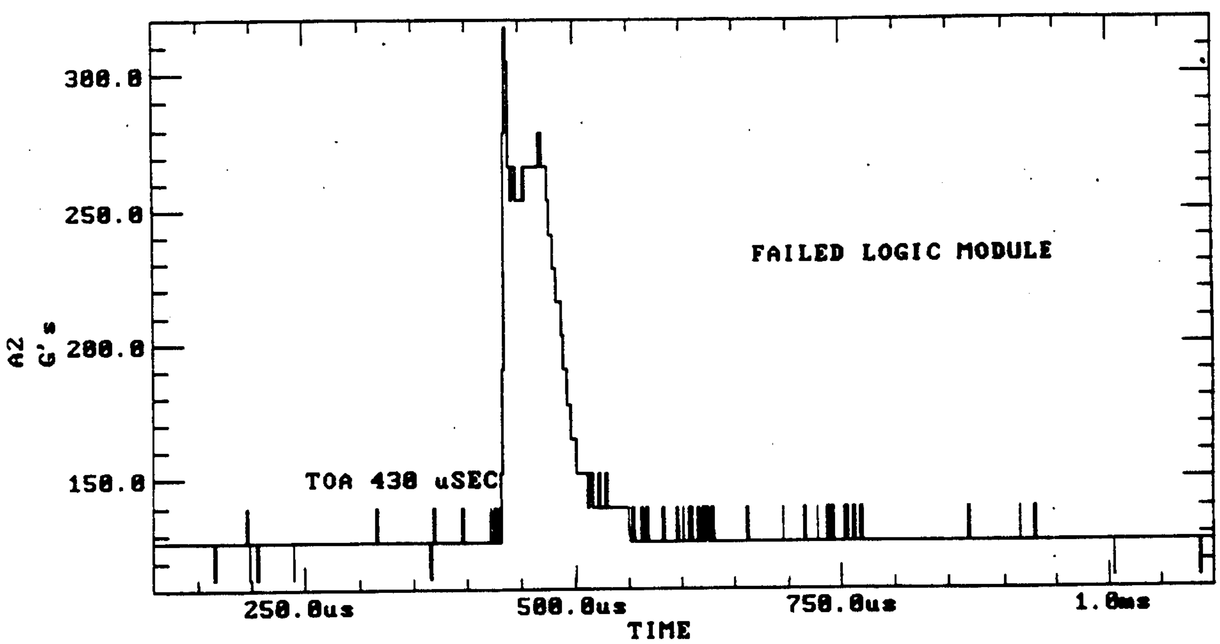

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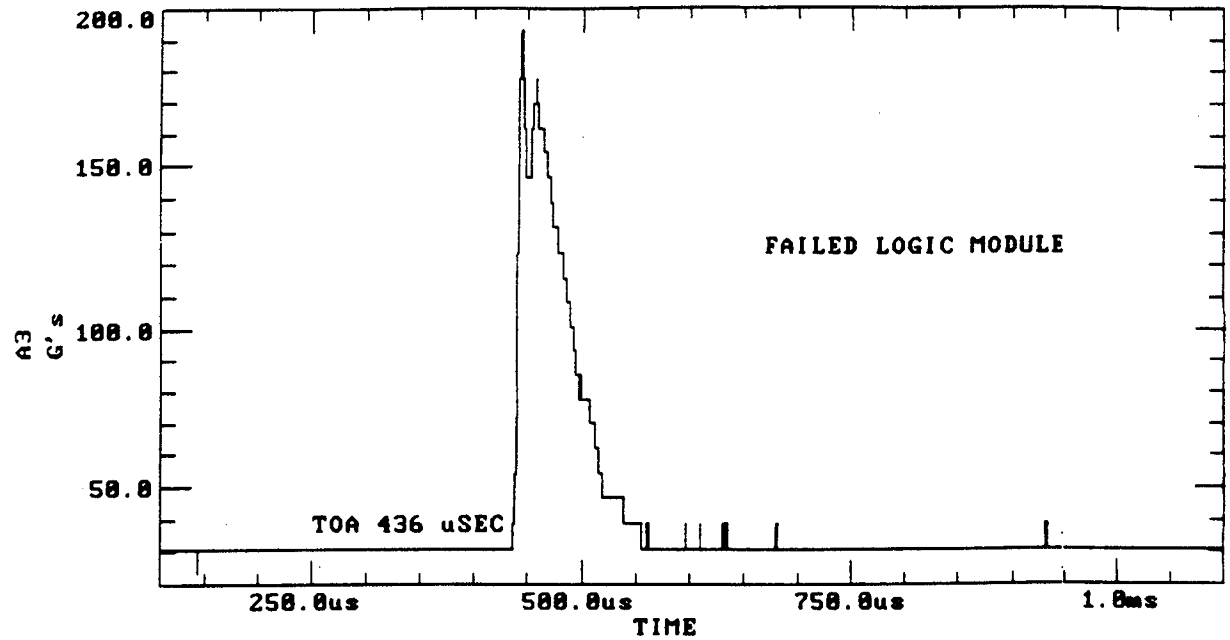

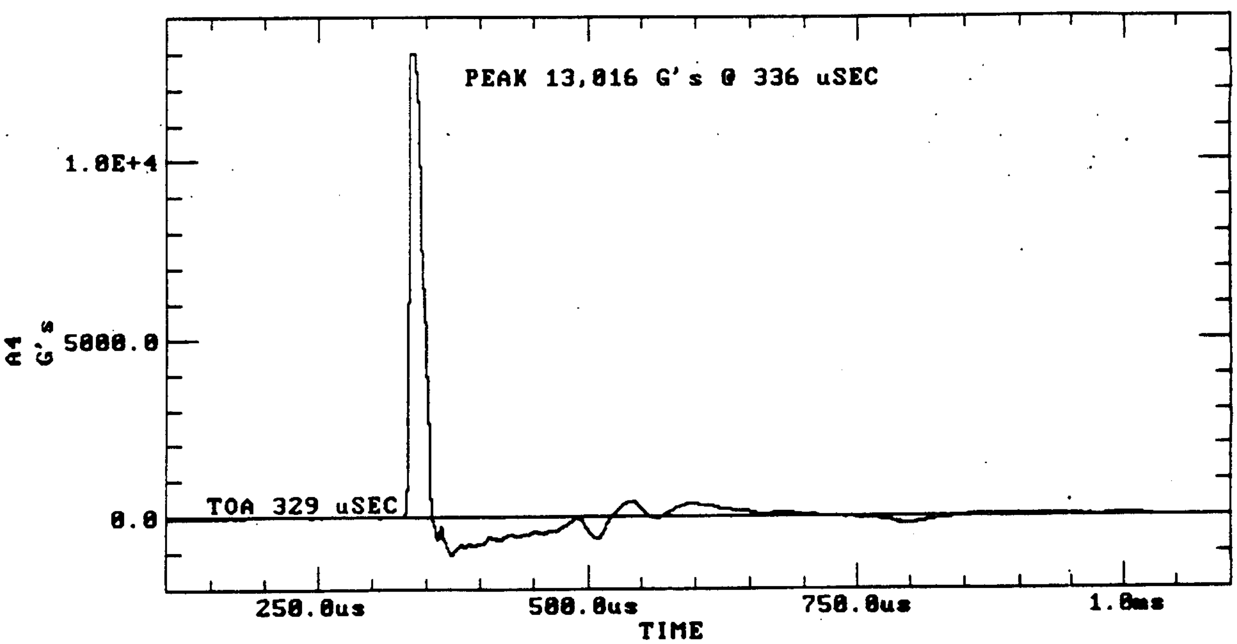

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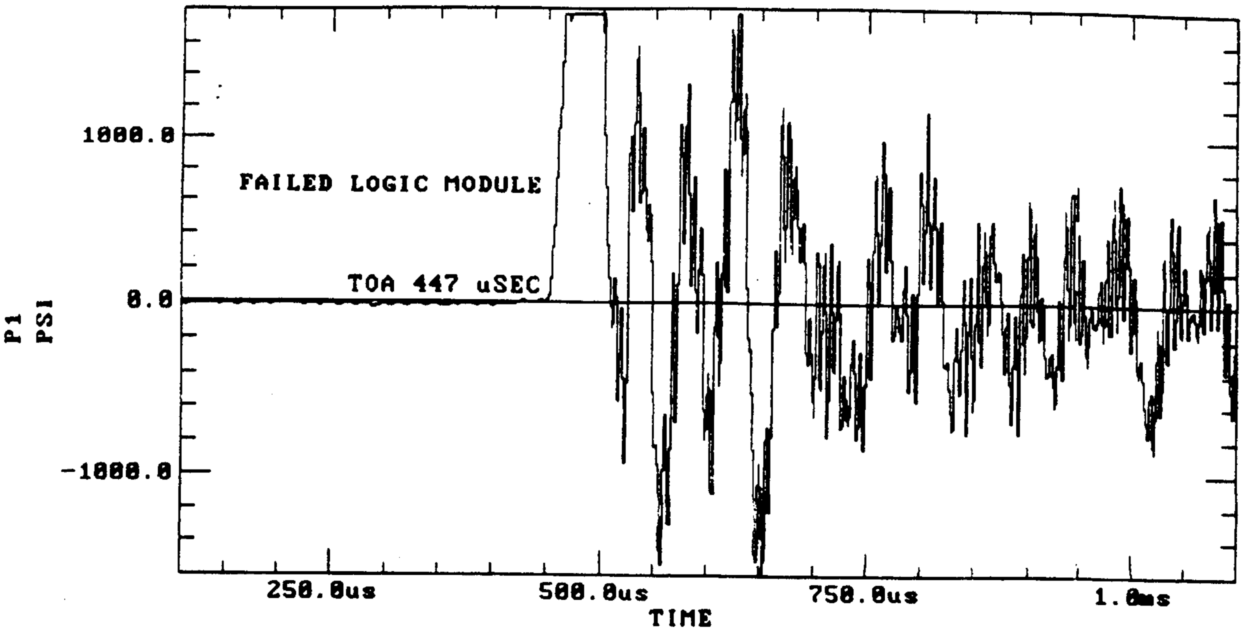

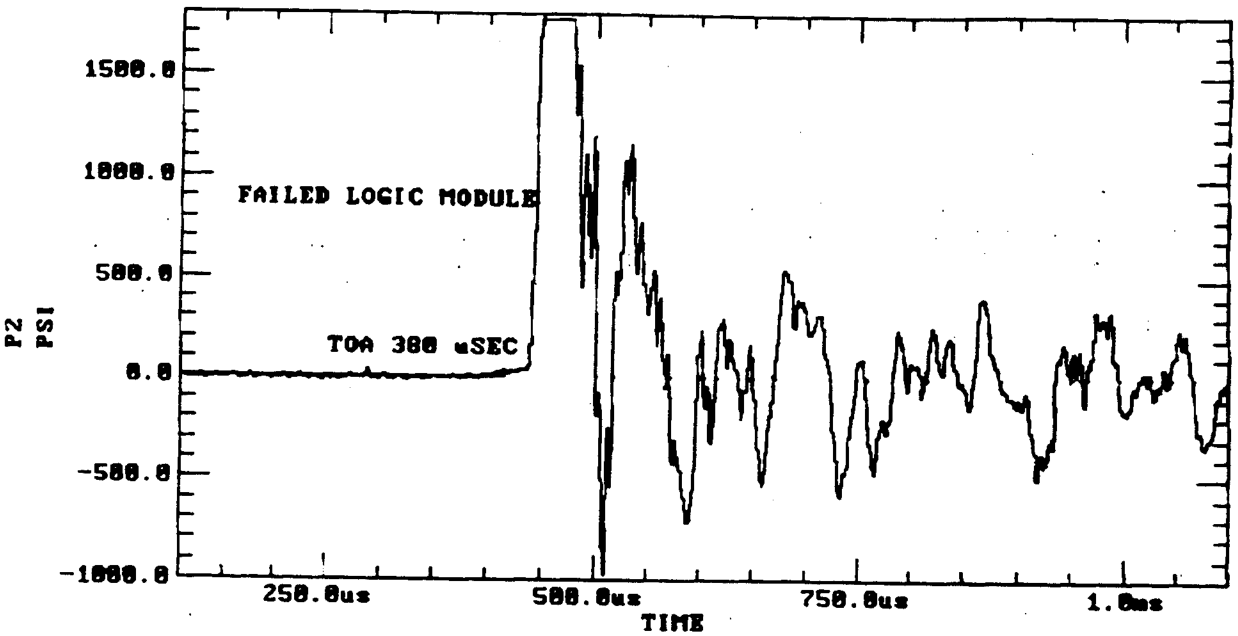

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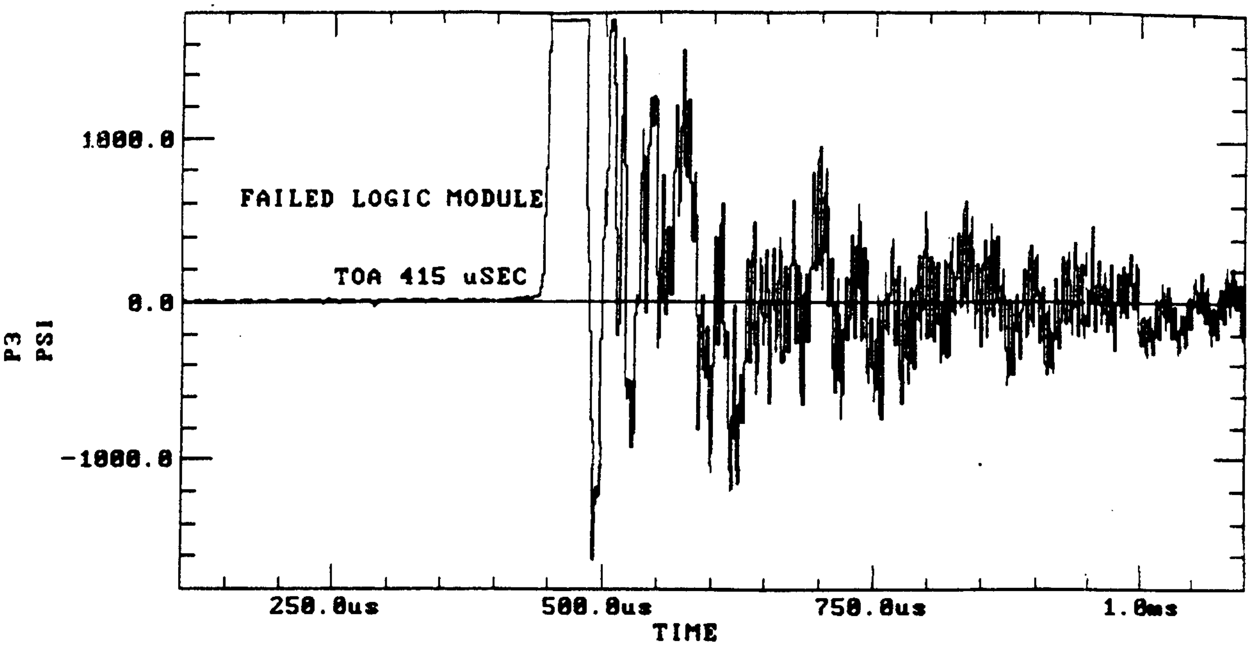

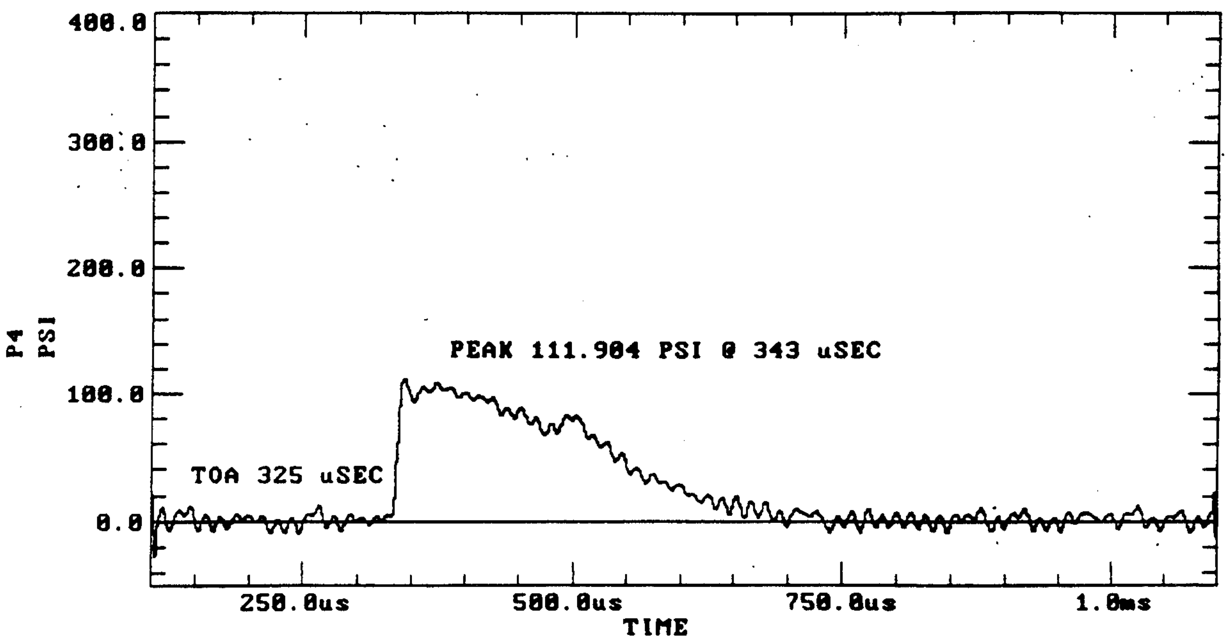

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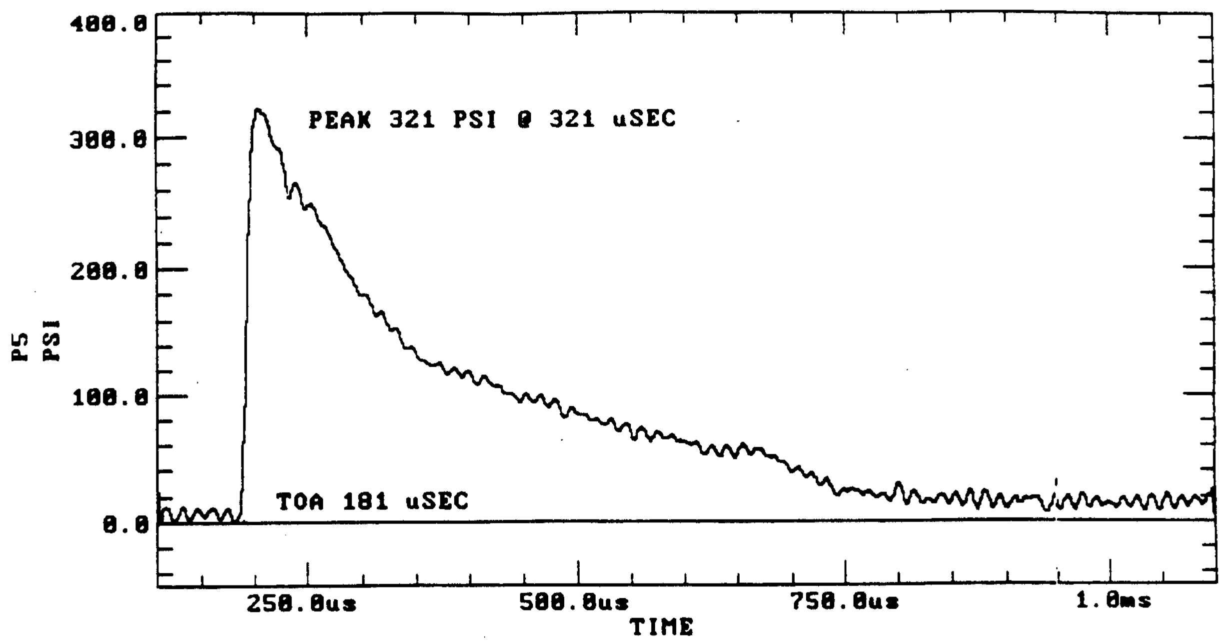

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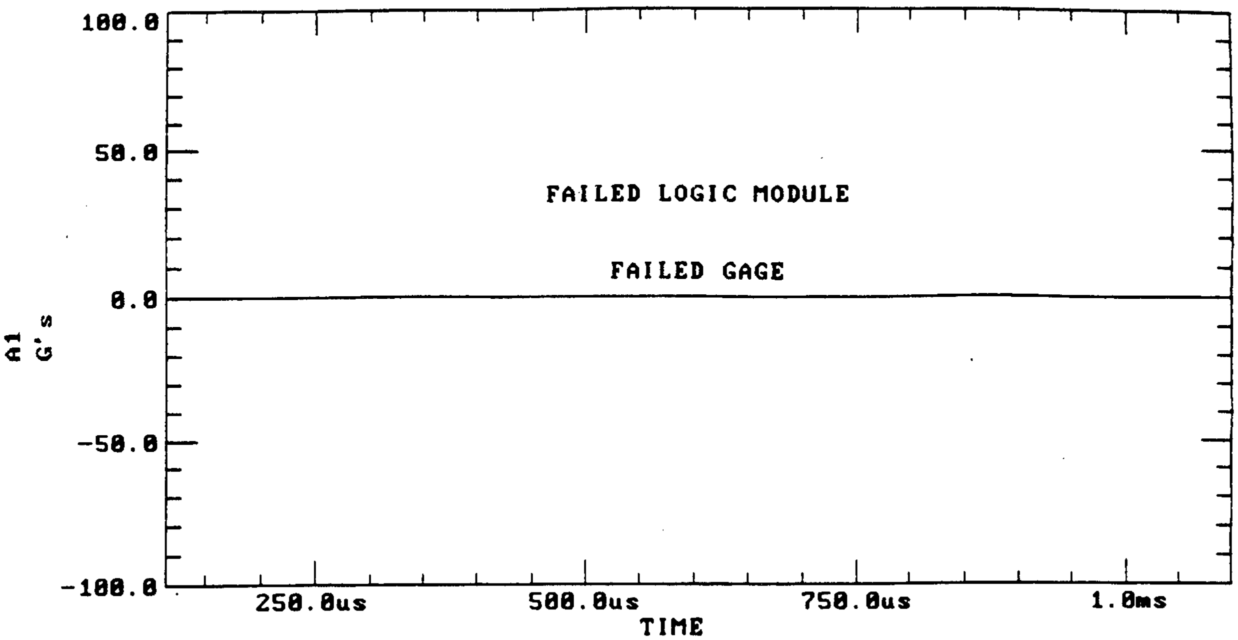

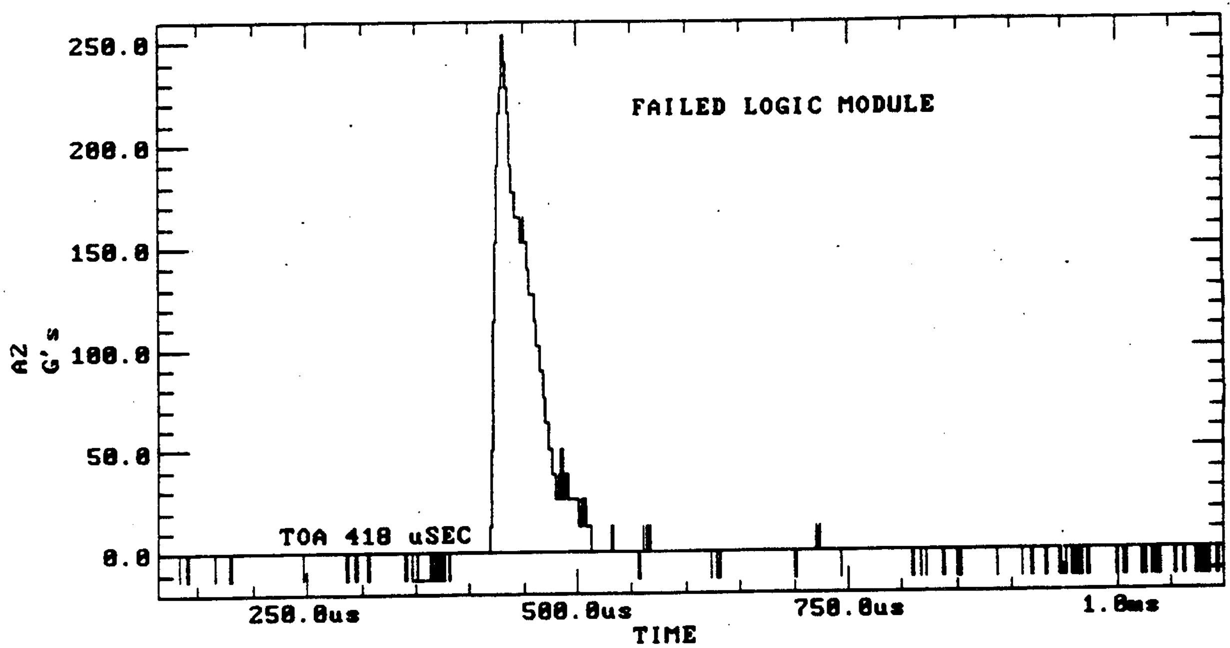

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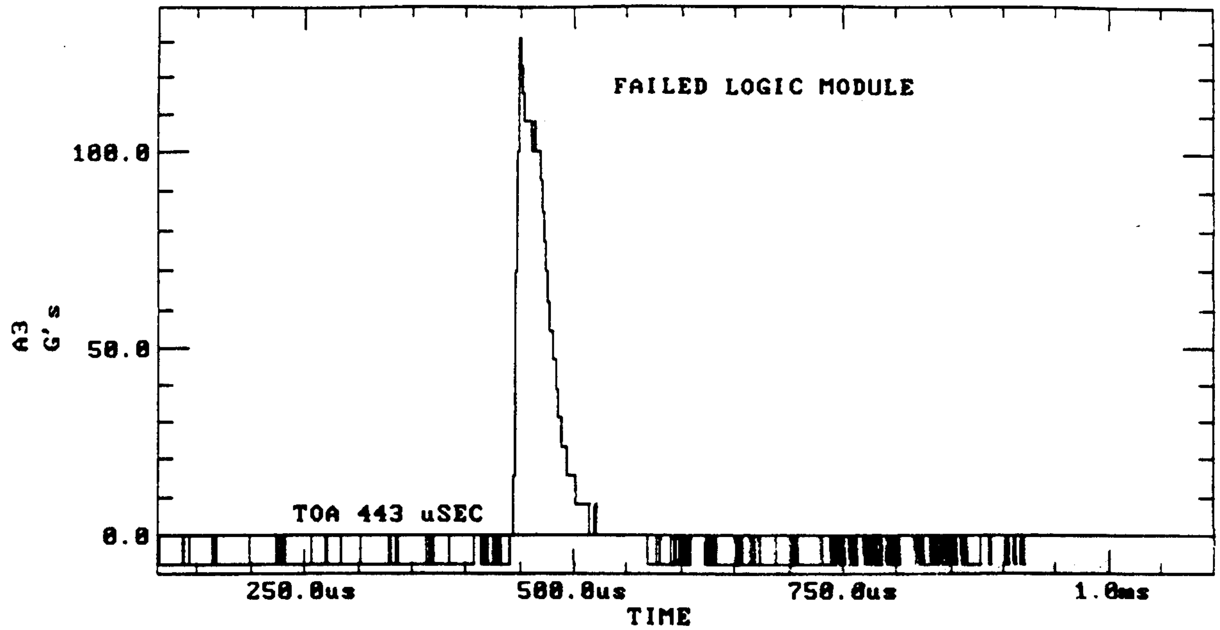

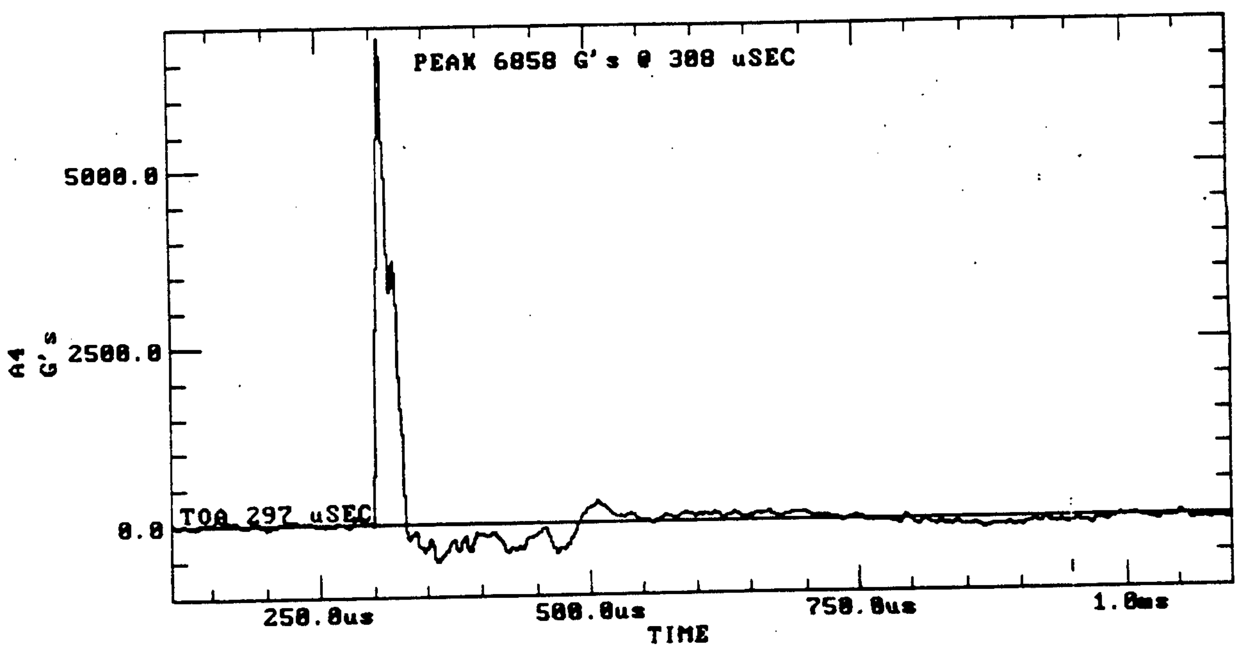

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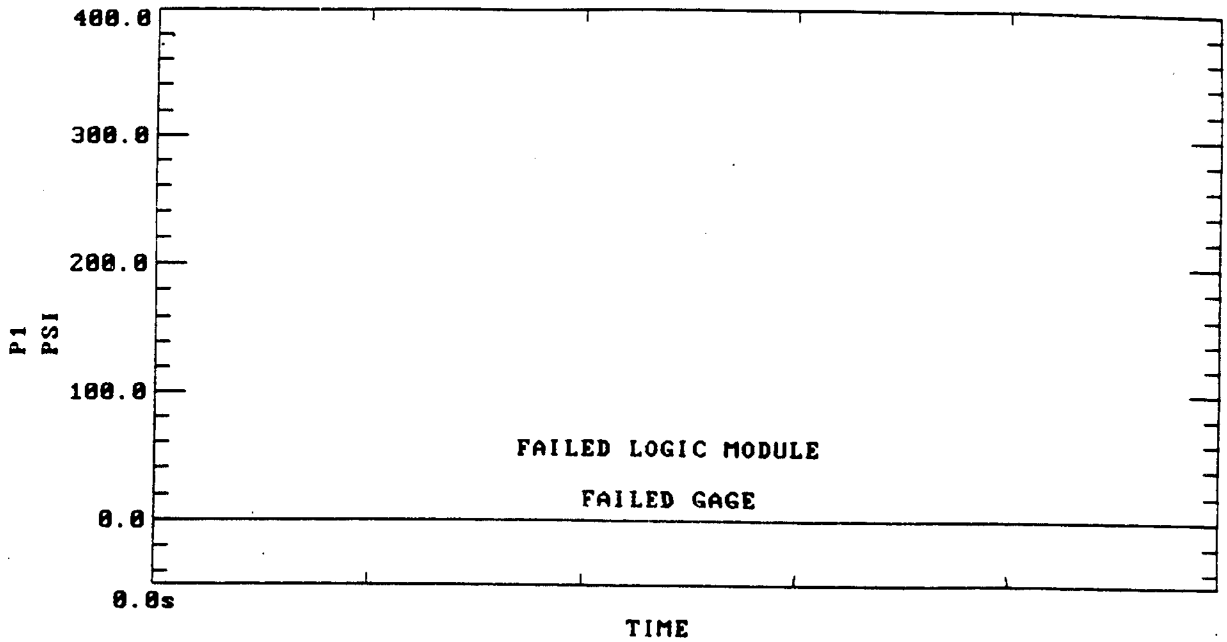

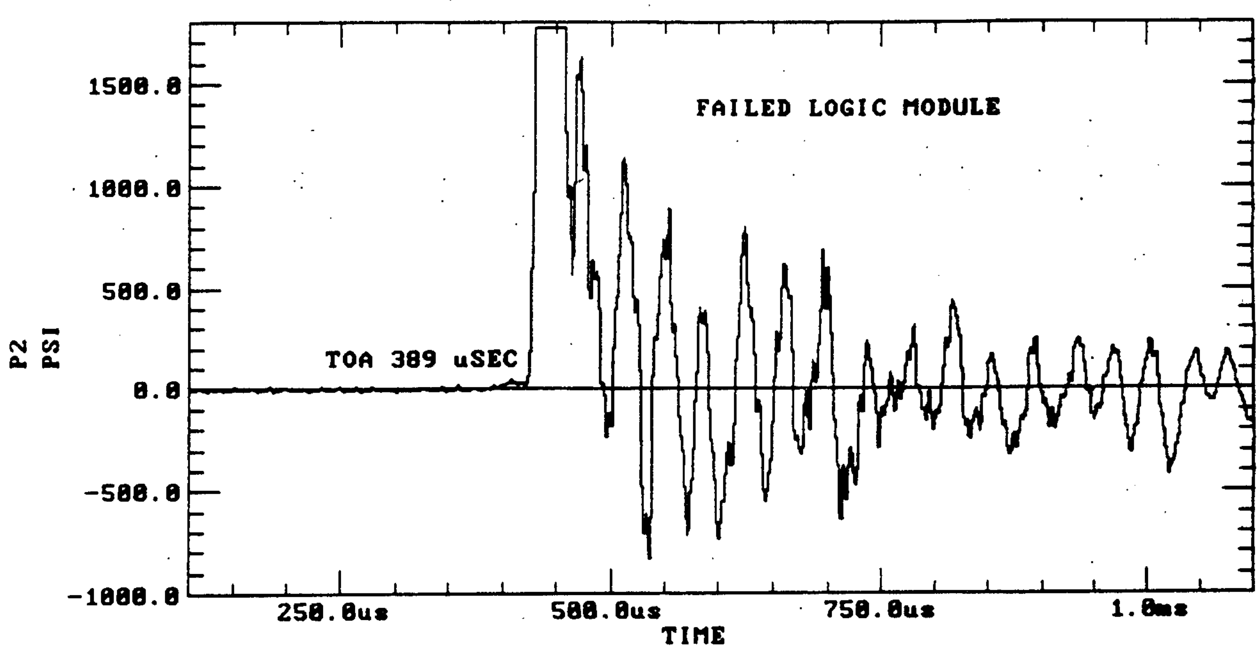

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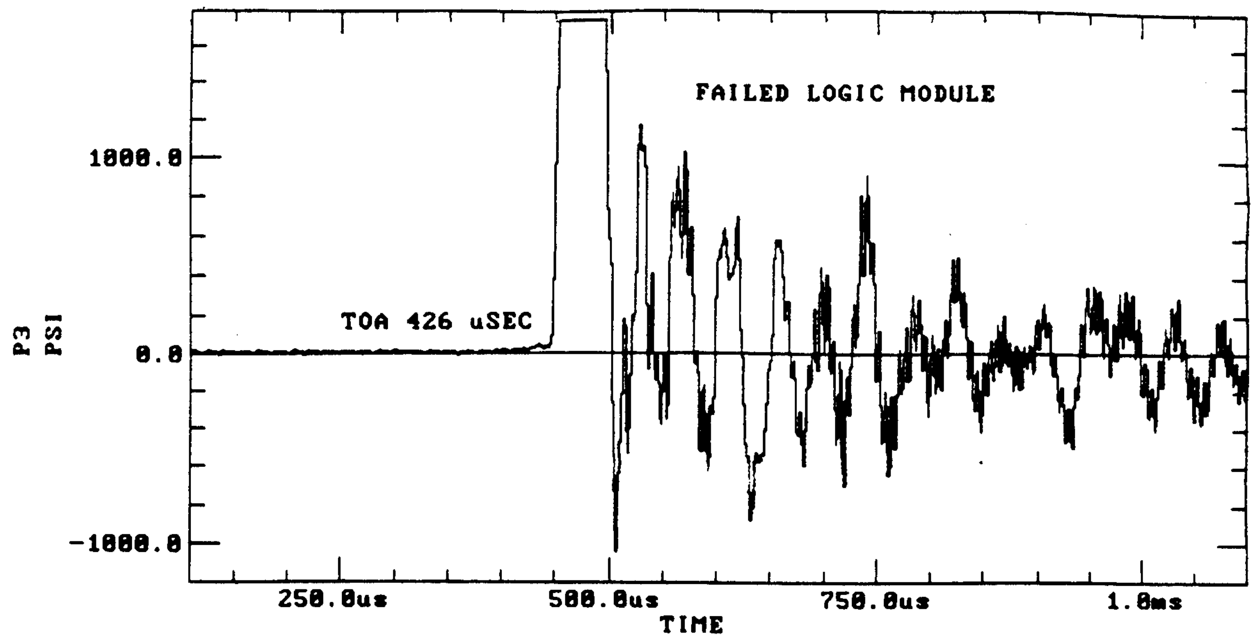

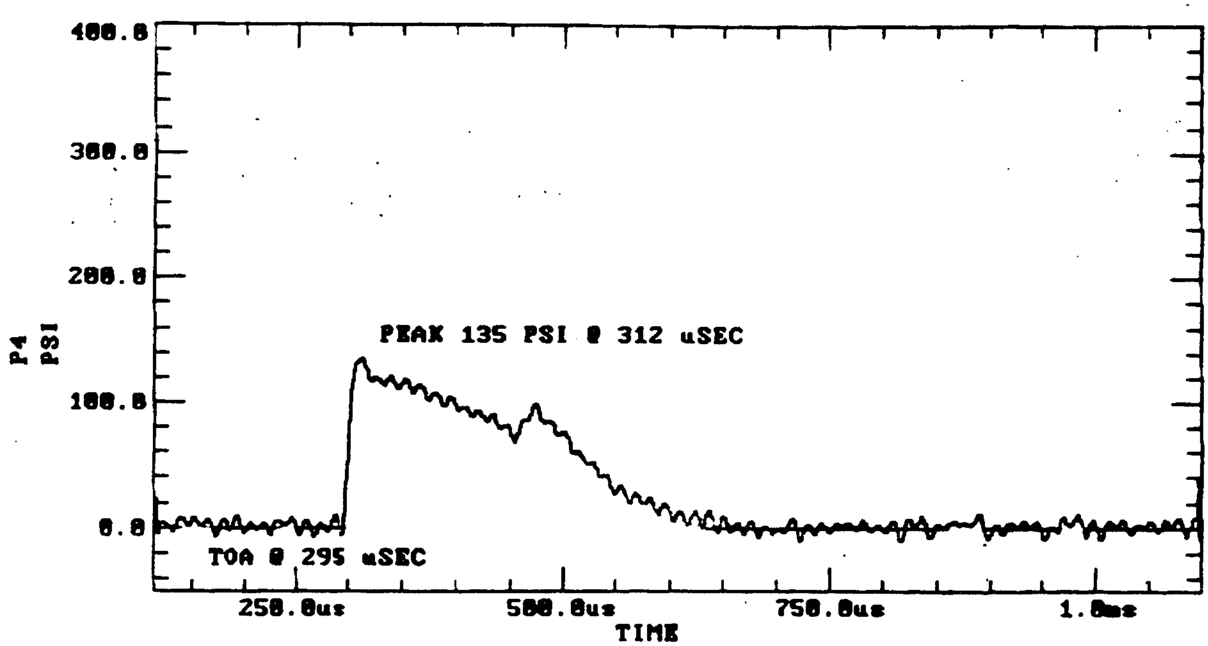

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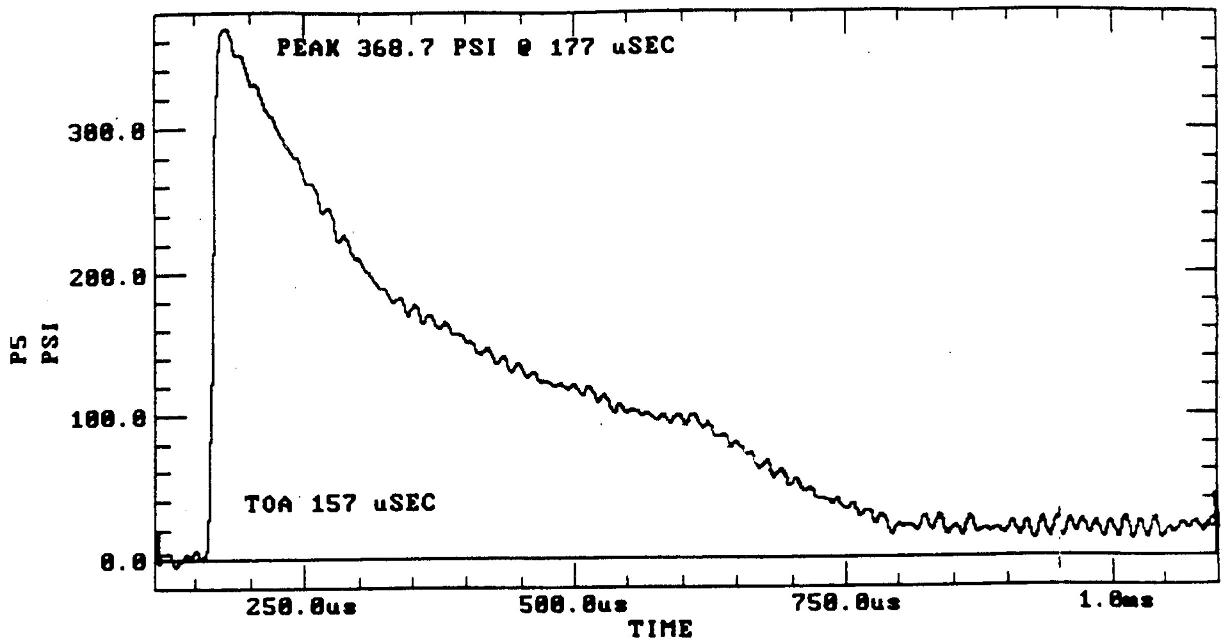

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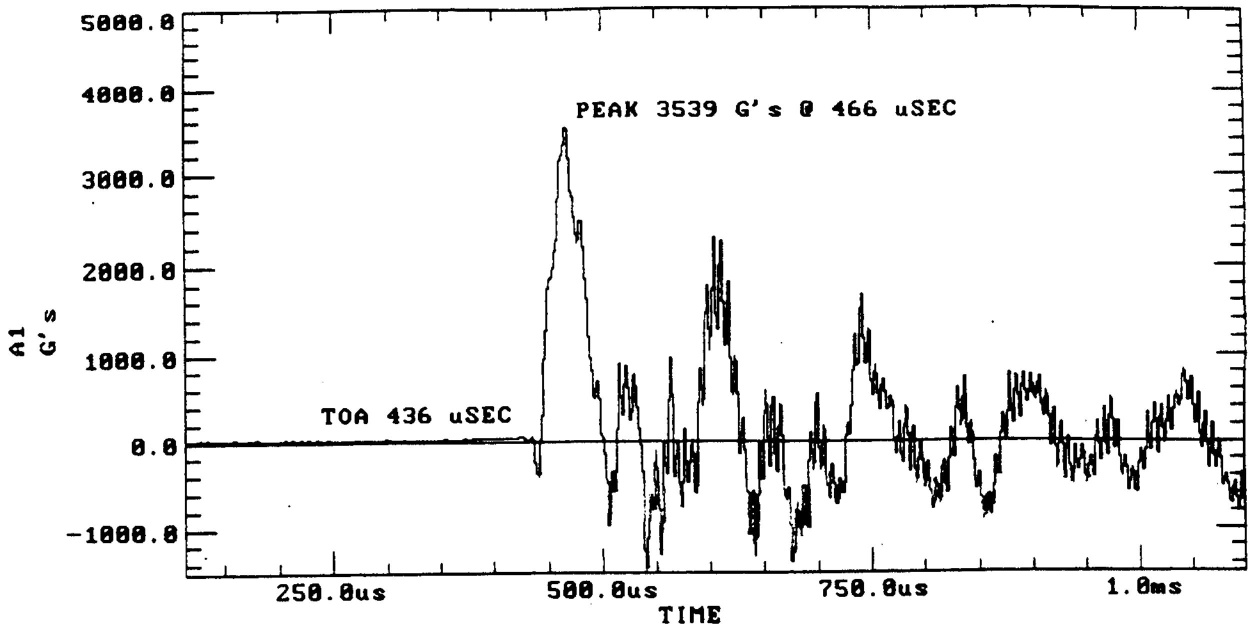

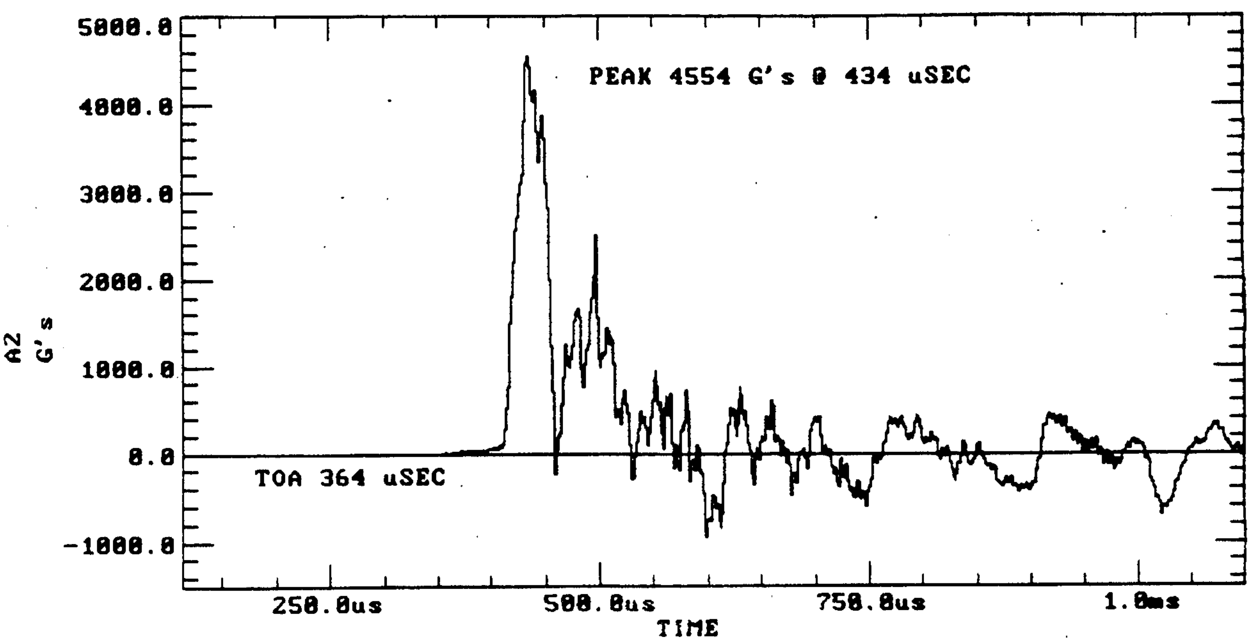

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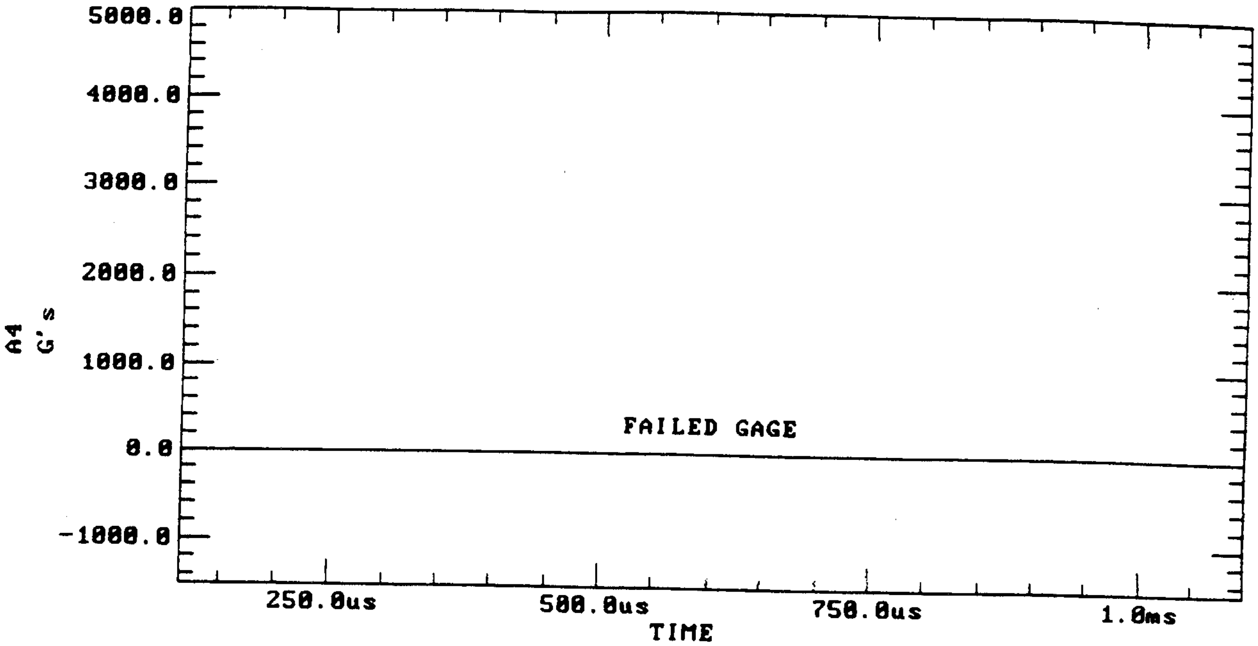

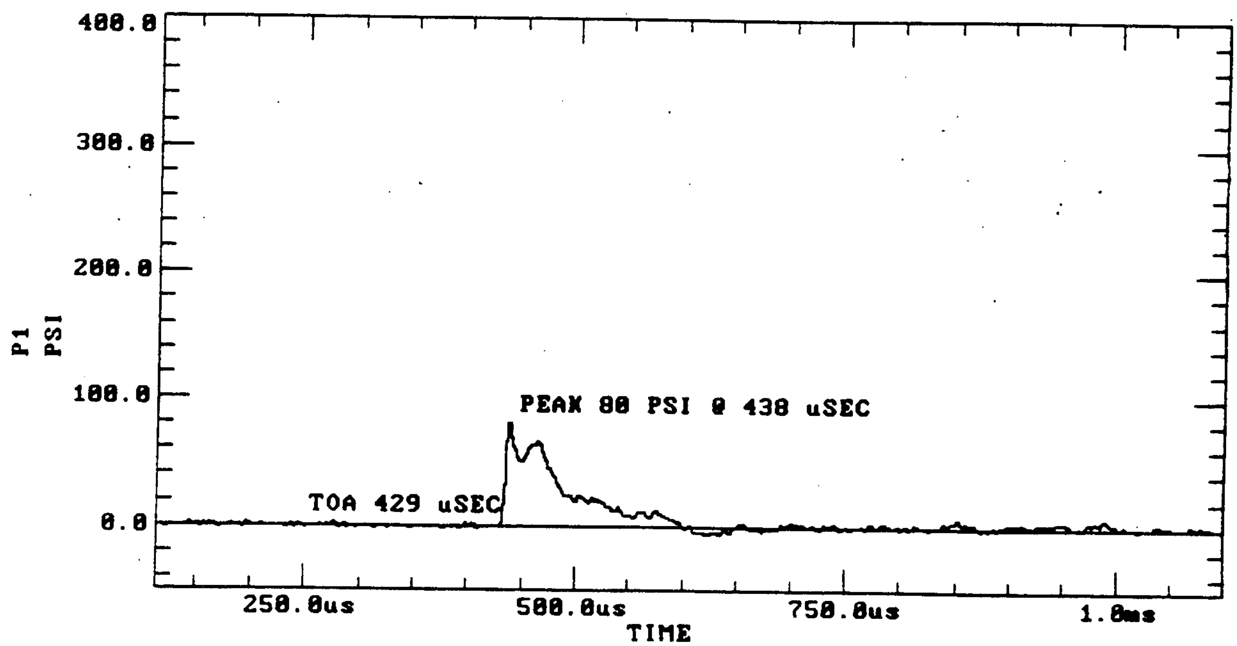

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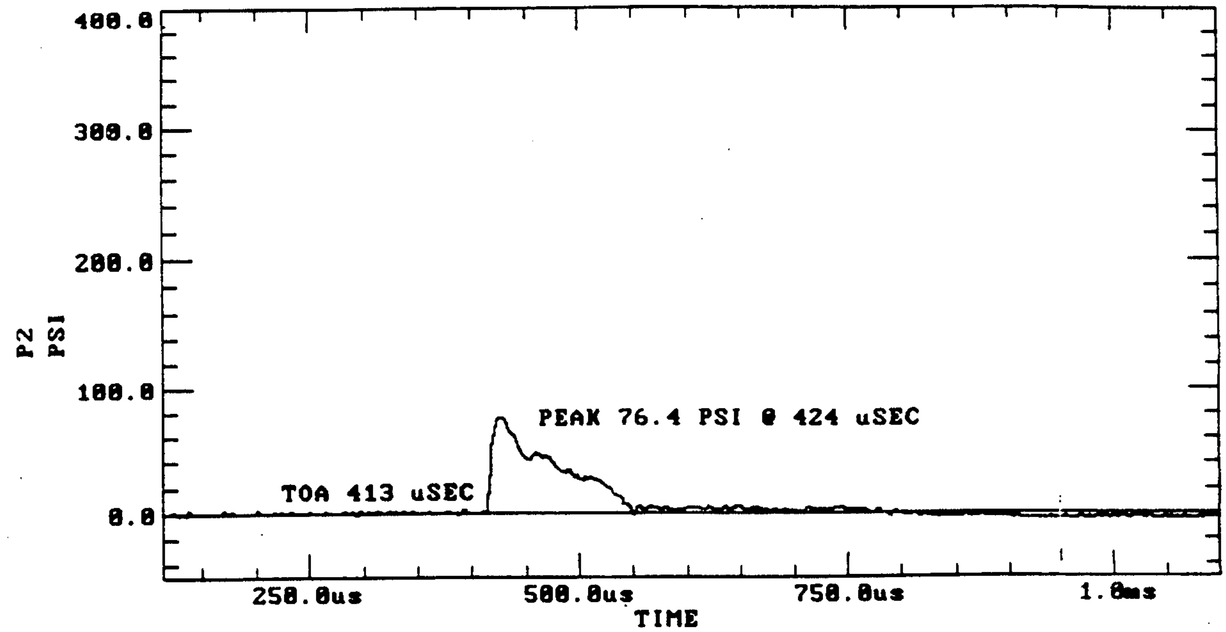

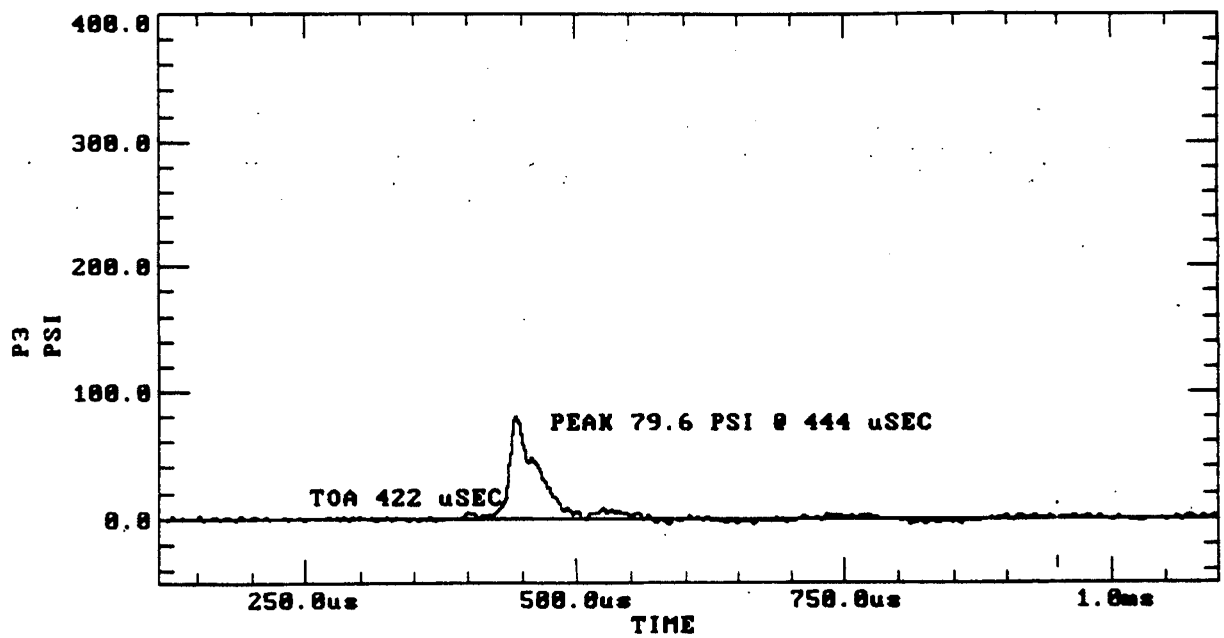

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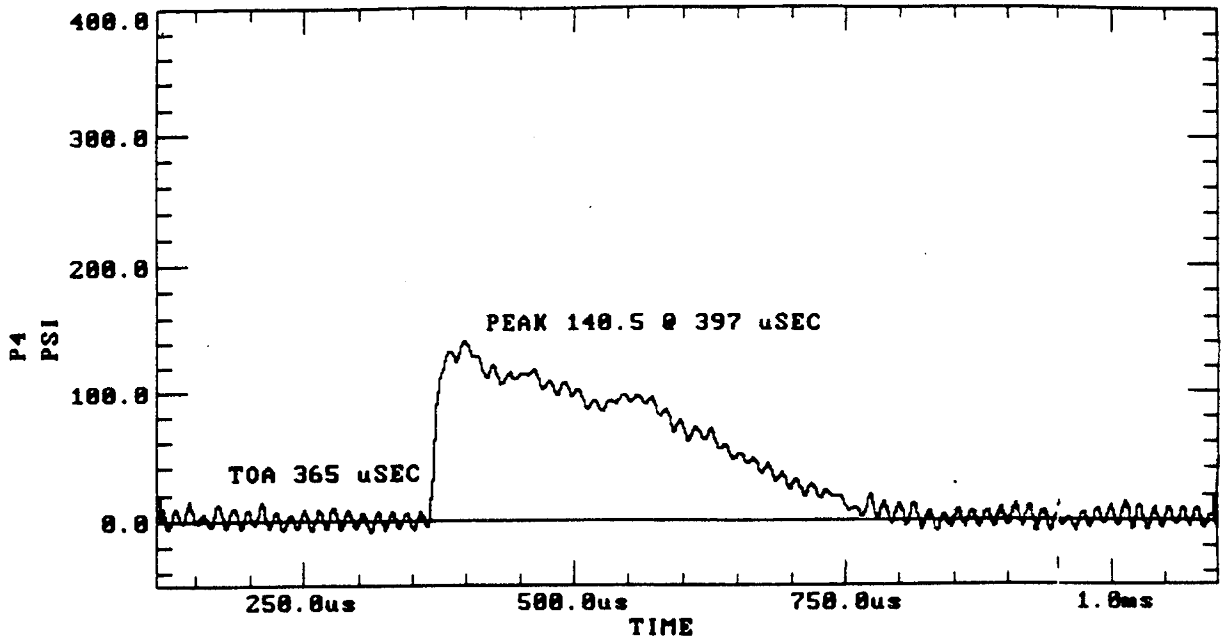

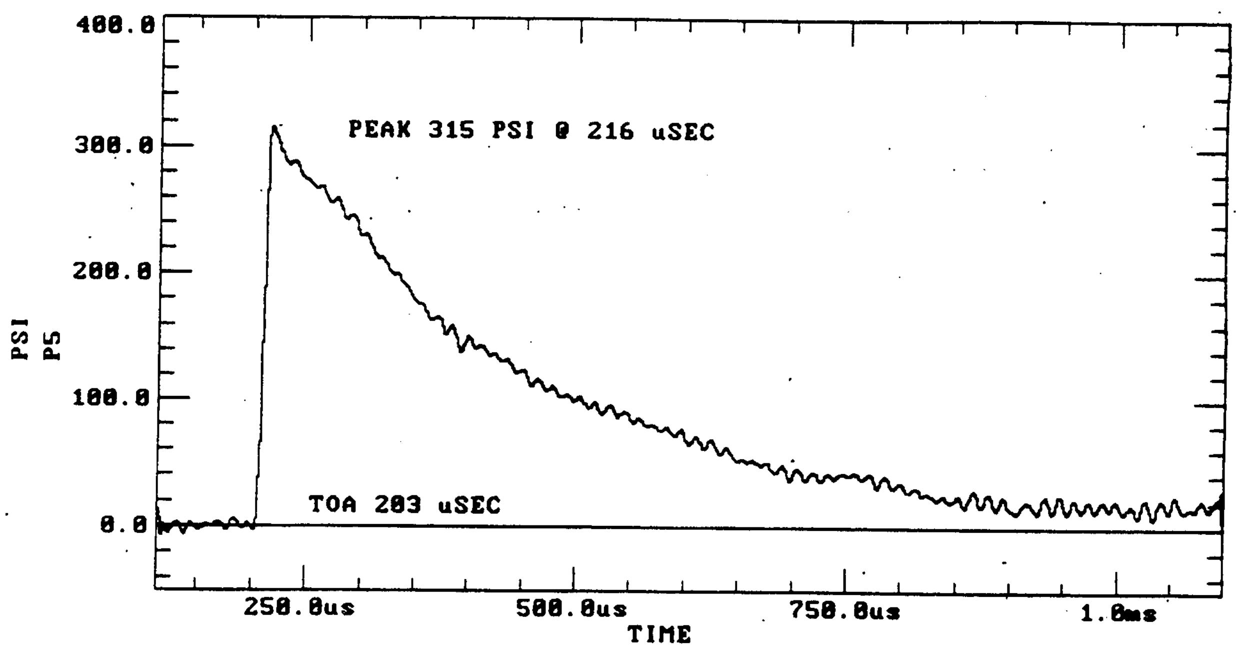

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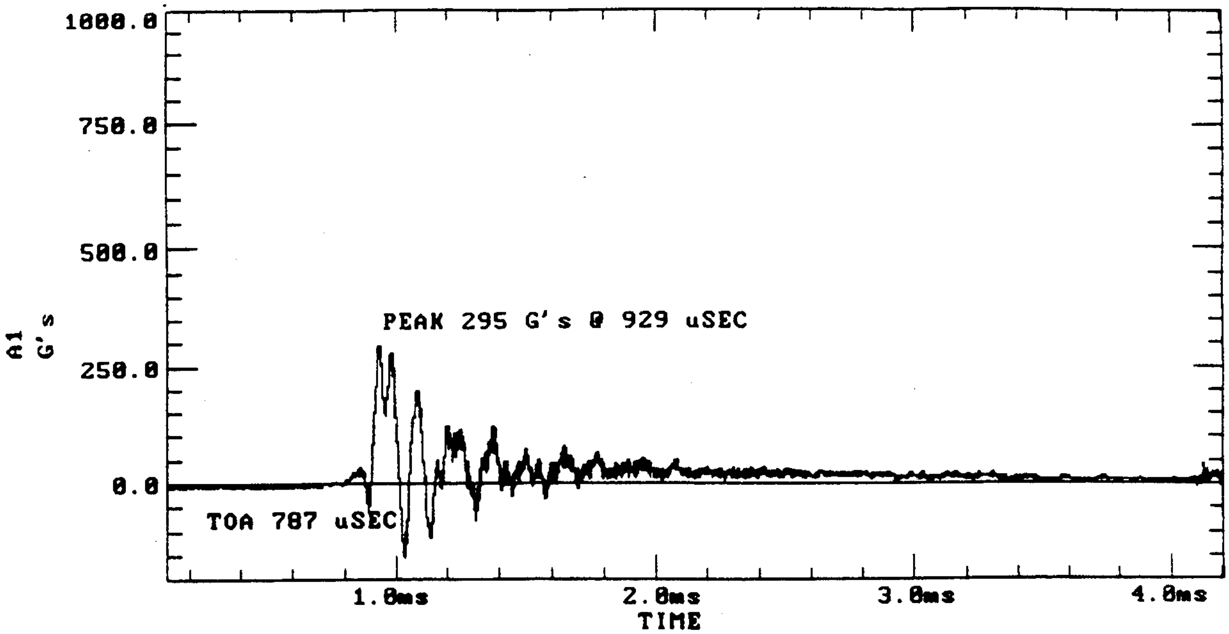

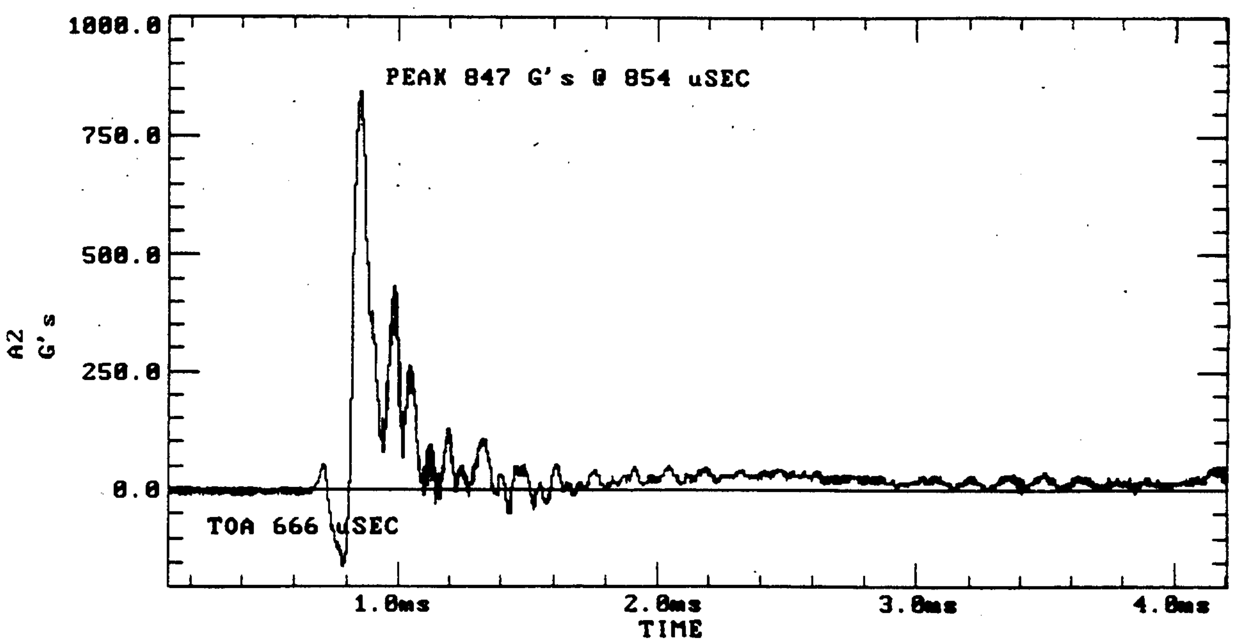

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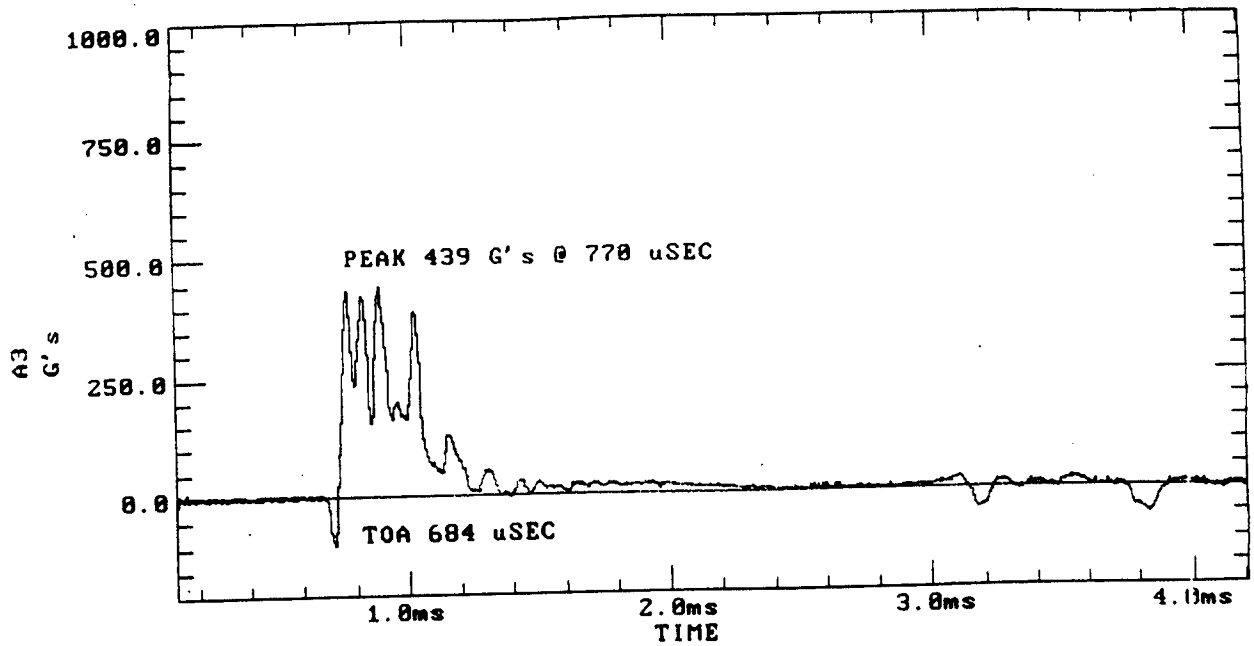

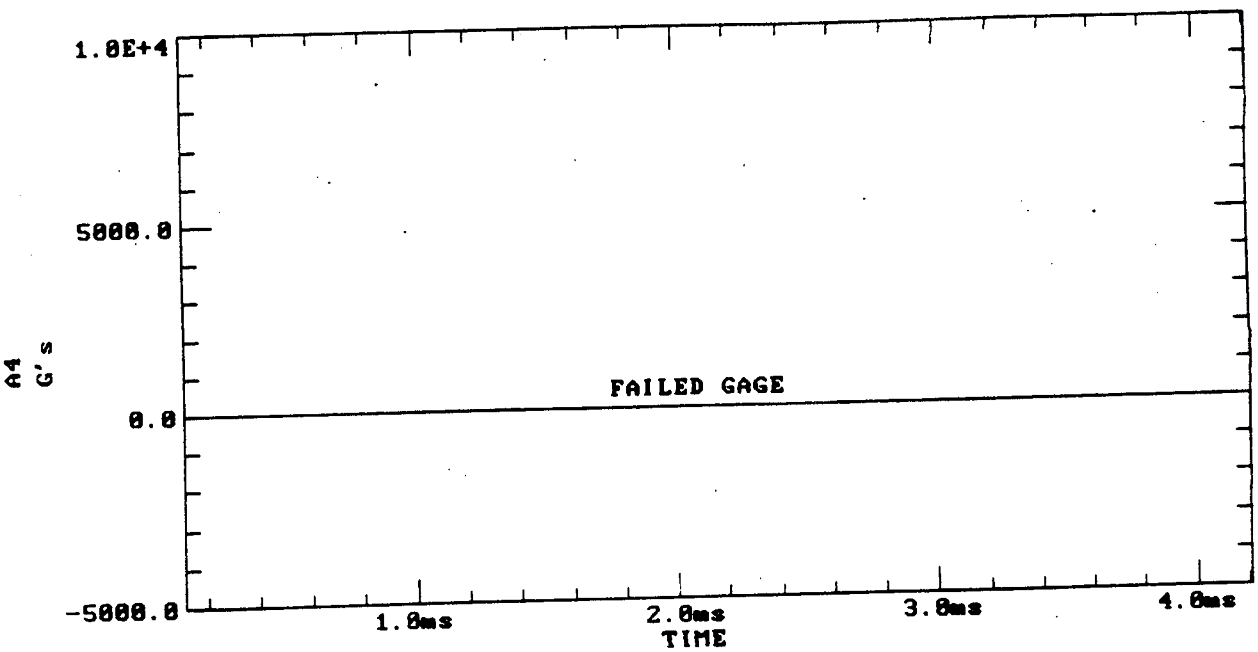

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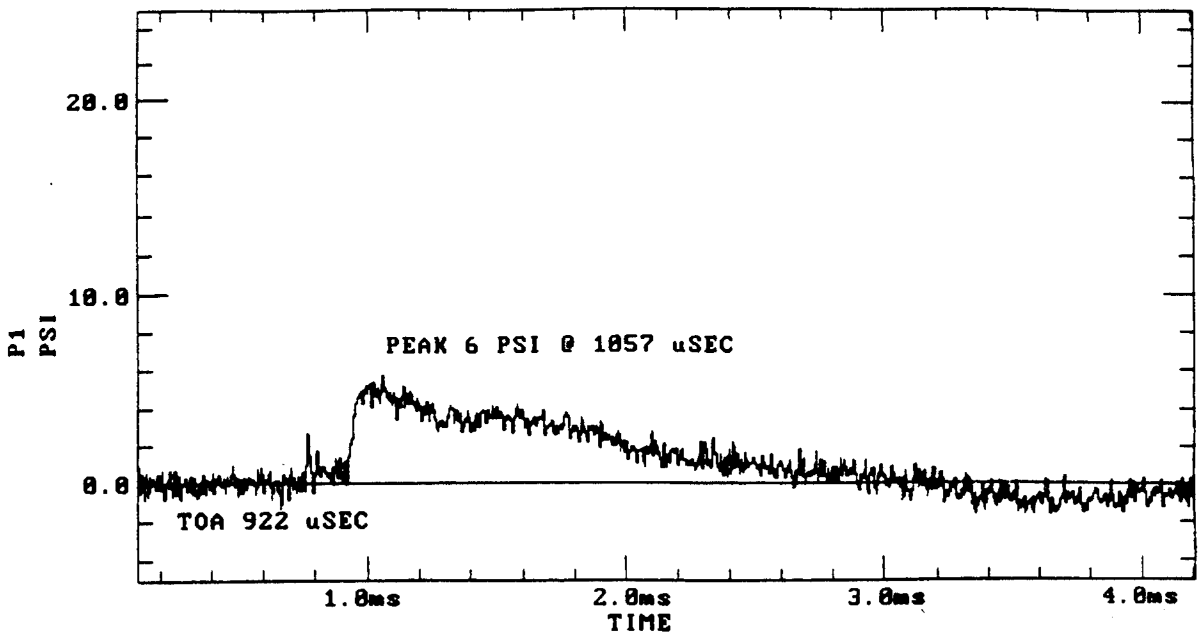

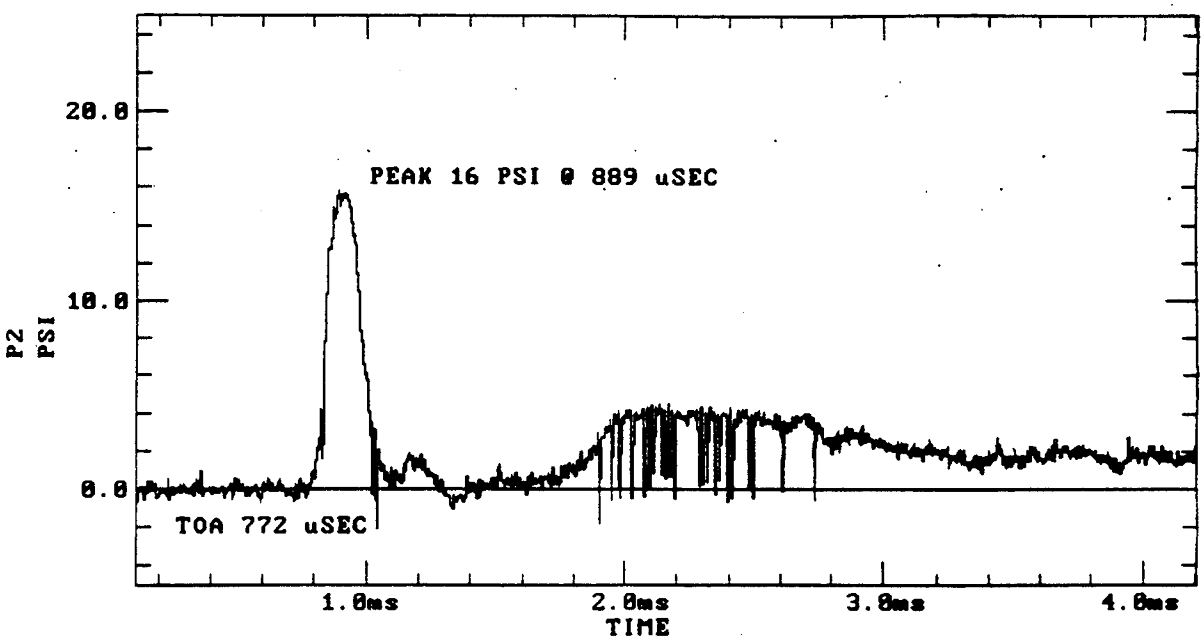

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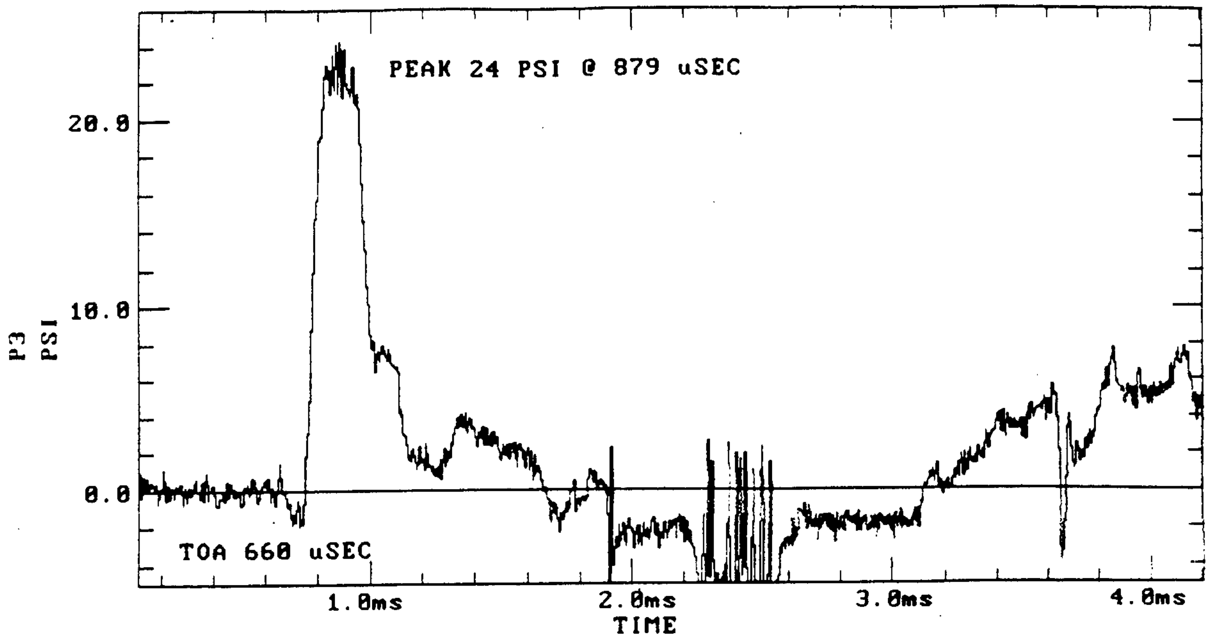
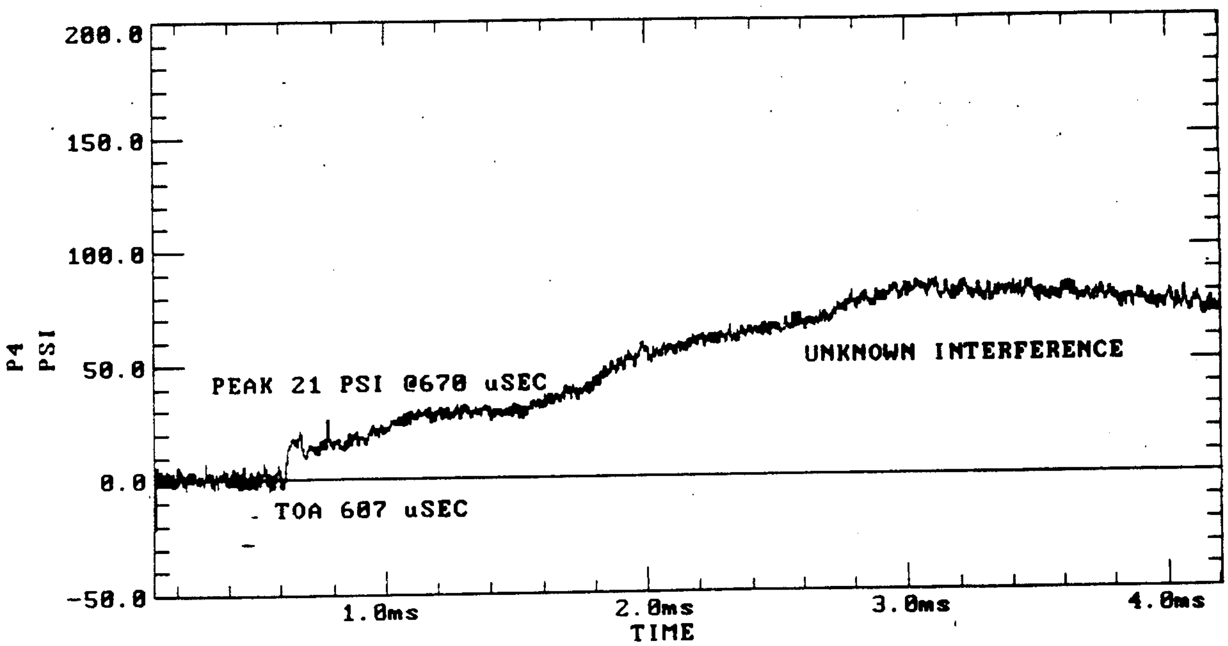
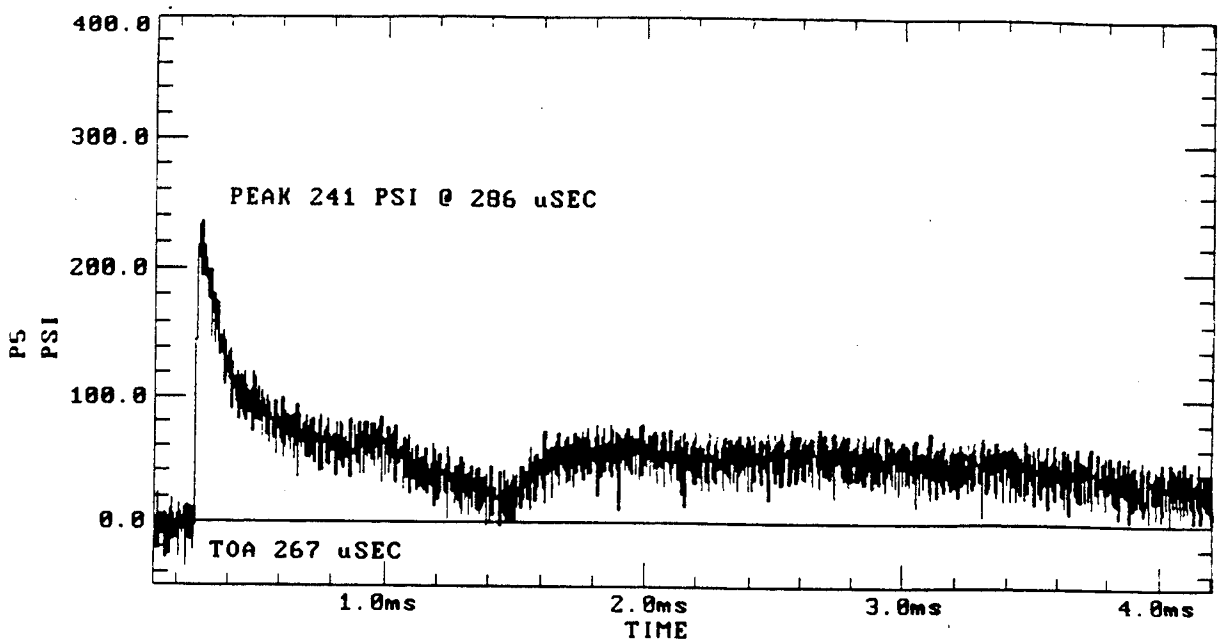




Test 27. Gage Data (continued)



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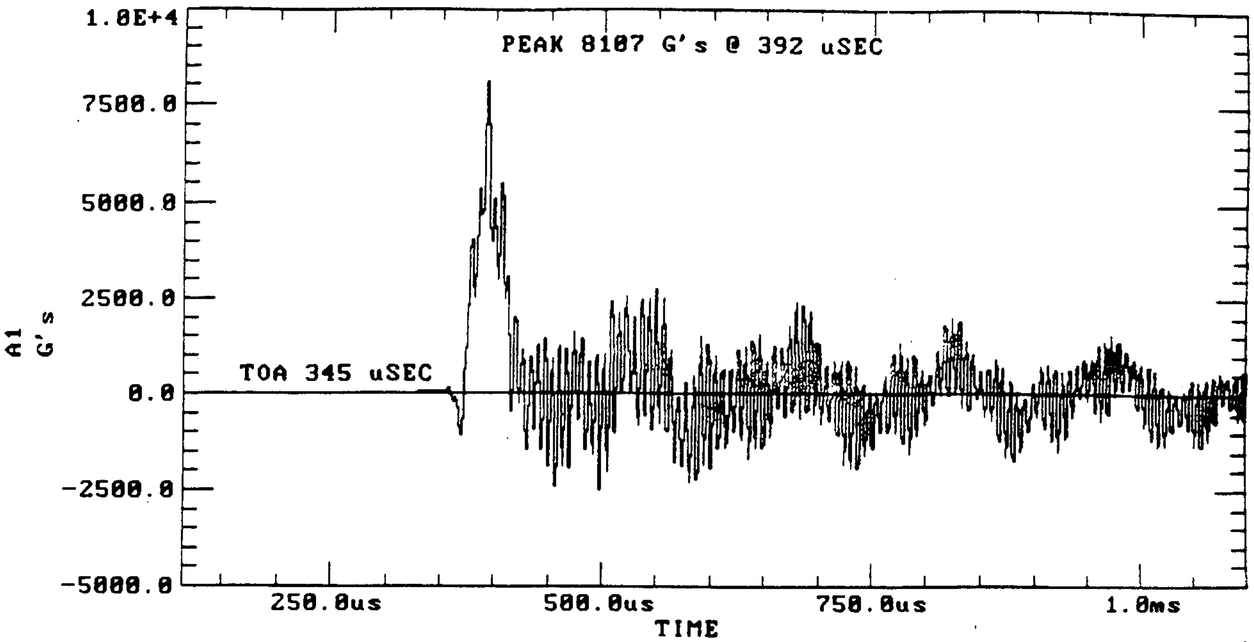

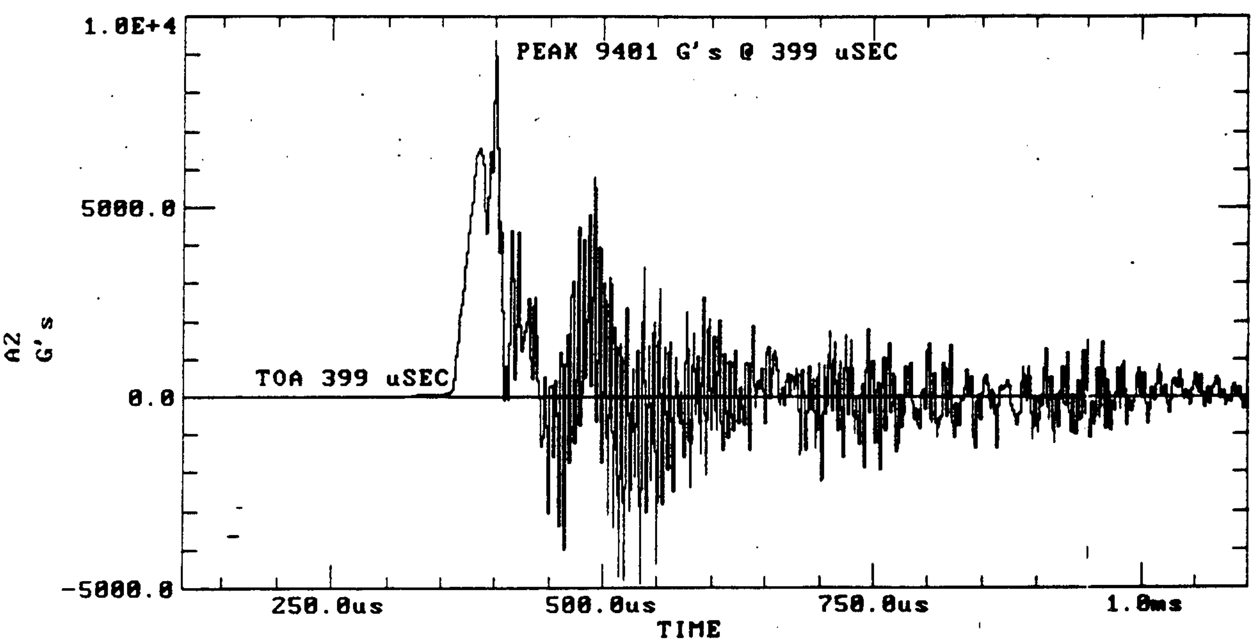

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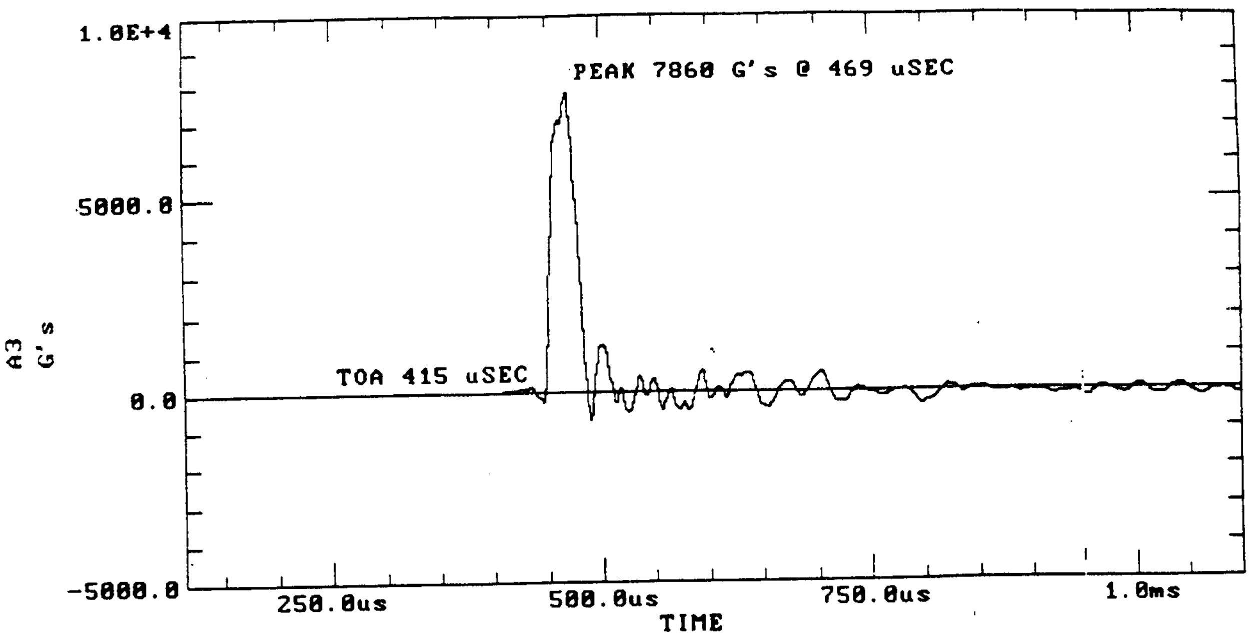

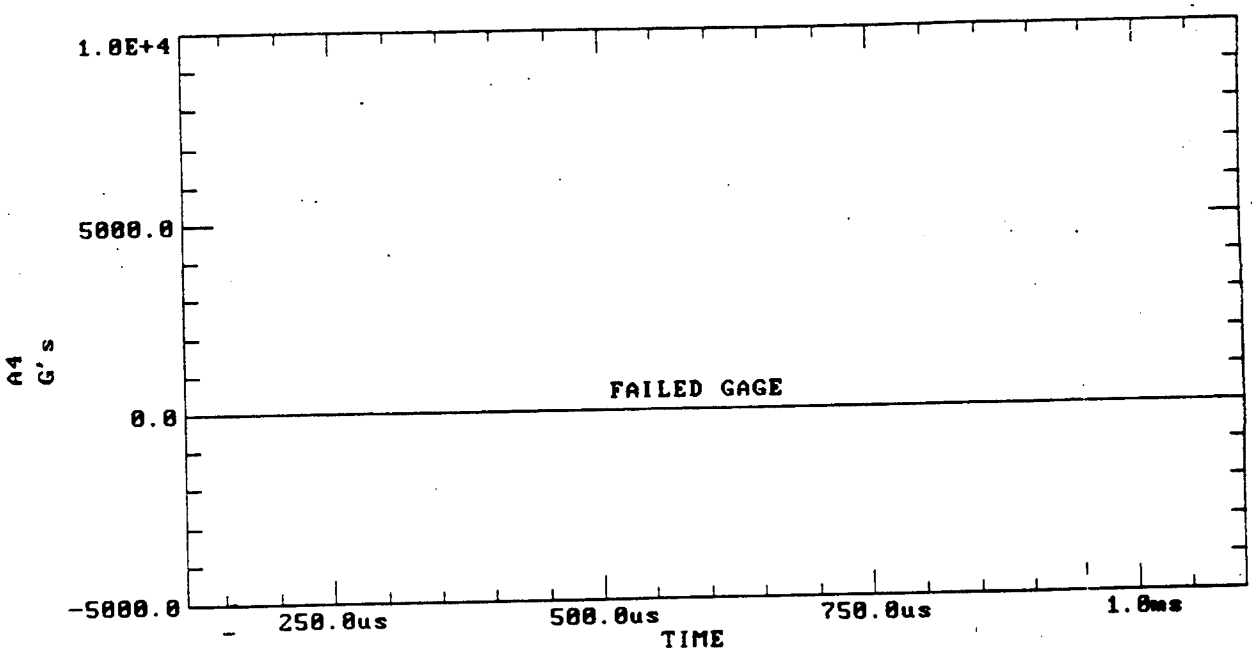

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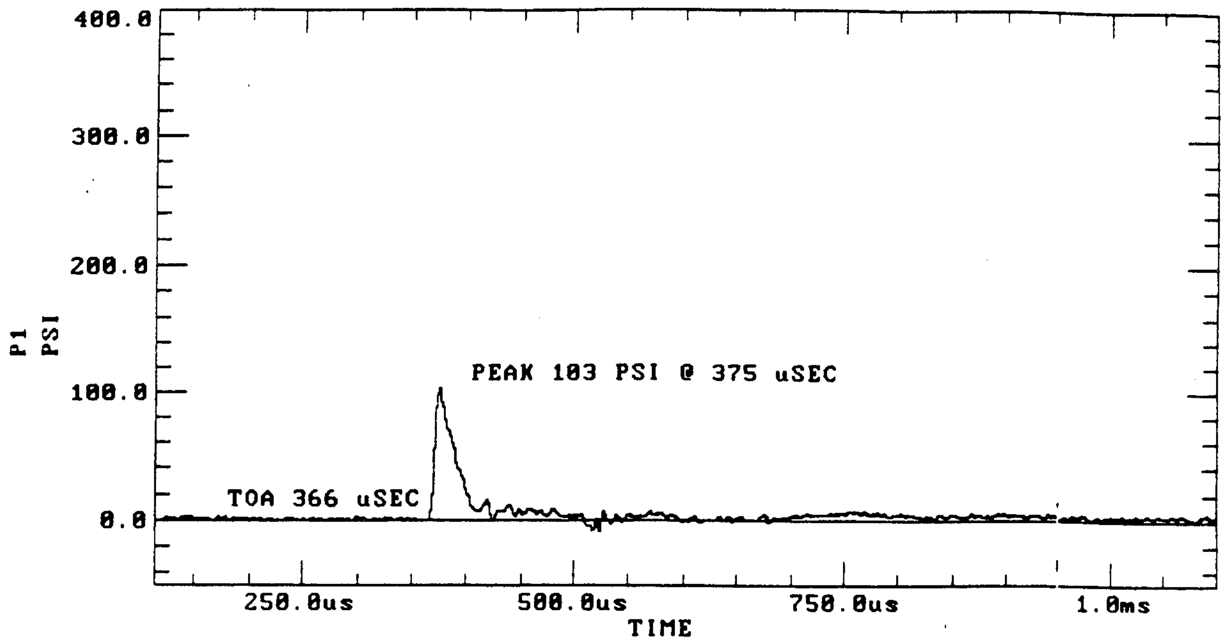

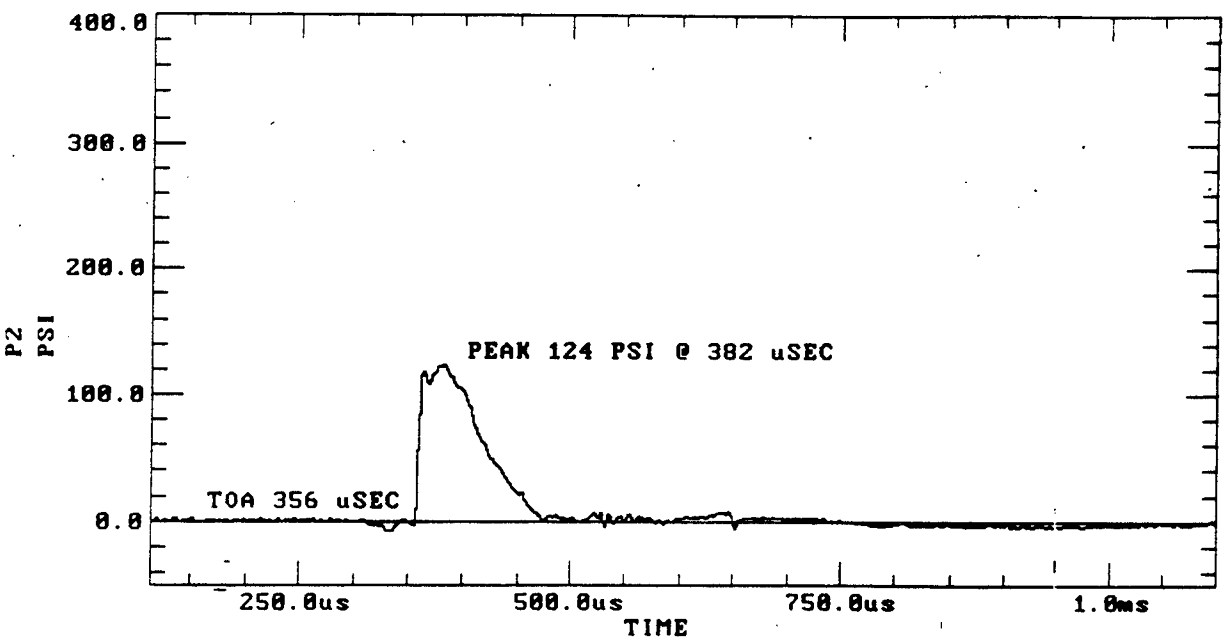

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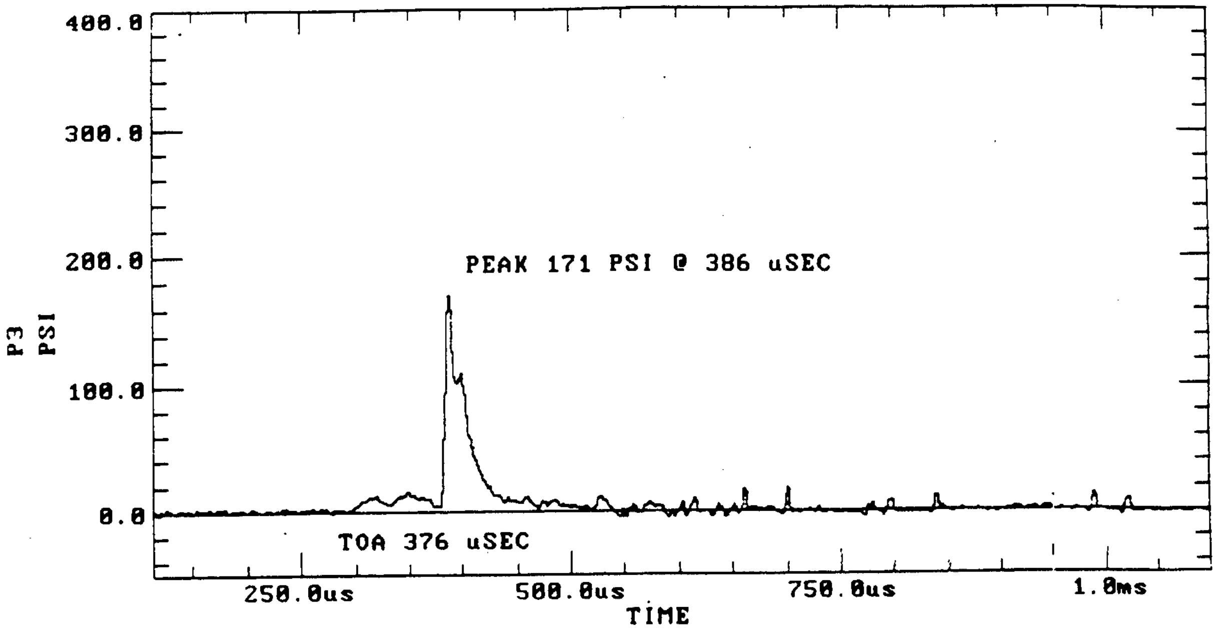

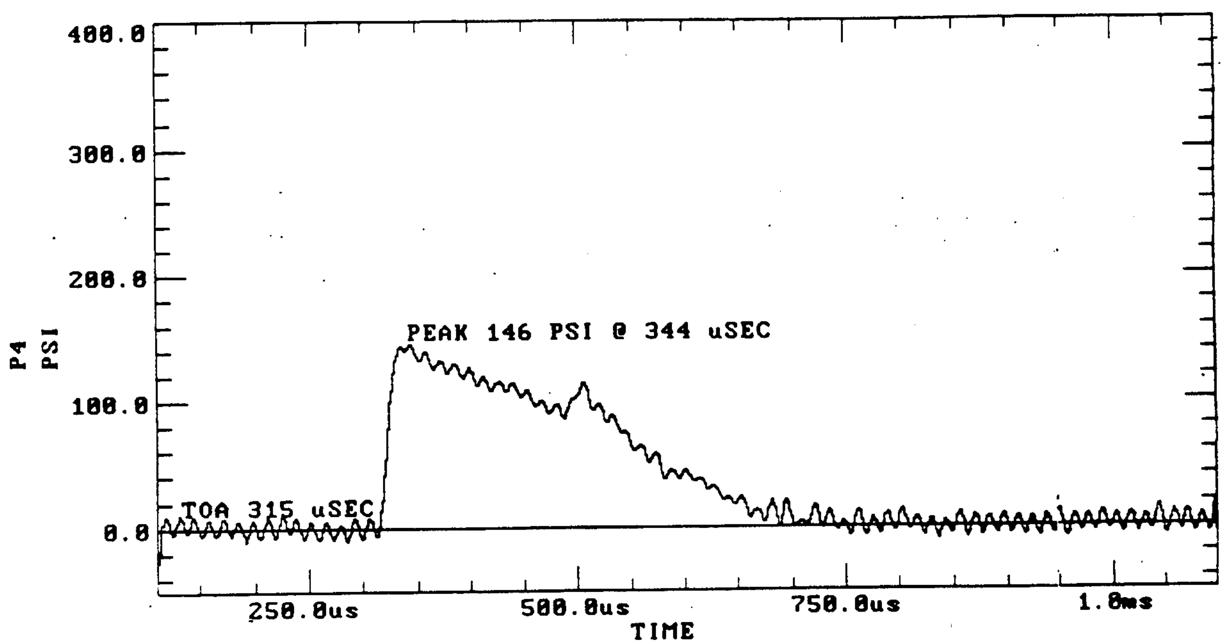

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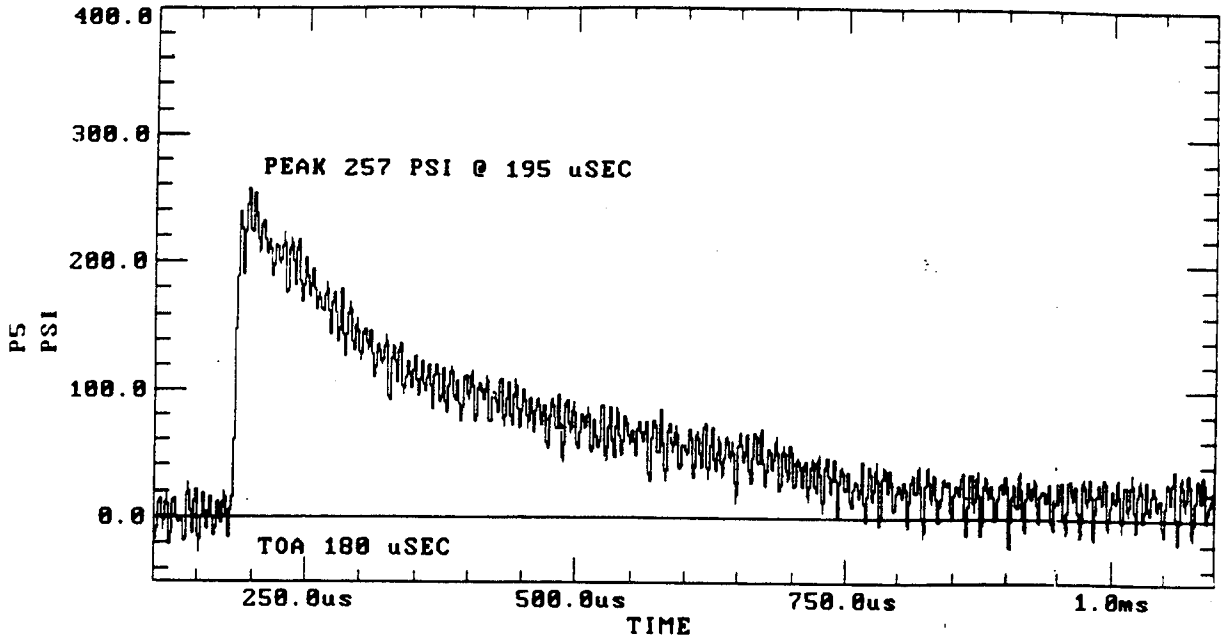

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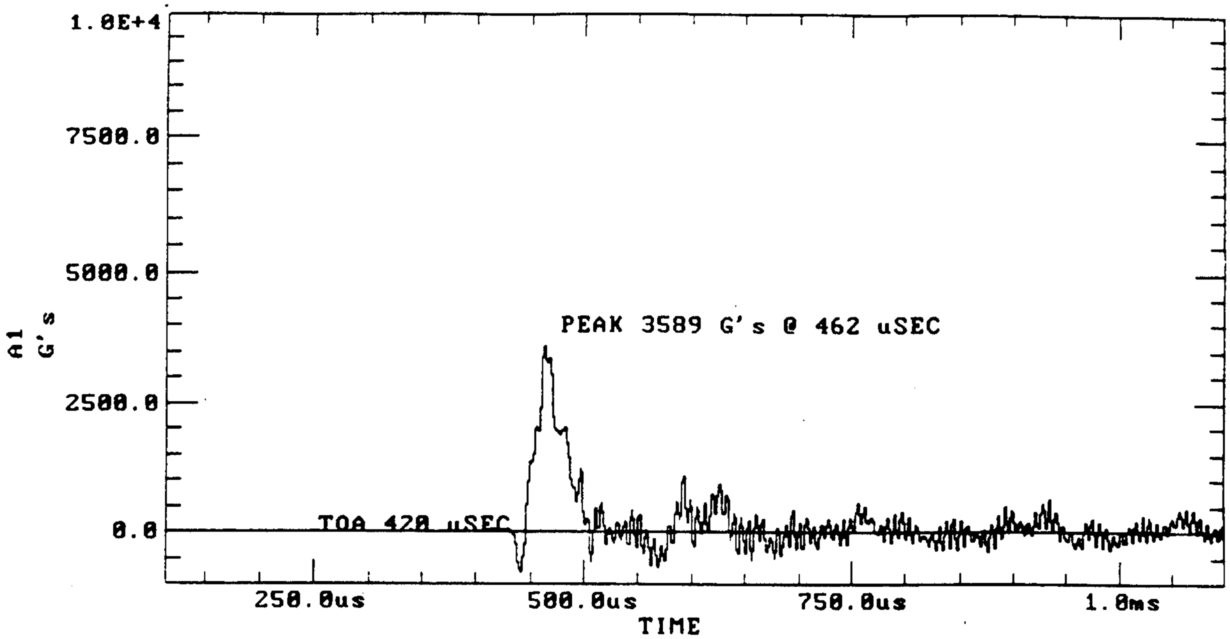

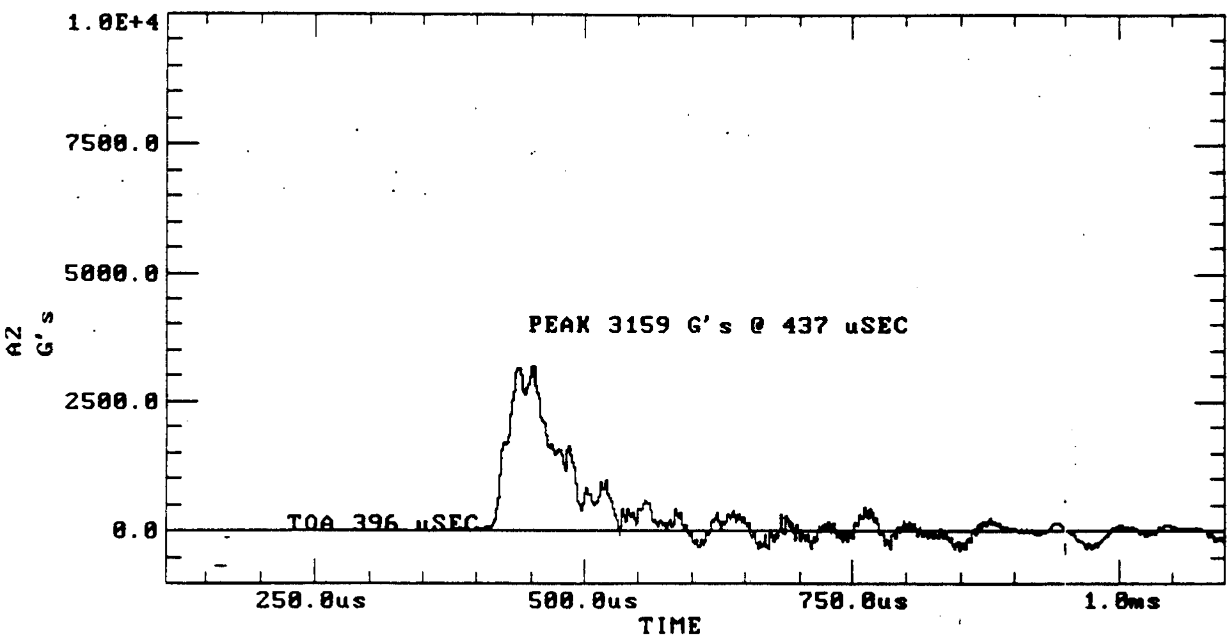

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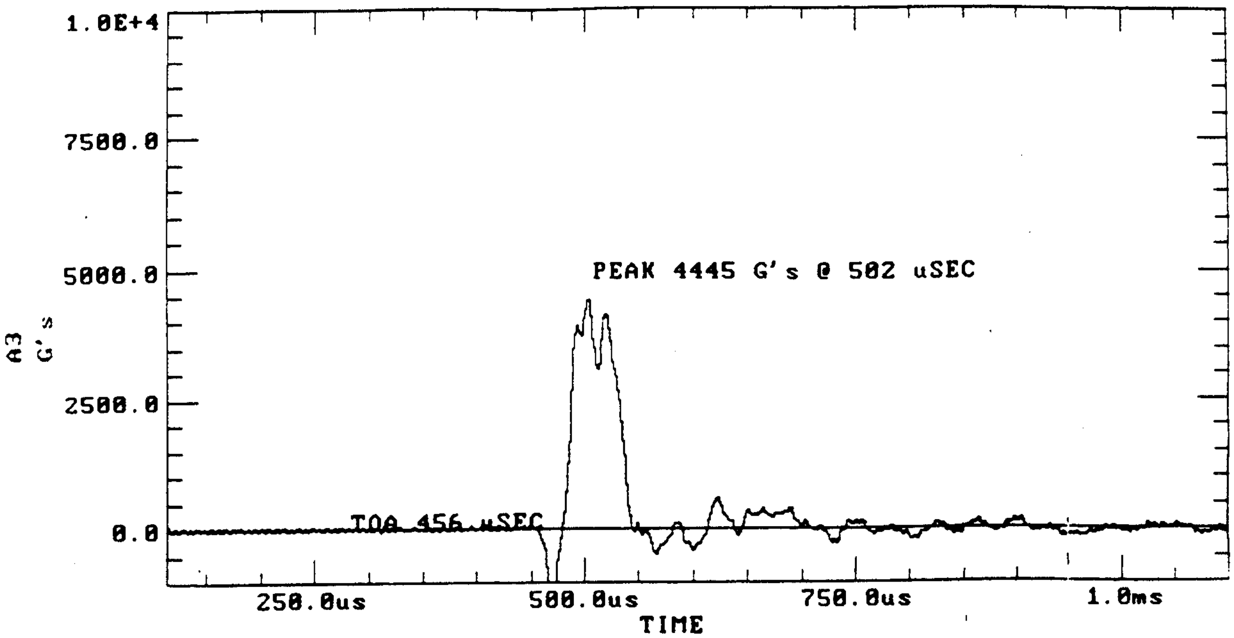

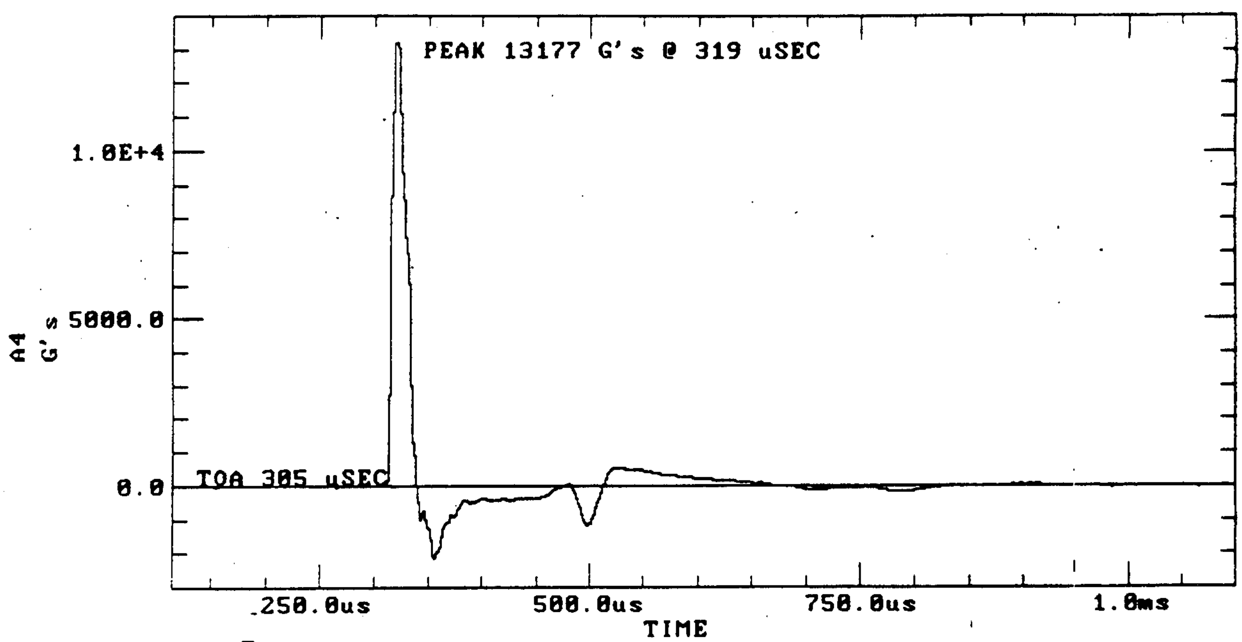

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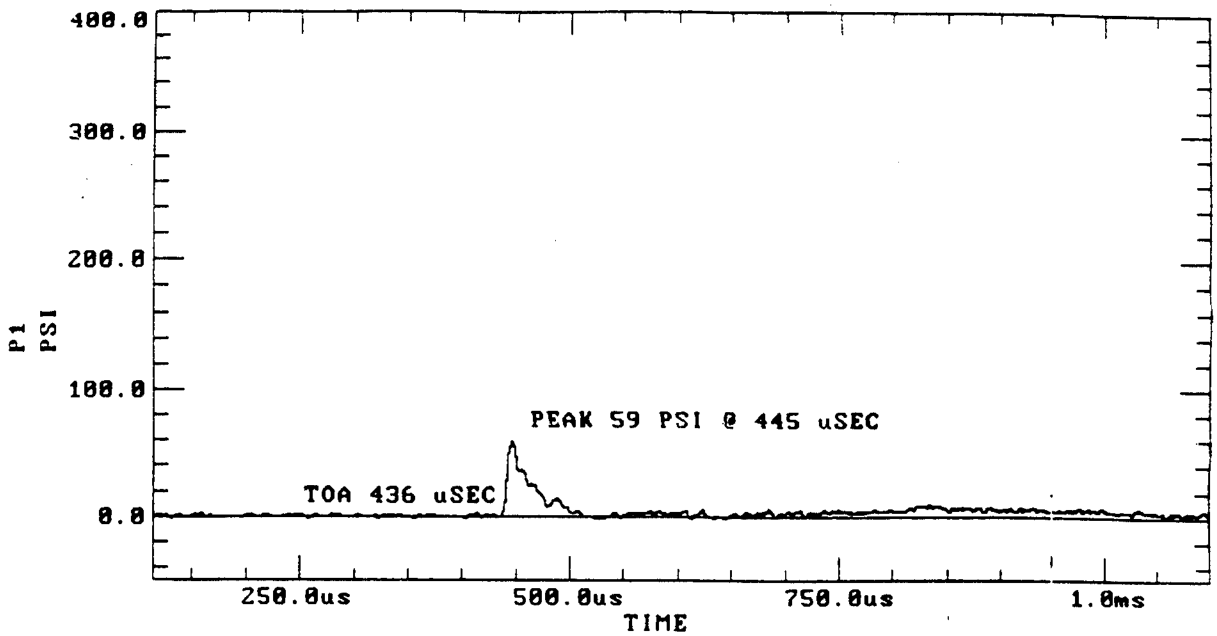

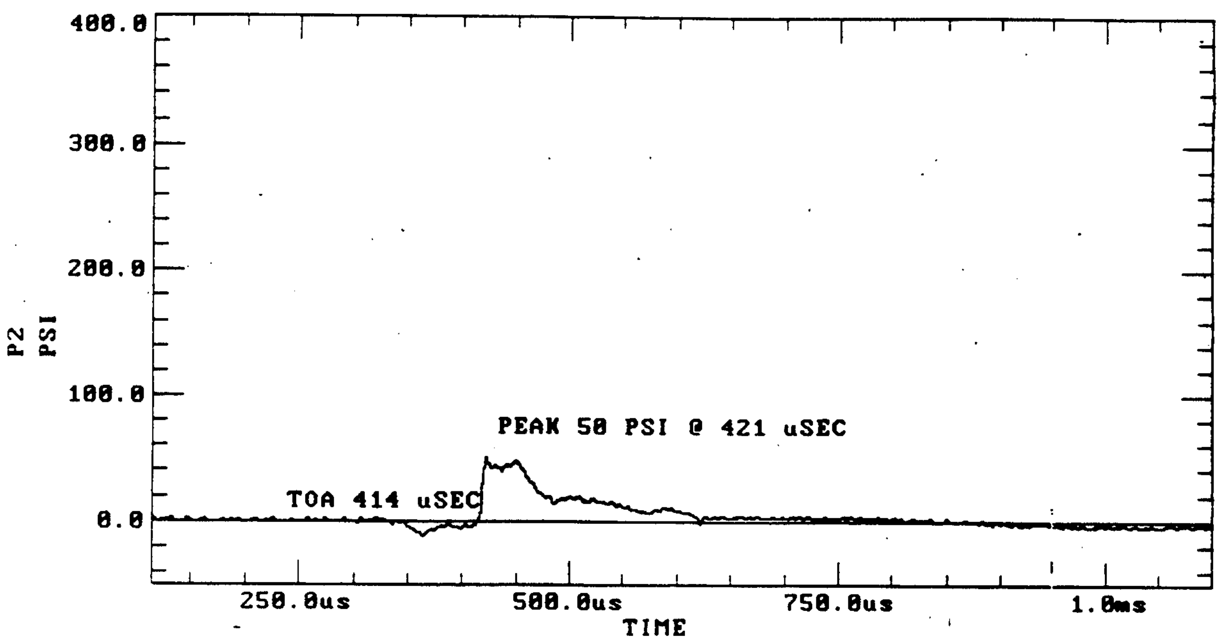

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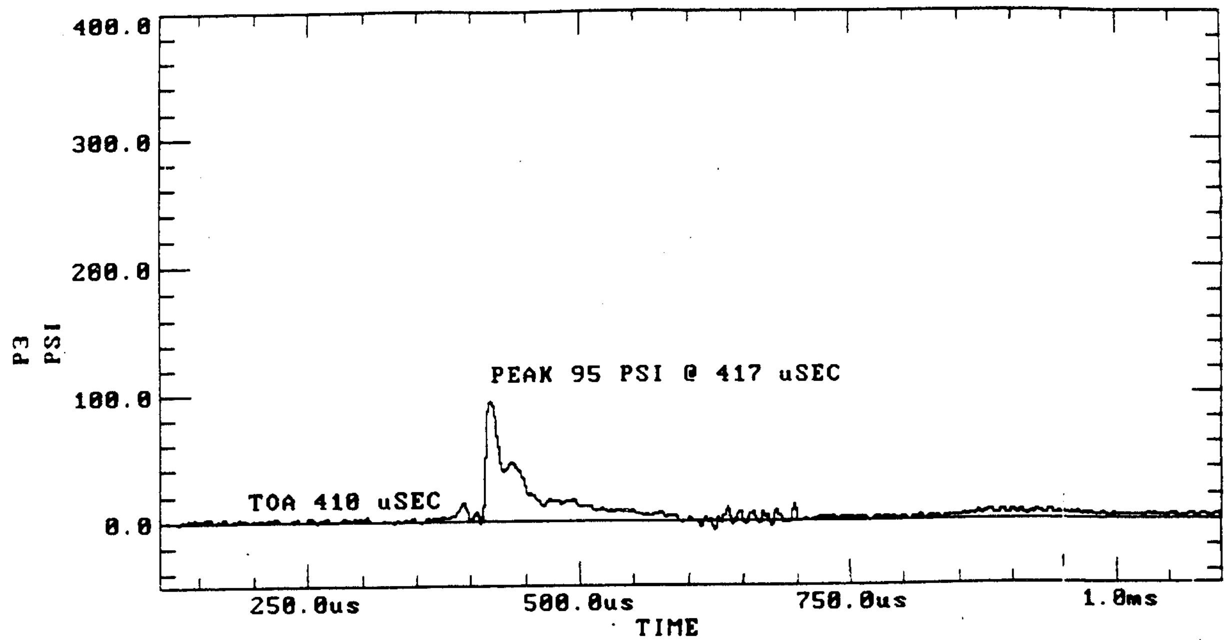

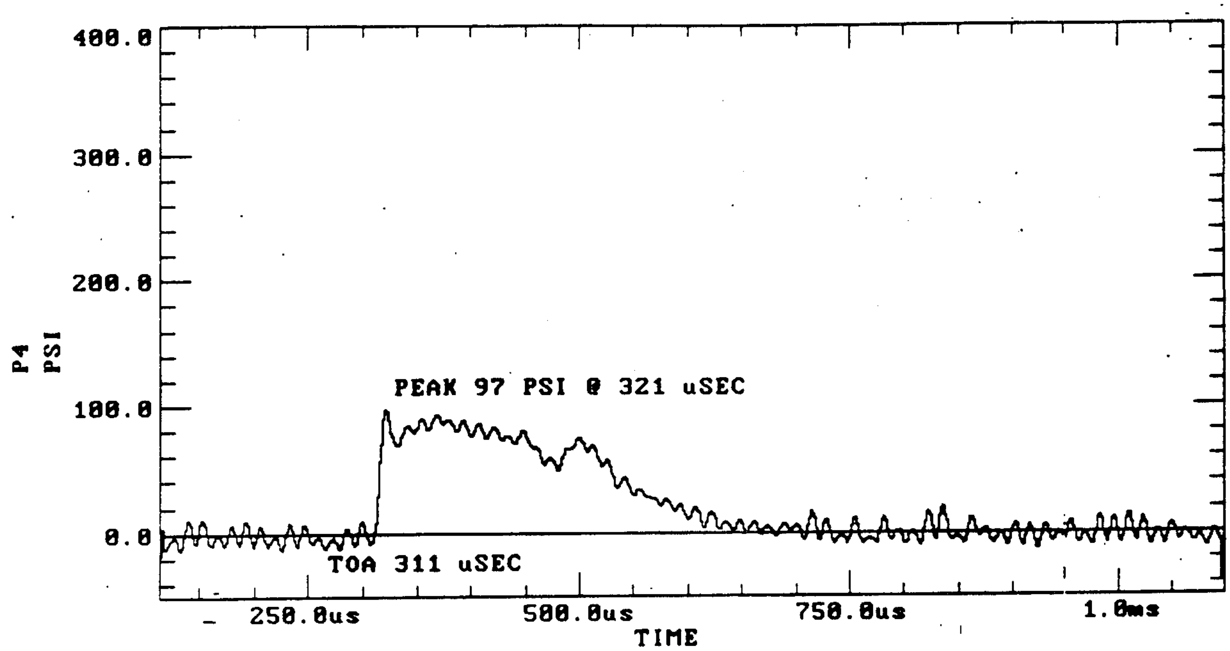

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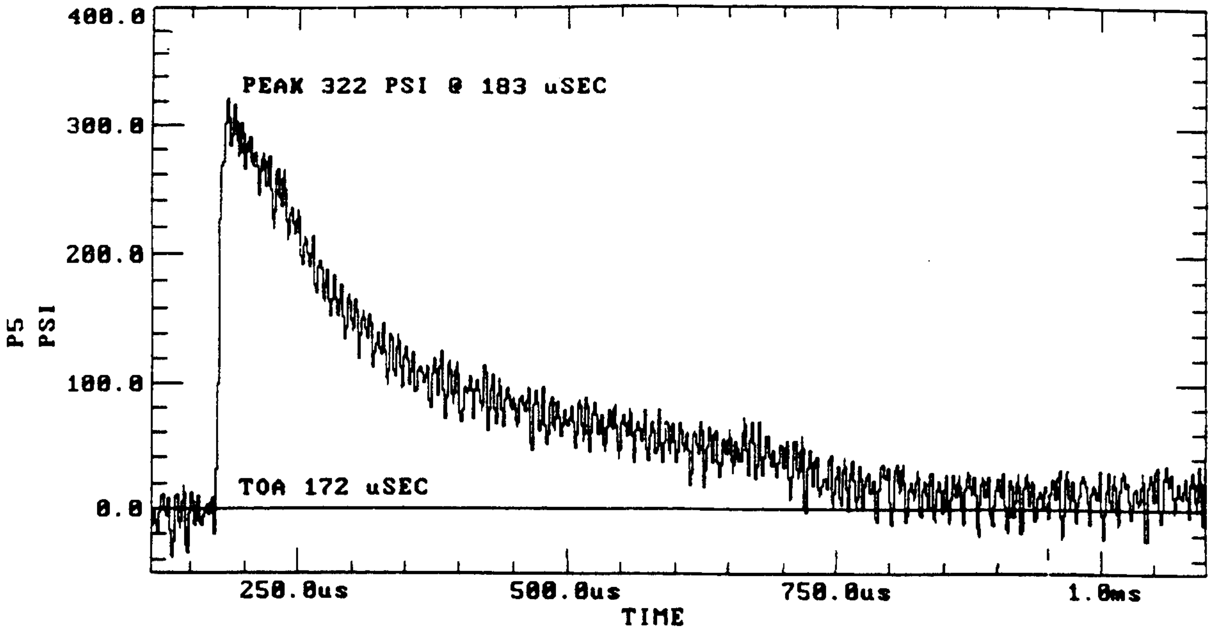

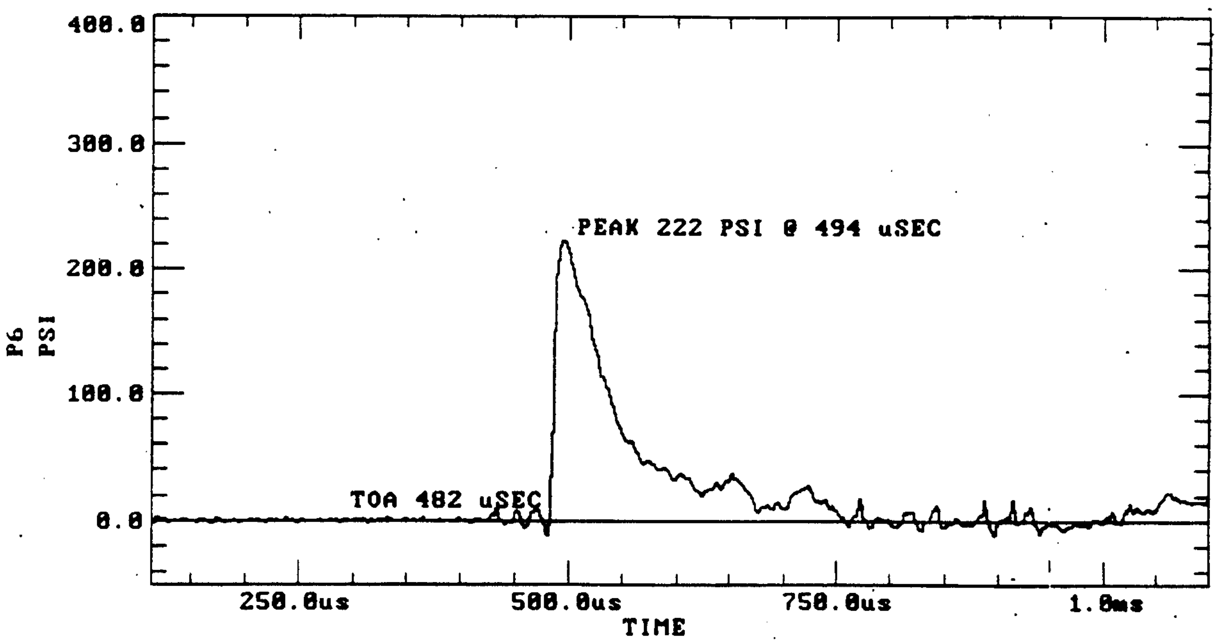

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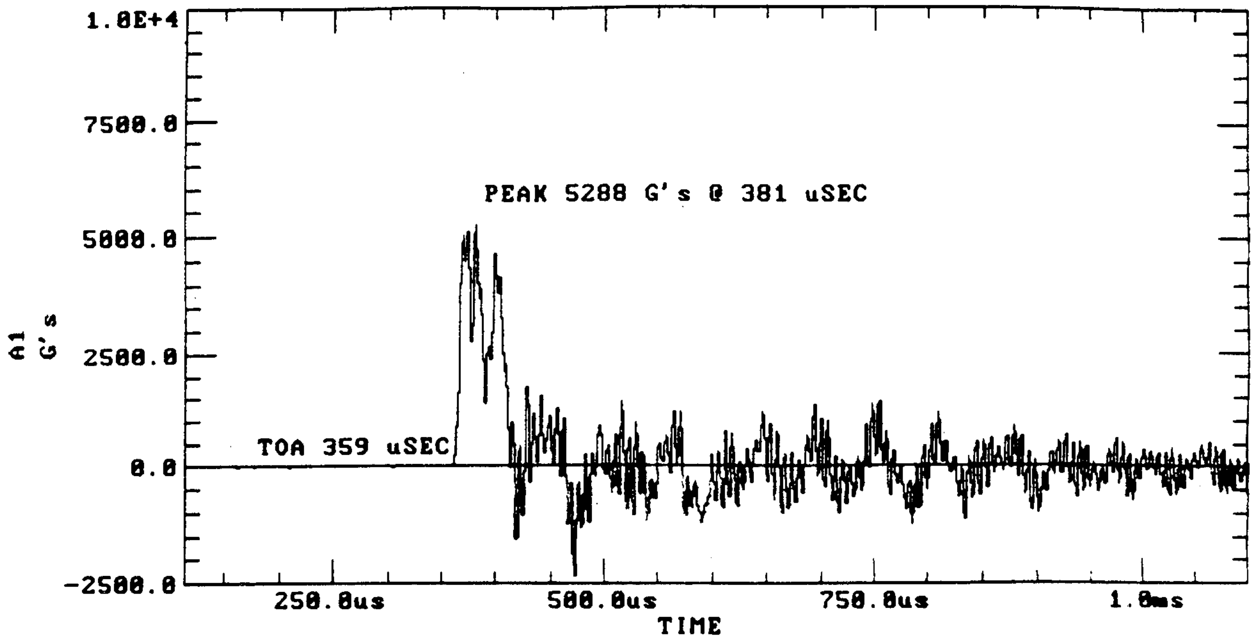

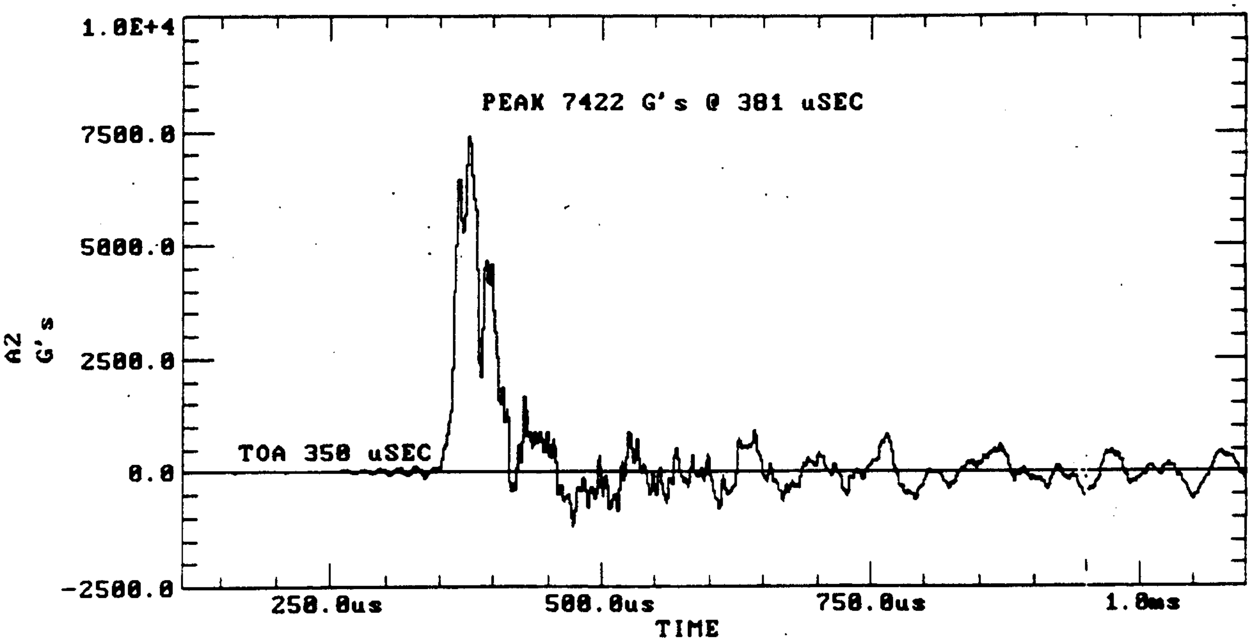

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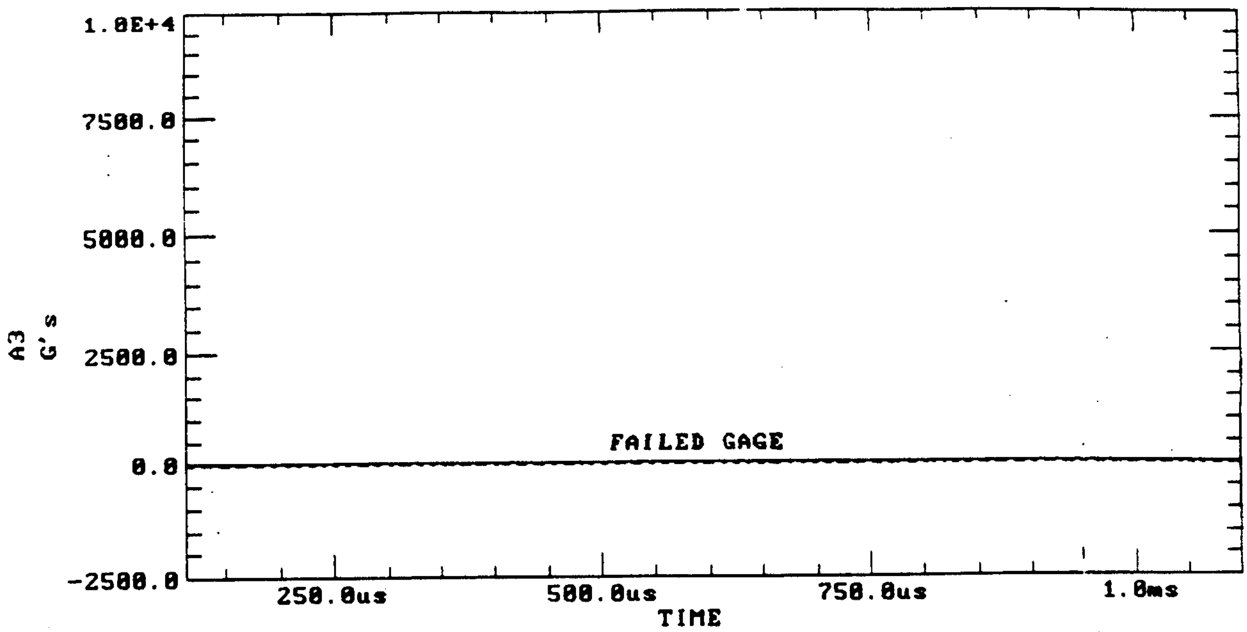

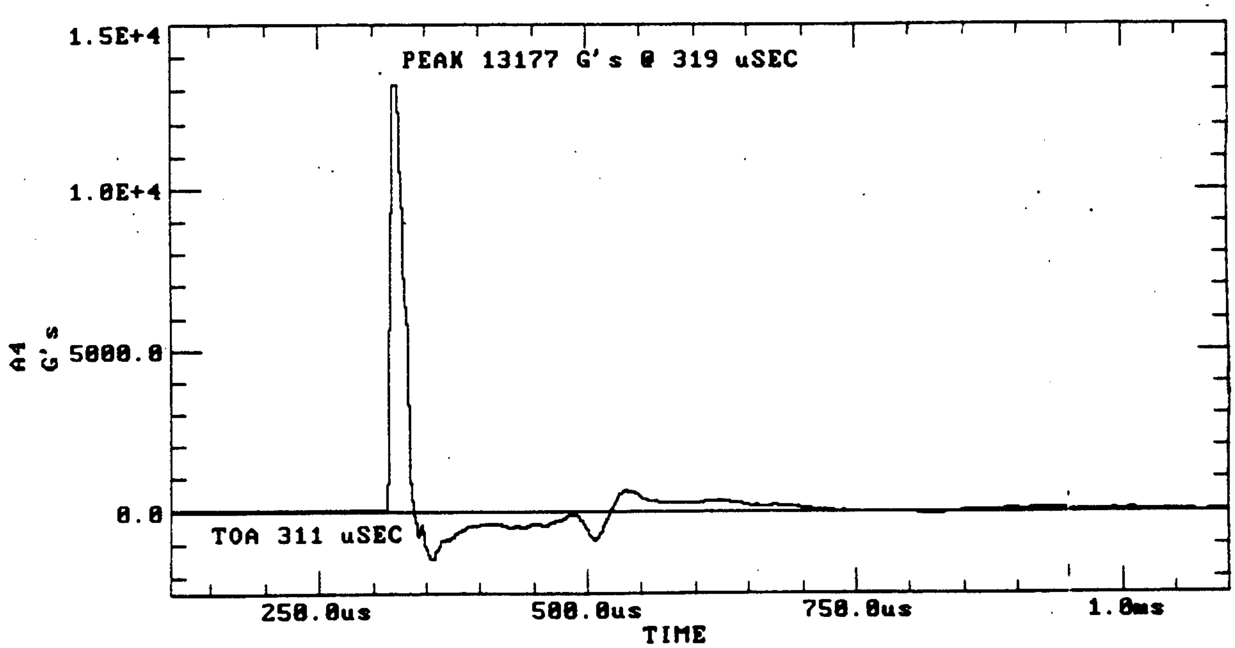

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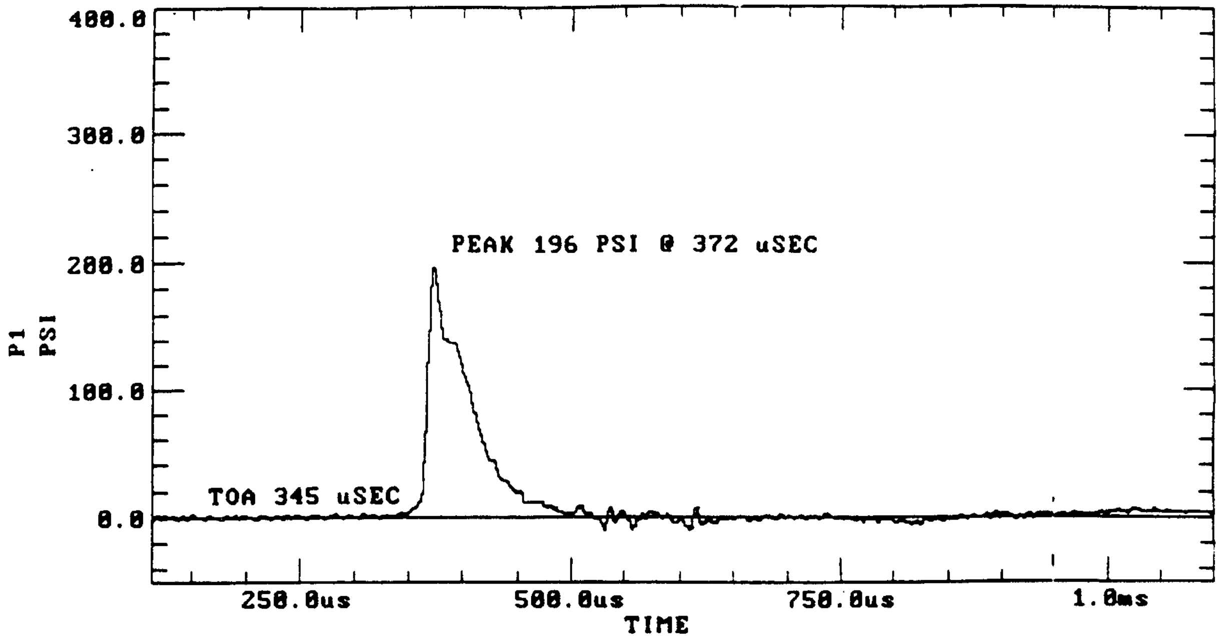

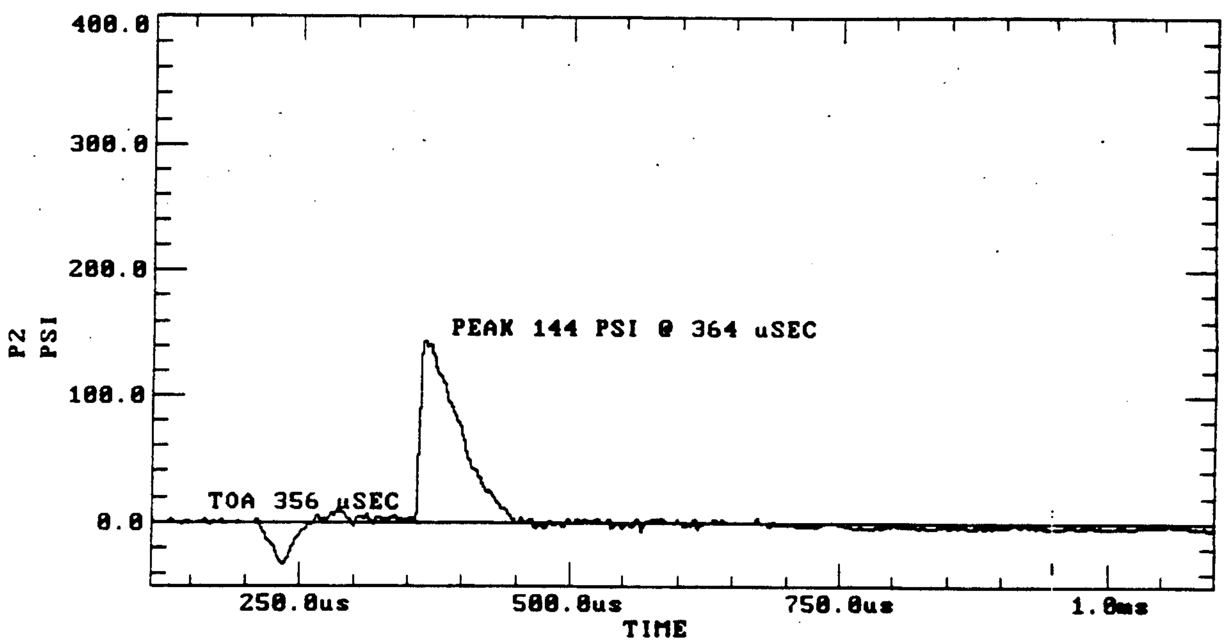

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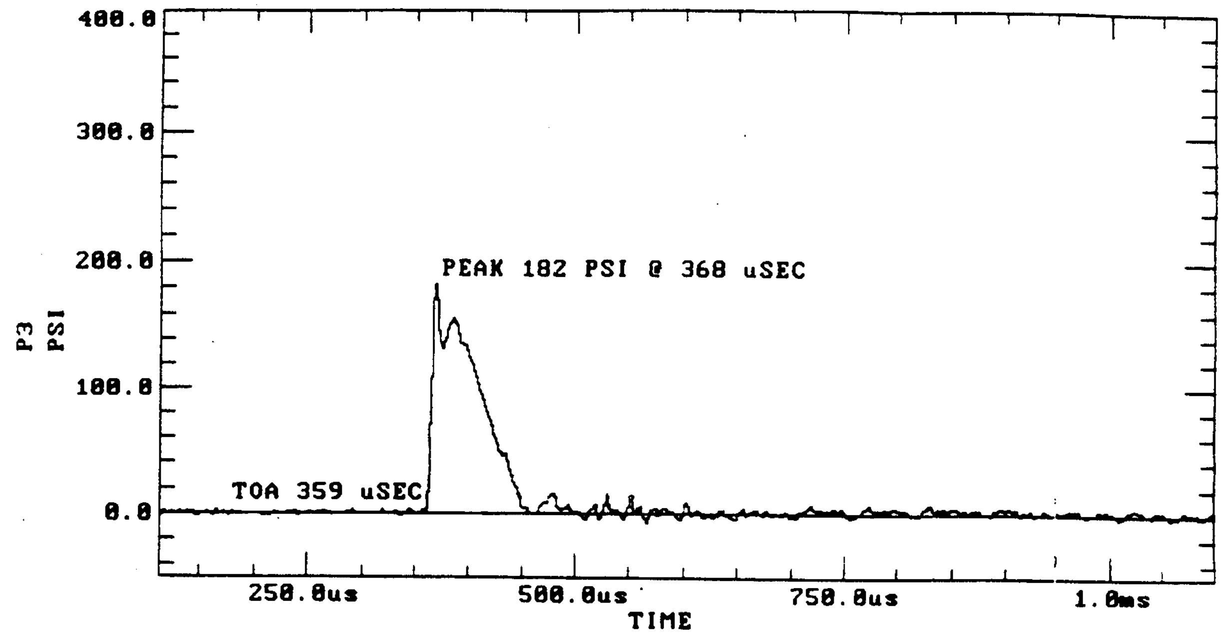

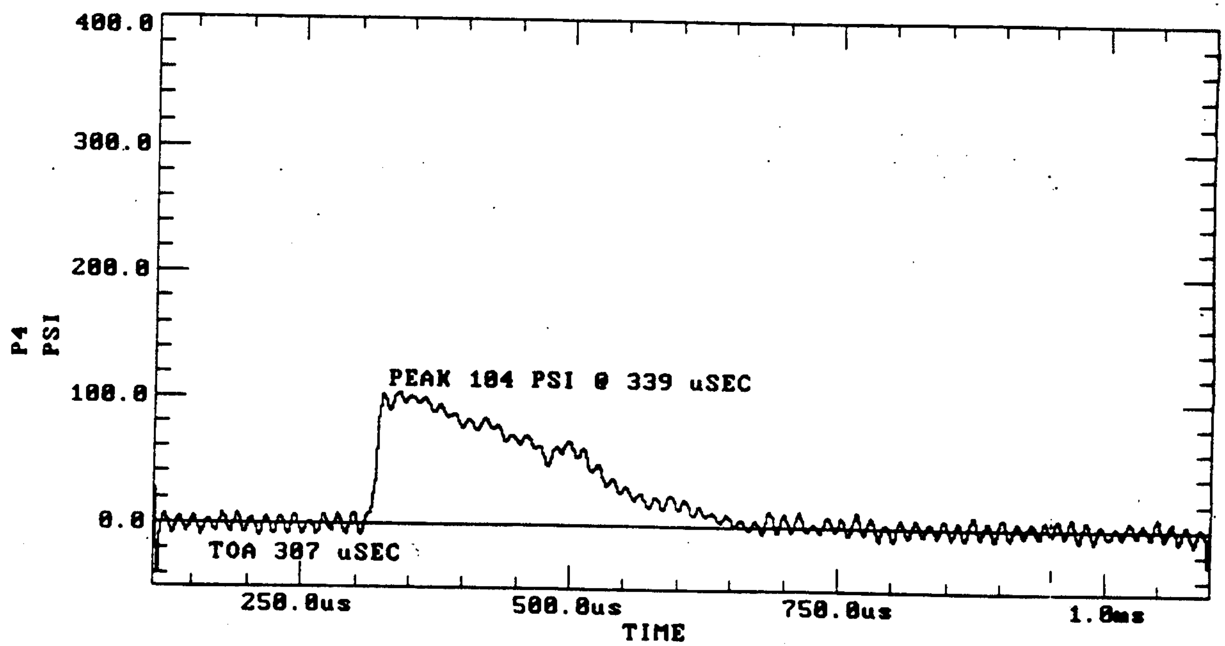

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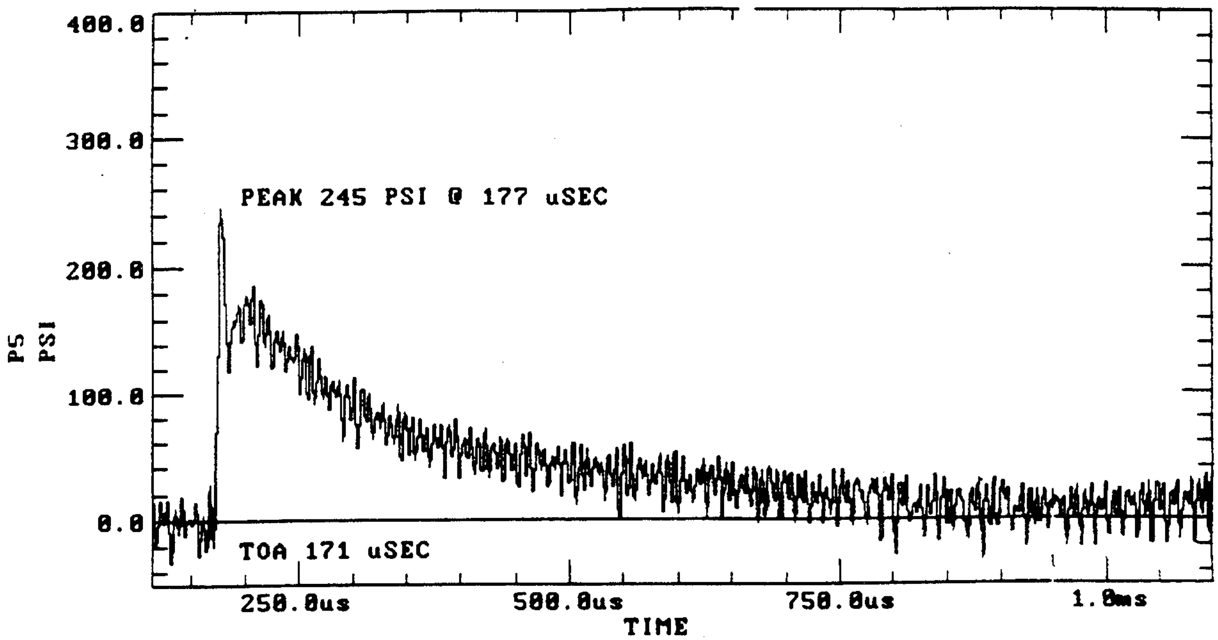

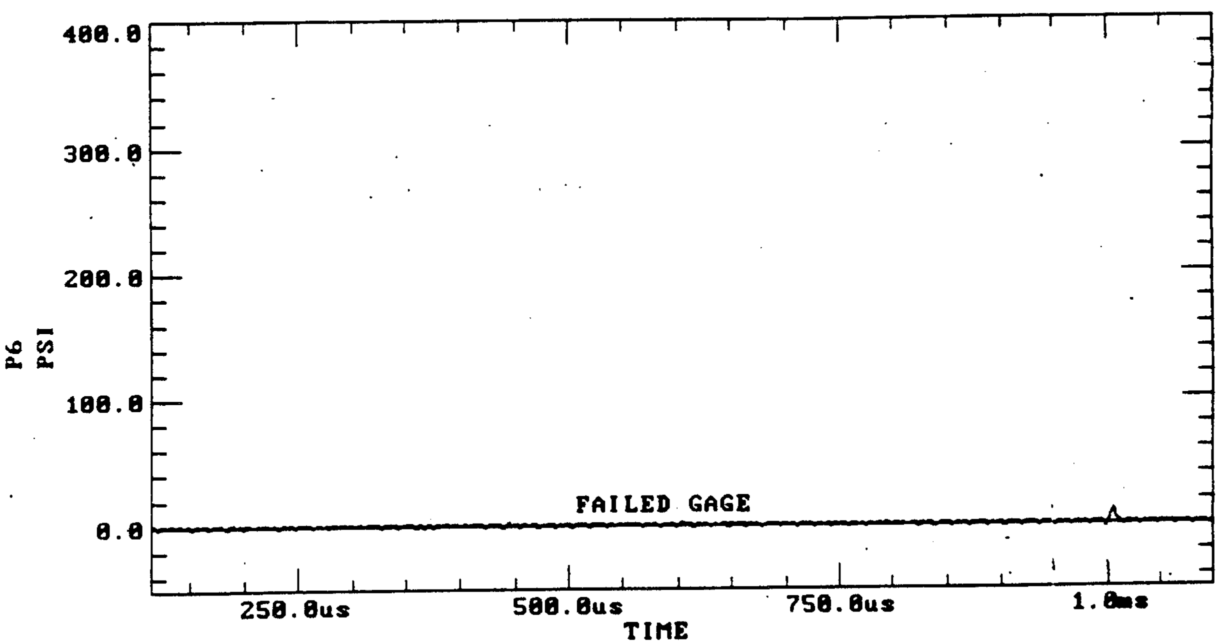

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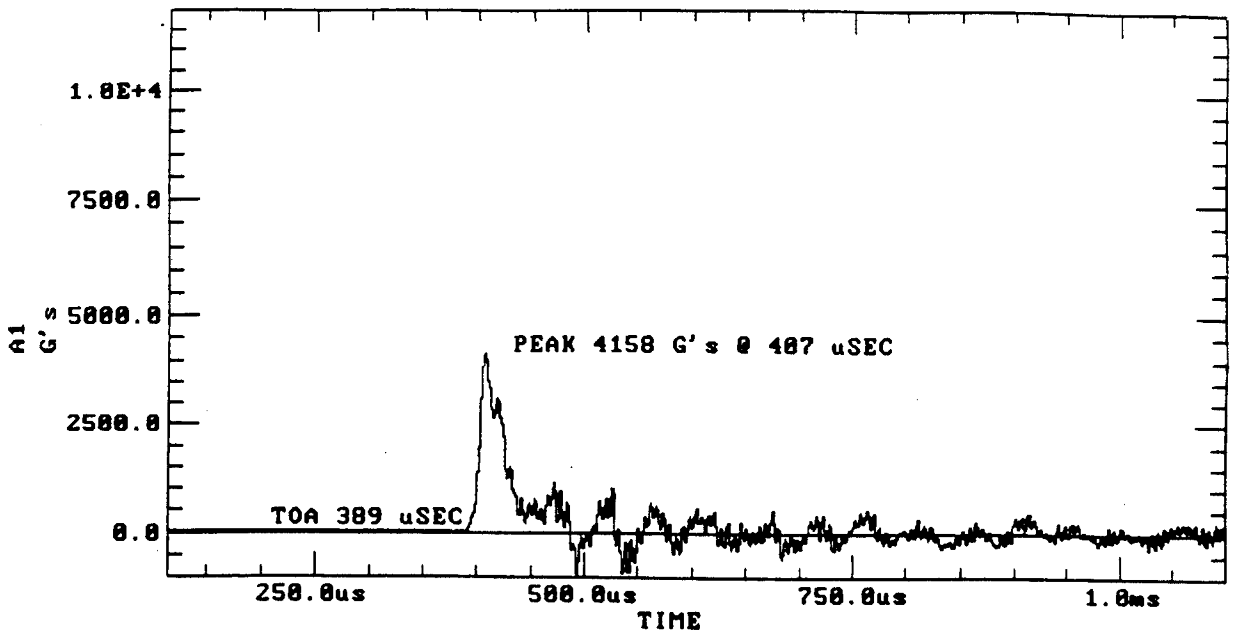

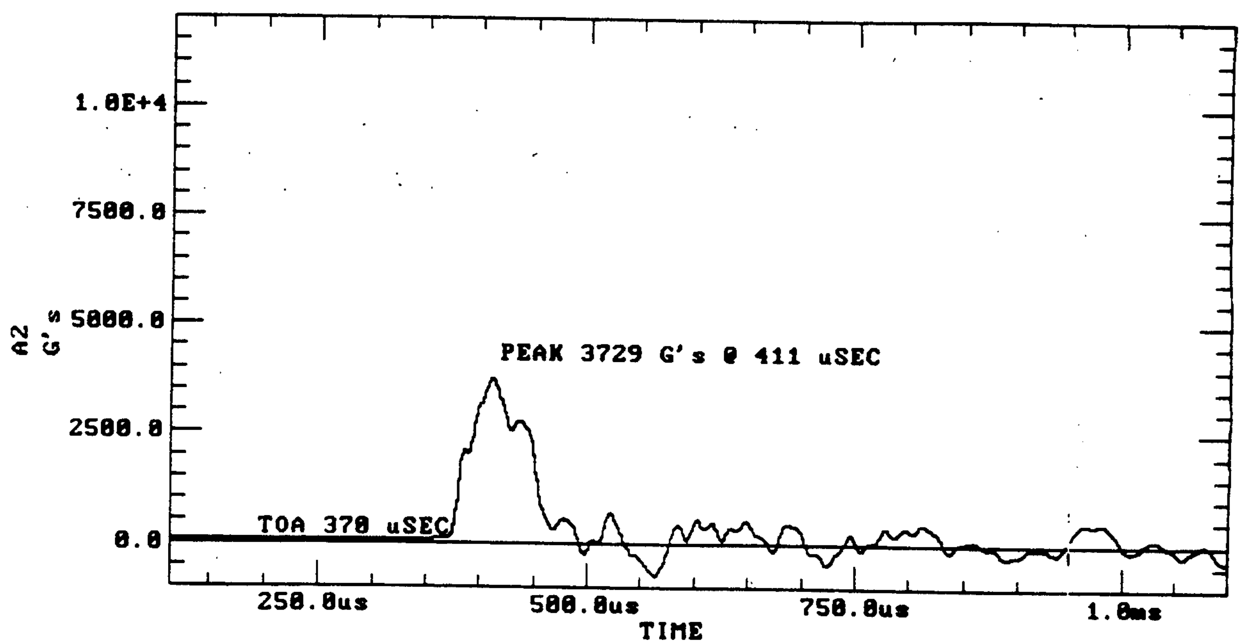

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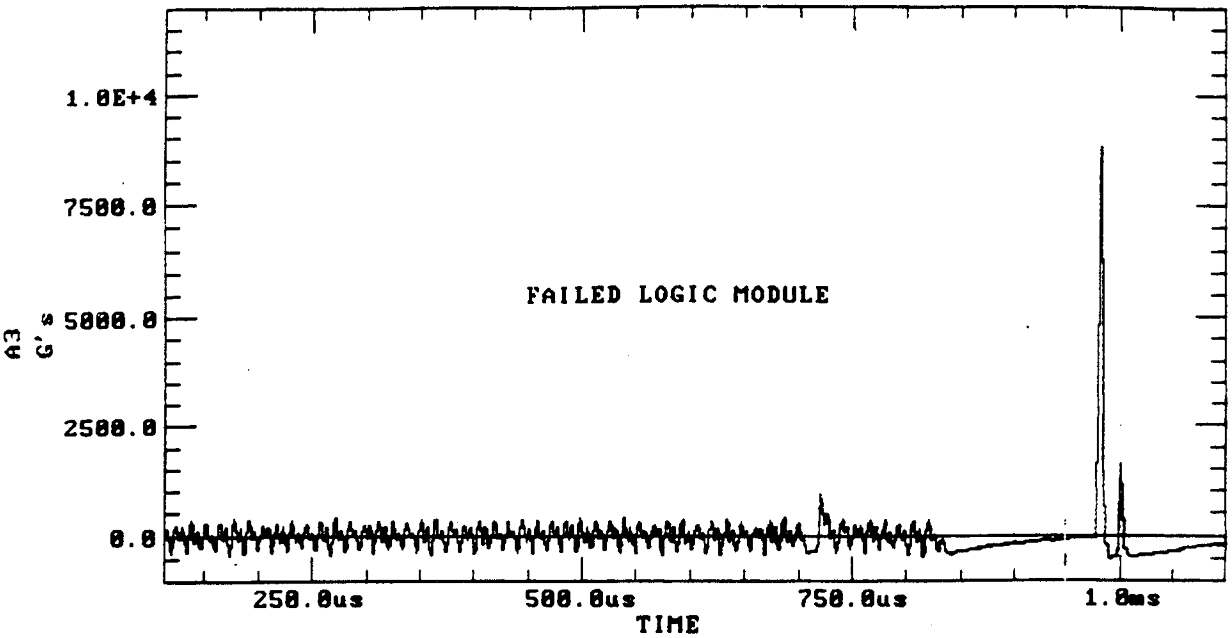

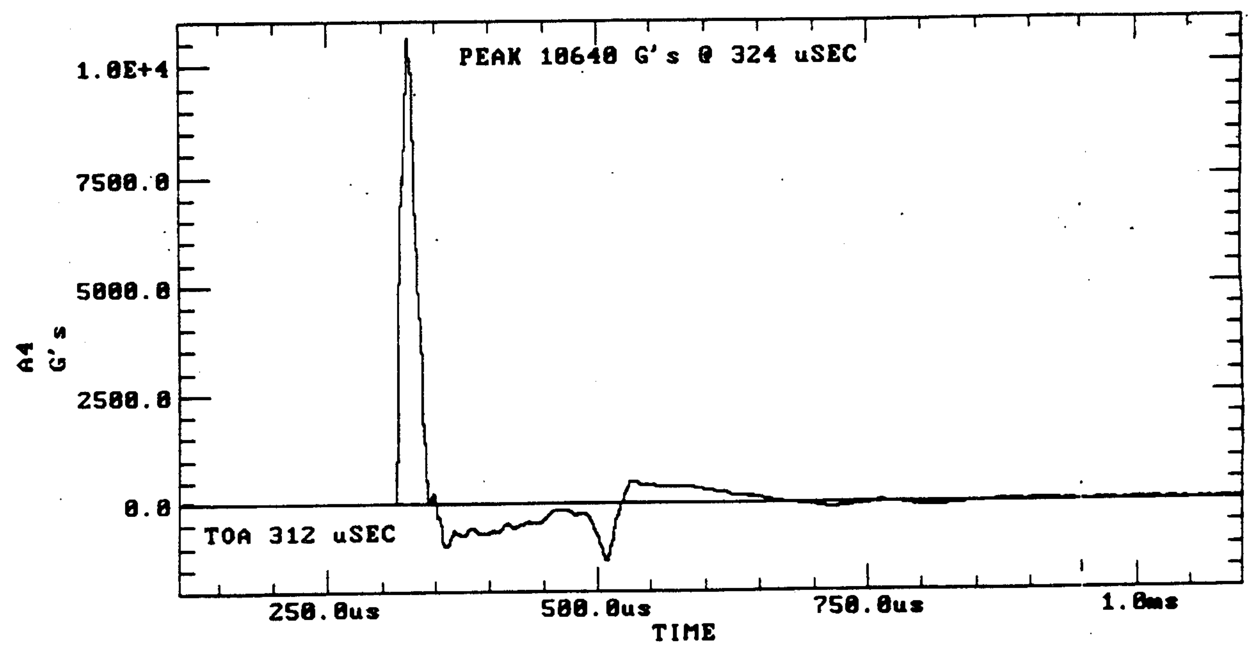

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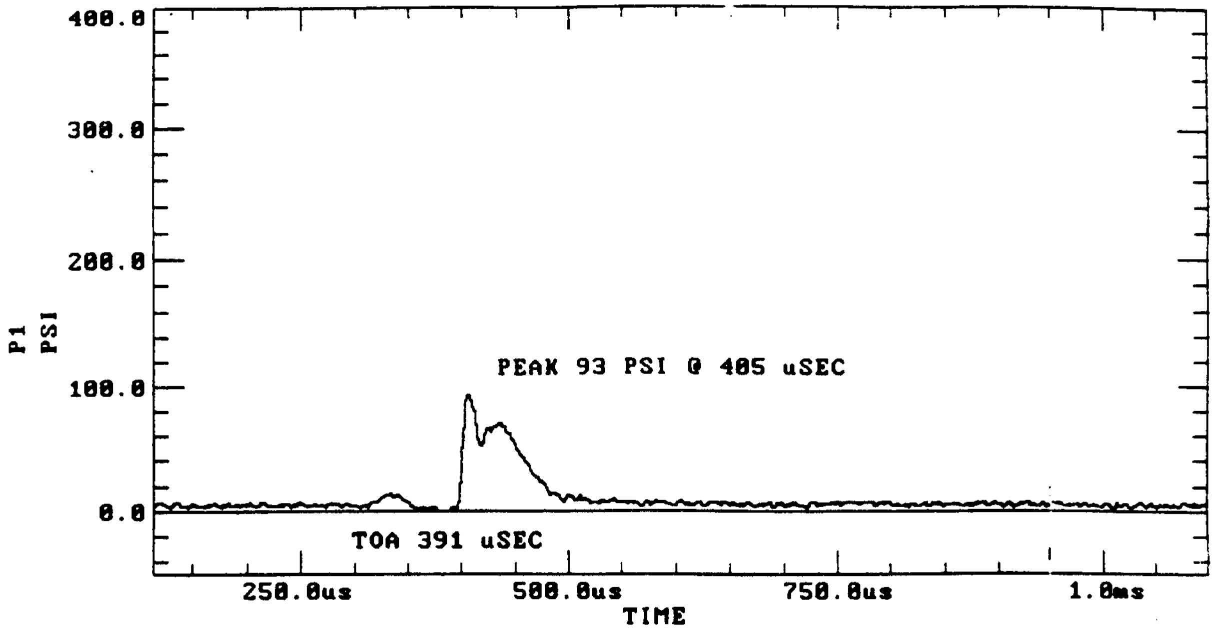

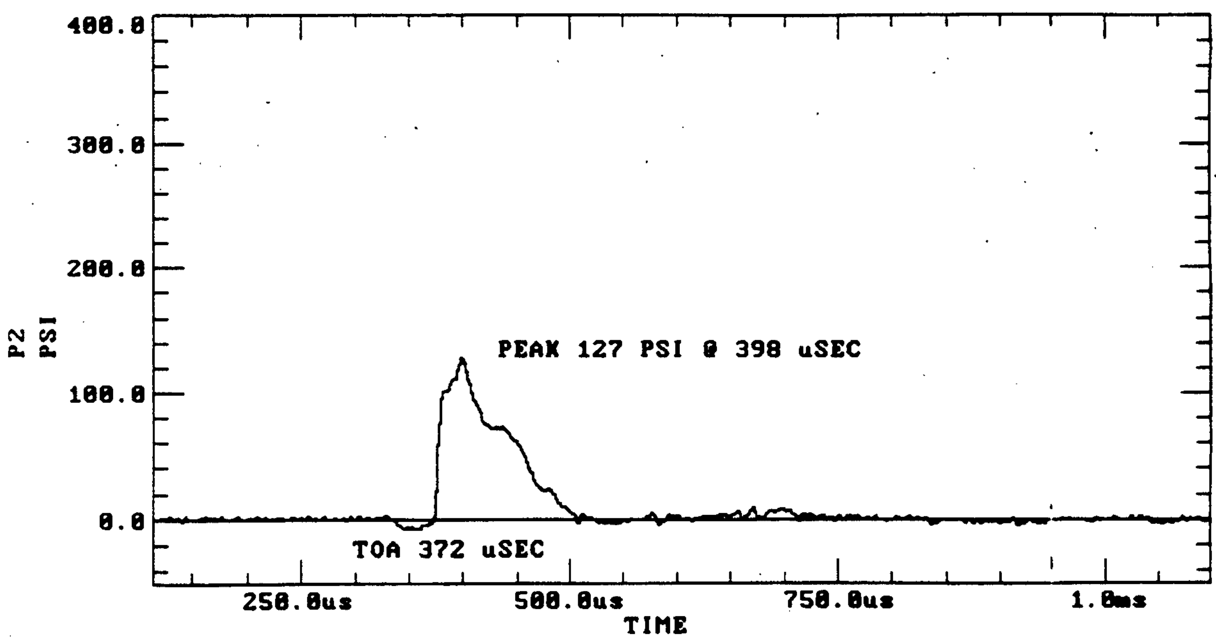

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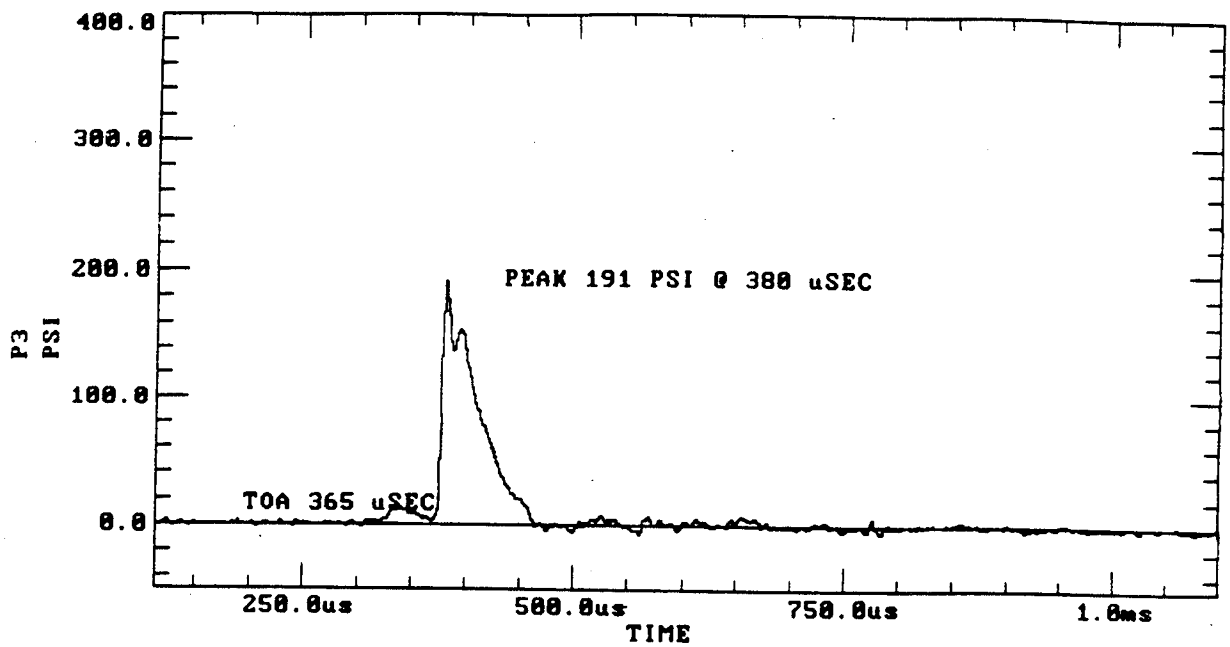

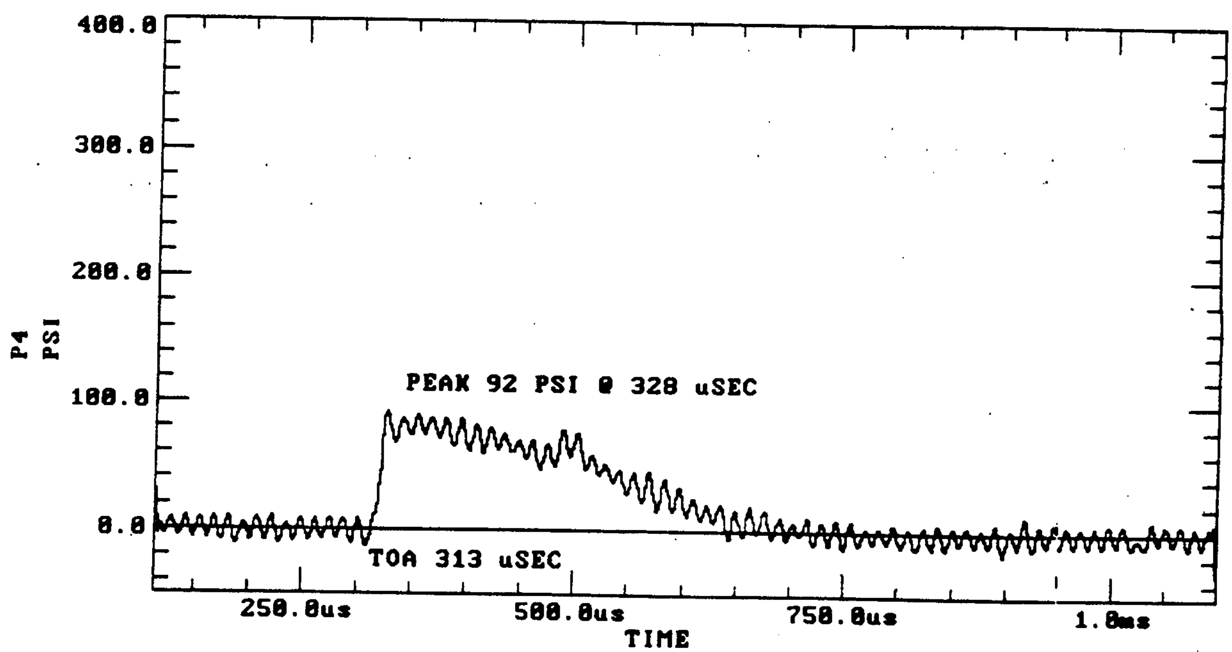

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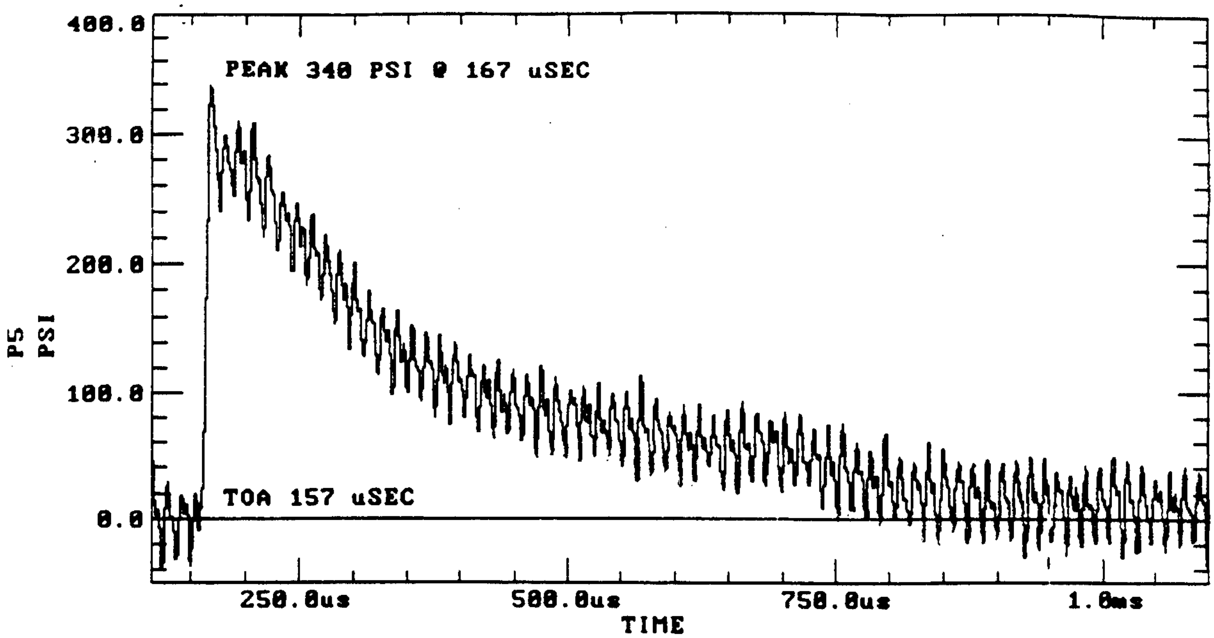

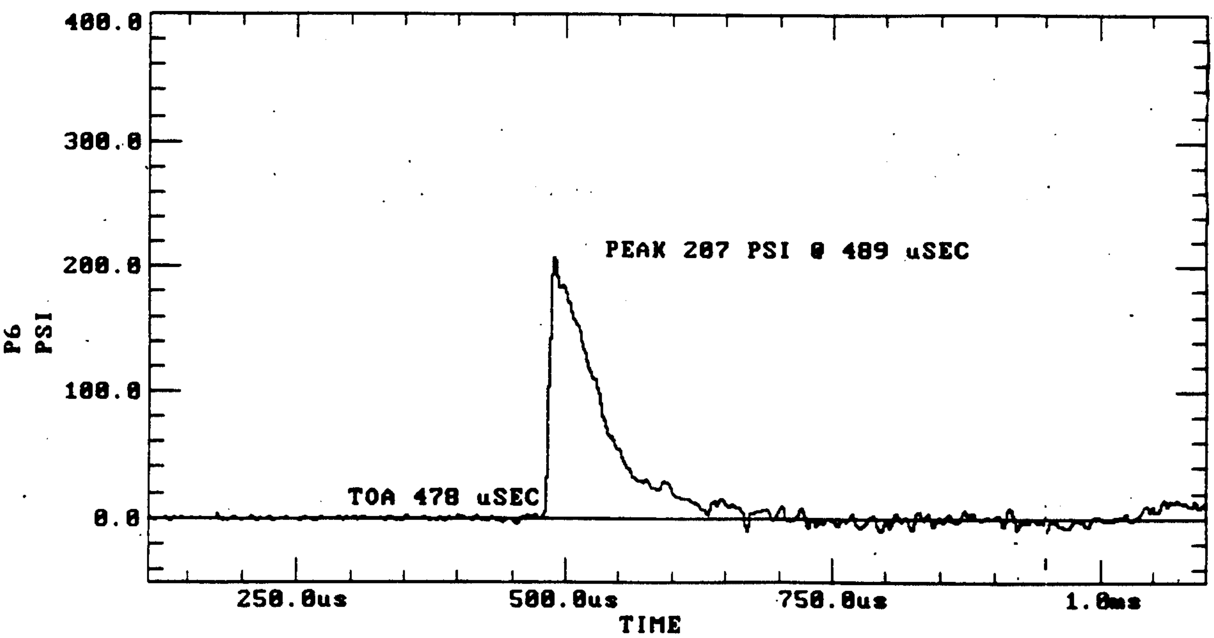

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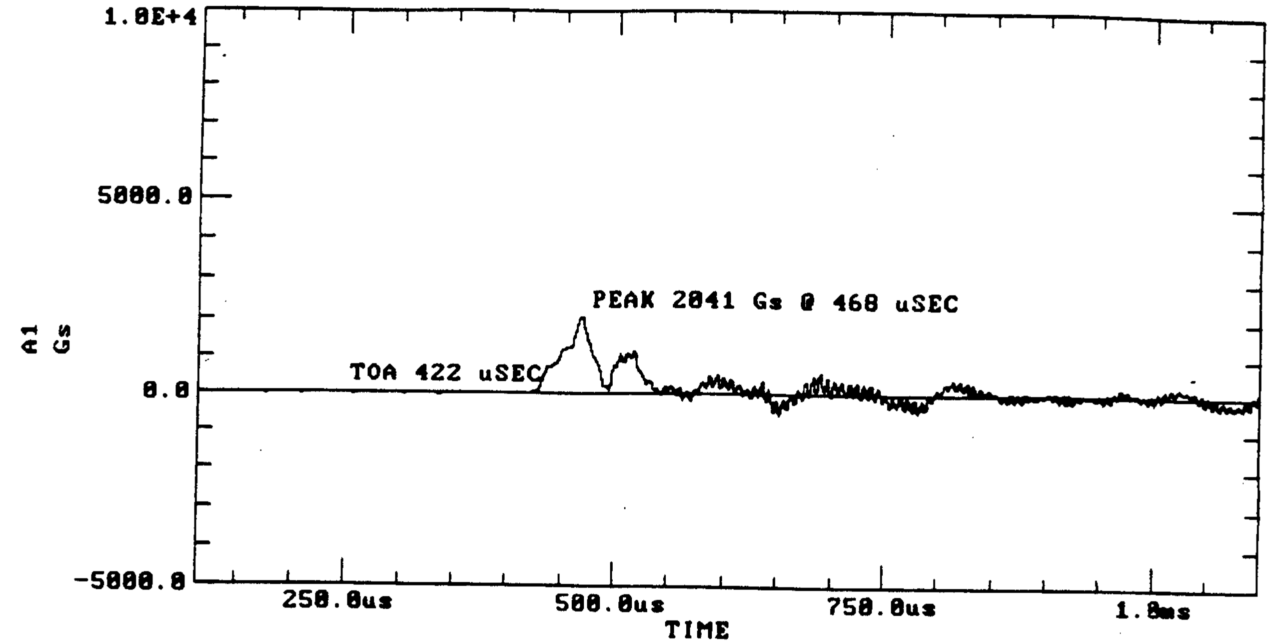

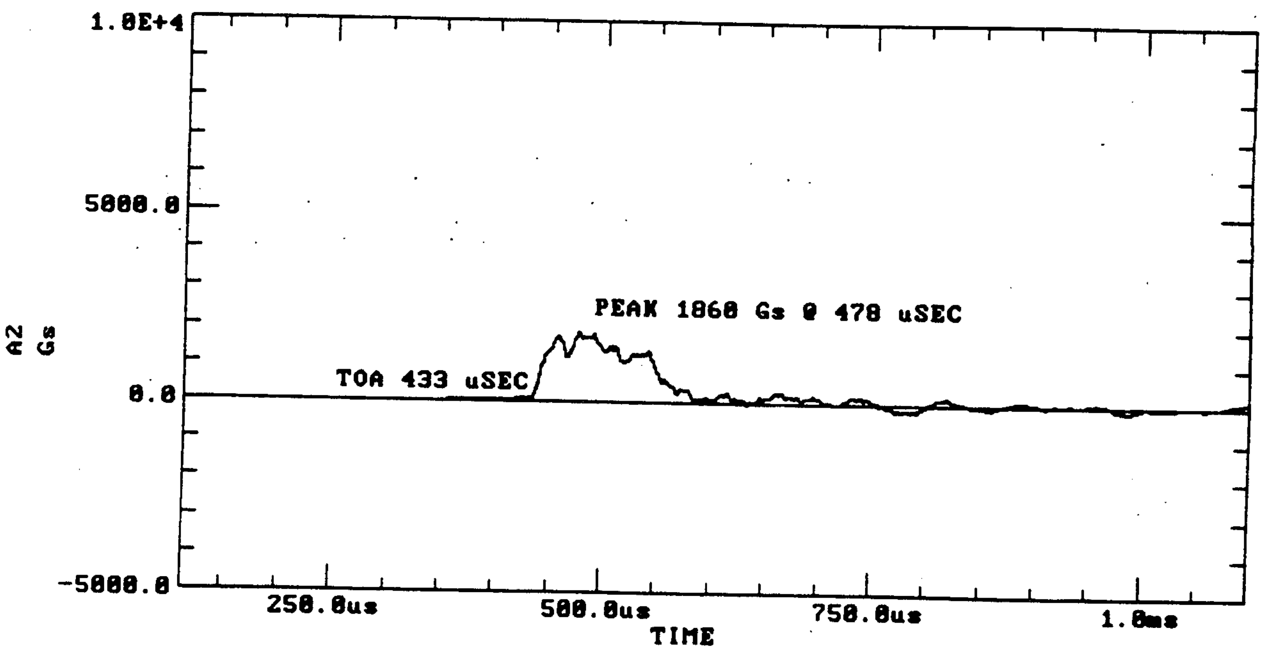

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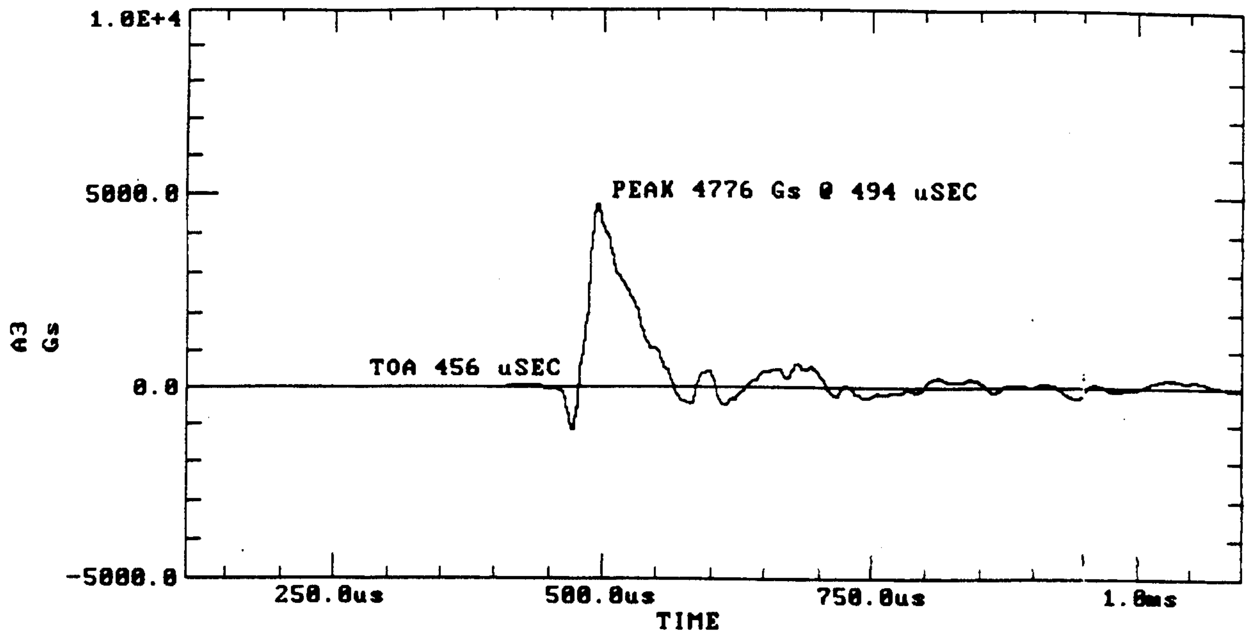

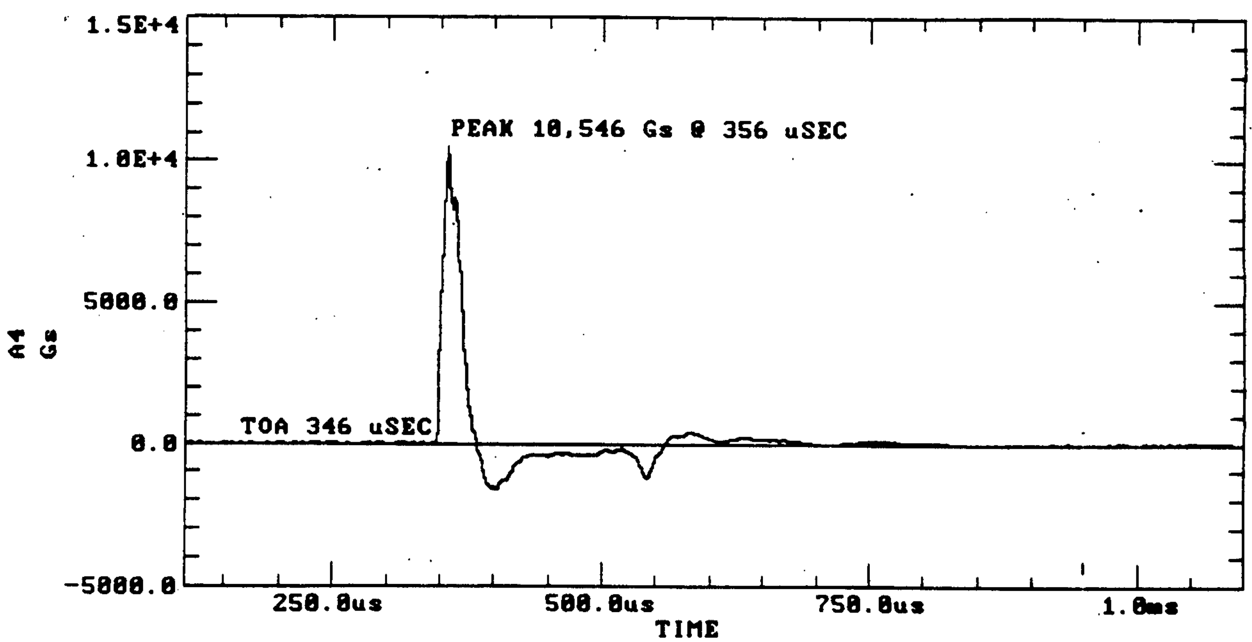

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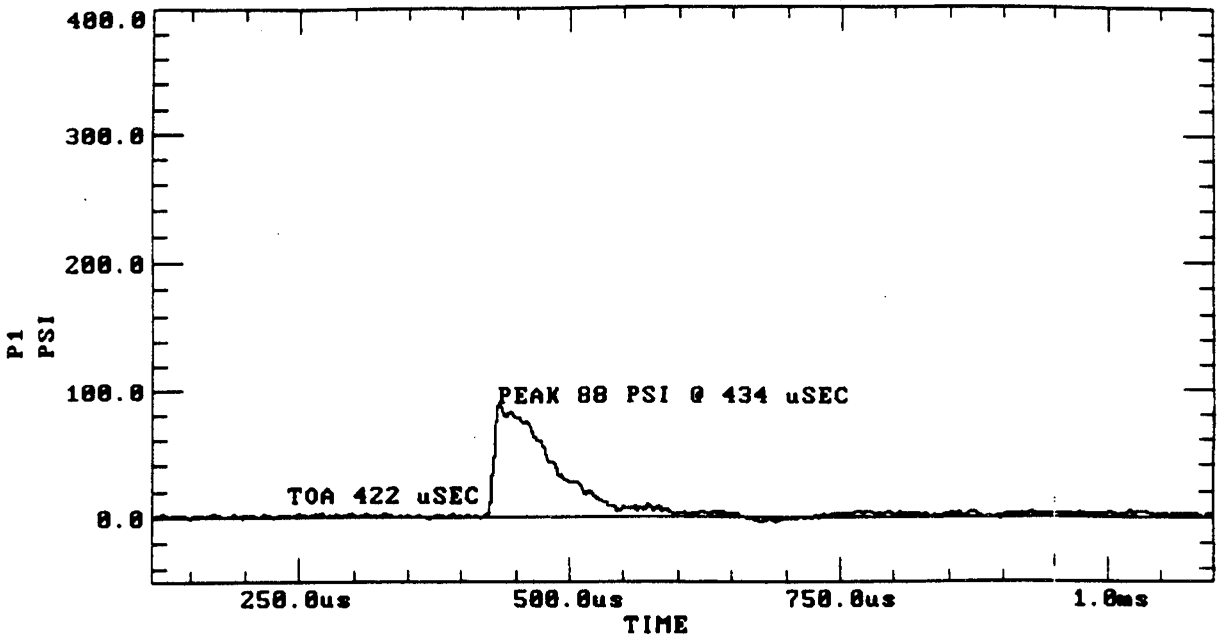

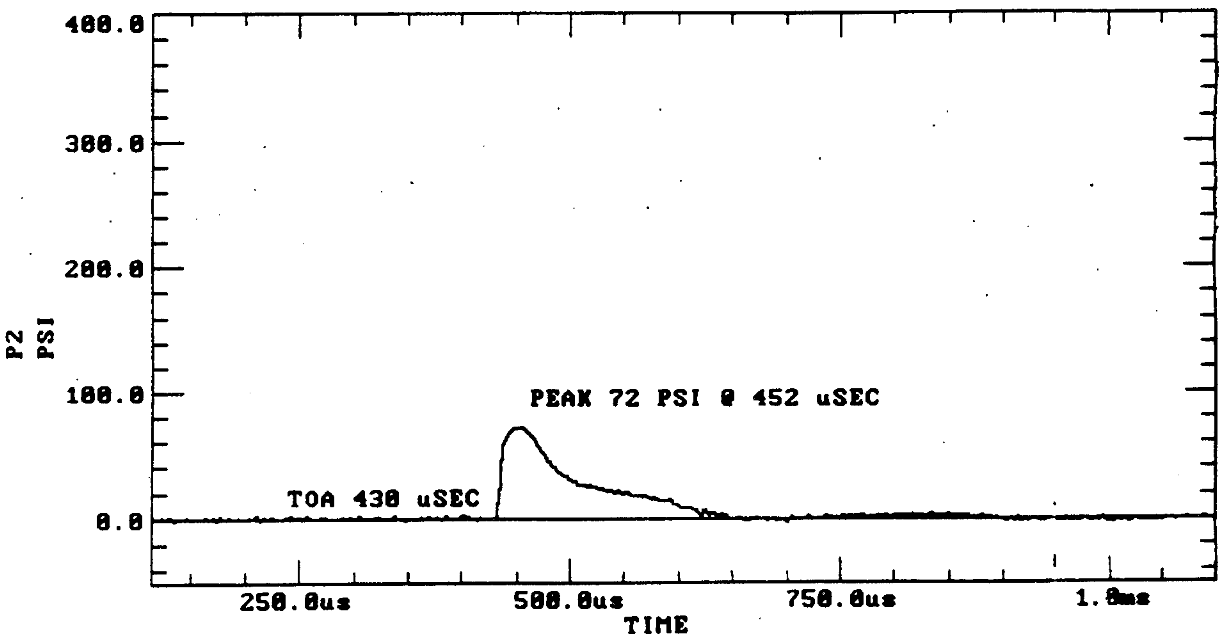

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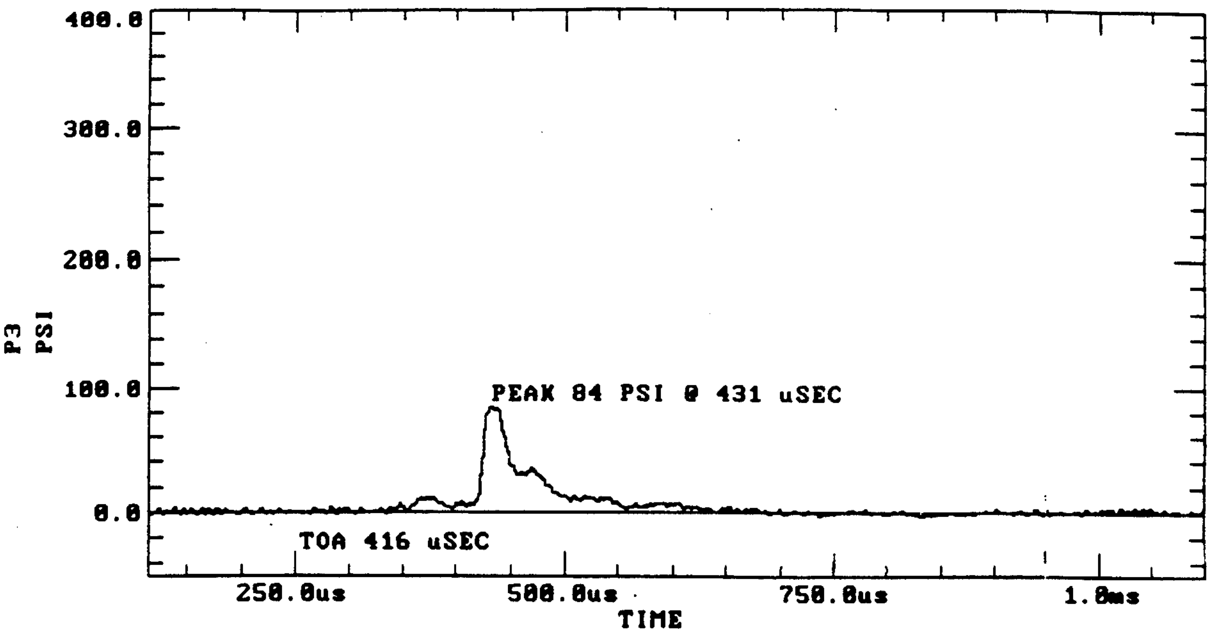

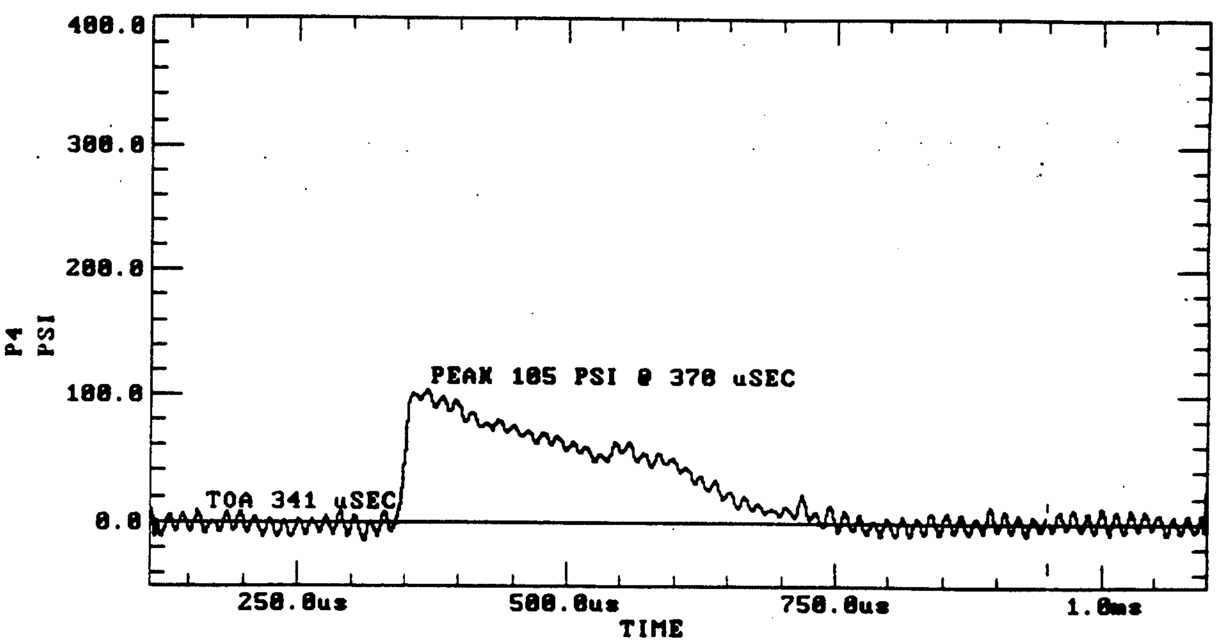

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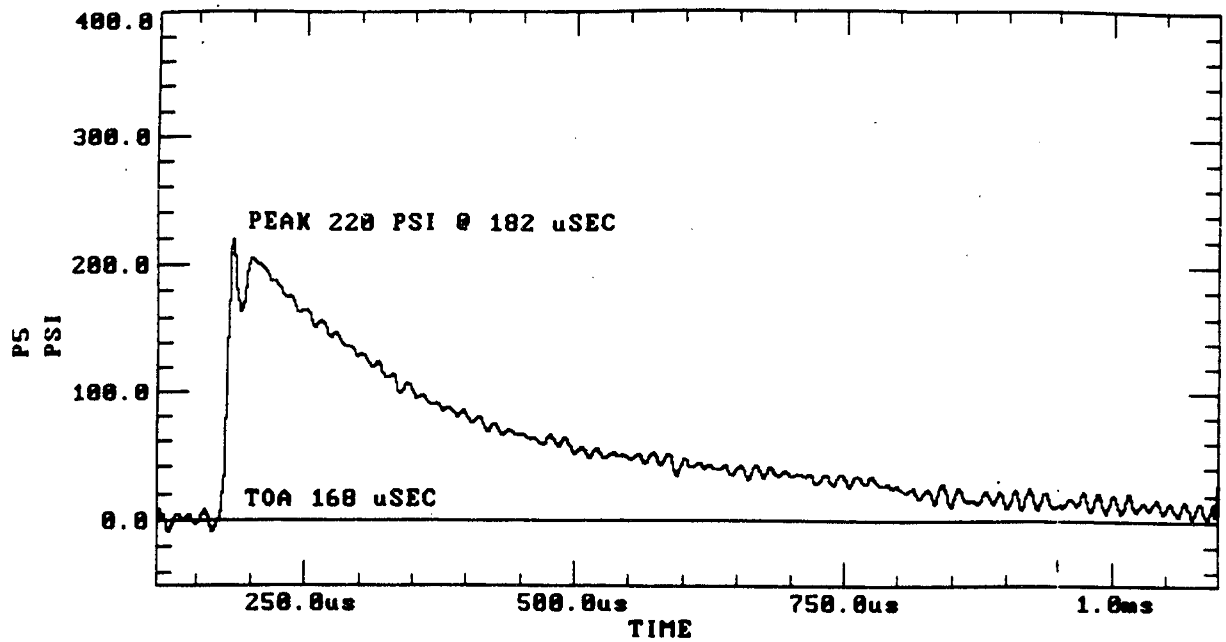

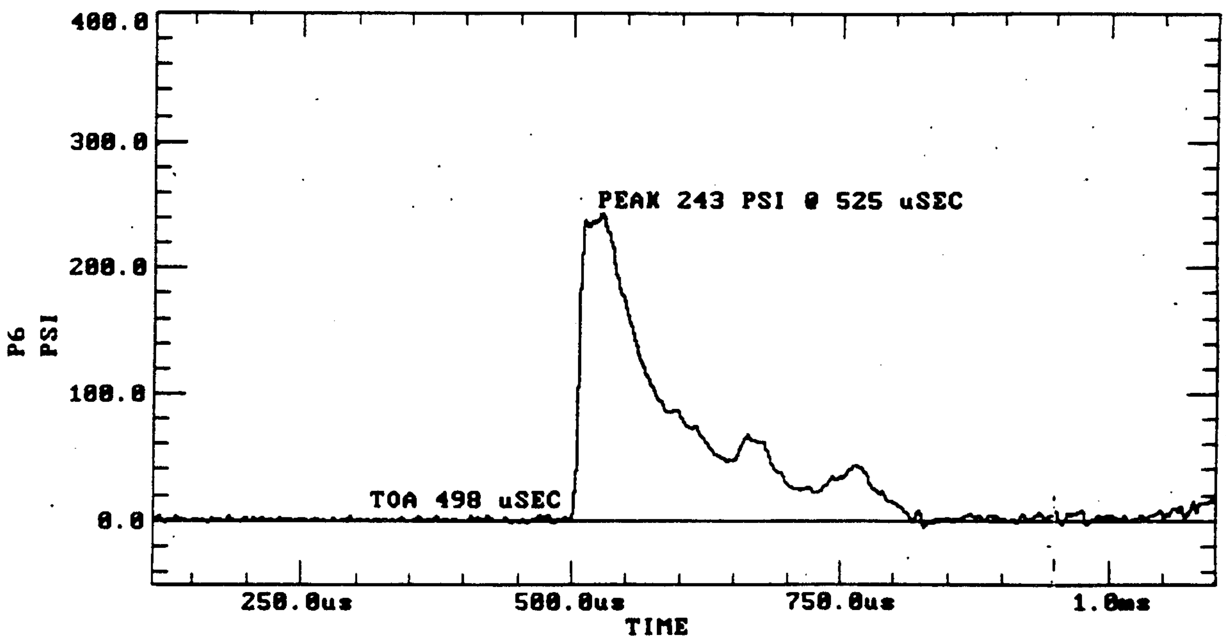

Test 32. Gage Data (continued)